TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
DEPARTMENT OF CIVIL ENGINEERING

FINAL YEAR PROJECT REPORT ON
"EARTHQUAKE RESISTANT DESIGN OF IRREGULAR MULTI-STOREYED RCC COMMERCIAL BUILDING"

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Supervisor:

Assoc. Prof. Dr. Kshitij Charana Shrestha

APRIL 2023

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

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CERTIFICATE

This is to certify that this project work entitled "**EARTHQUAKE RESISTANT DESIGN OF IRREGULAR MULTI STOREYED RCC COMMERCIAL 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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ACKNOWLEDGEMENT

ABSTRACT

As a part of the four year course of Bachelor's in Civil Engineering at Tribhuvan University in fourth year, Project I and II were included on the curriculum where we were given an opportunity to analyze and design a building by applying the theoretical knowledge that we obtained over the period of 4 years. In this project "Earthquake Resistant Design of Multi-Storied RCC Commercial Building", we idealized, analyzed and designed a commercial building abiding by IS codes and NBC codes.

The main aim of this project work was to synthesize comprehensively the knowledge gained during course work for a practical application of civil engineering discipline in real life.

Reinforced Concrete is the most commonly used construction material for building houses, complexes in Nepal. Structures built must provide a feeling of safety, comfort and must not harm residents living even during calamities like fire, earthquake etc. Hence, in this report all the measures are adopted to make the structure functional, stable, economic and safe. Codes like IS:456; 2000, IS:1893;2016, IS 13920-2016, NBC:105;2020 SP 16-1980, SP 34-1987 were used and software like AUTOCAD, ETABS, SAP etc were used for drawing, analyzing and modeling the structure.

Despite the resource and time limitation, the project has been completed successfully on time. With the hope that the design and drawings will be enough as per ductile design consideration and Codal provision, we have completed the report on the building project. Although every care has been taken to remove all the mistakes, yet some of them might have remained unnoticed. We will feel grateful if errors are brought or for any suggestions, comments or opinions.

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NOTATIONS & ABBREVIATIONS

α_x, α_y	BM coefficients for Rectangular Slab Panels
ϕ	Diameter of Bar, Angle of internal friction of soil
δm	Percentage reduction in moment
τc	Shear Stress in Concrete
$\tau c, max$	Max. shear stress in concrete with shear reinforcement
τbd	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 or b	Width or shorter dimension in plan
bf	Effective width of flange
d	Effective Depth
d′	Effective Cover
D	Overall Depth
Df	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
I	Importance Factor (For Base Shear Calculation)
I_{xx}, I_{yy}	Moment of Inertia (along x and y direction)
K	Coefficient of Constant or factor
k_a, k_p	Active and Passive Earth Pressure

K	Stiffness
L	Length of Member
l_{eff}	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
q, q_u	Permissible and Ultimate bearing capacity of soil
Q_i	Design Lateral Force in ith Level
Q	Stability Index
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	Design Seismic Base Shear
V	Shear Force
W_i	Seismic Weight of ith 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
MOI	Moment of Inertia

1. OVERVIEW

1.1   Introduction

In our project, "EARTHQUAKE RESISTANT DESIGN OF IRREGULAR MULTI-STOREYED RCC COMMERCIAL BUILDING" we have performed the structural analysis of a twelve storey RCC building based on strong column-weak beam mechanism where shear walls and columns are vertical members to resist lateral/earthquake forces acting in the building, making our design of earthquake resistant building possible. With the ever increasing population and commercialization of Kathmandu, multi-storey buildings have become the trend or demand of people. Hence, we opted to learn the analysis and design of multi-storeyed RCC buildings in our final year curriculum based project.

1.2 Theme of Project Work

Under this project work, we have undertaken computer aided structural analysis and design of multi-storey building with the main aim of acquaintance of knowledge, skill and emphasis on practical application of design procedures, utilization of analytical process. Familiarization with computer software and codal provisions is also another aim of the project.

1.3 Objectives

Following are the objectives of the project work:

1. Preparation of architectural plan for a building meeting all utility requirements of client.
2. Identification of structural elements of concern.
3. Modeling of the structure using ETABS.
4. Analysis of the structure under several load combinations.
5. Check building behavior under all codes' criteria.
6. Sectional design of structural members.
7. Preparation of detailed structural drawing of the design.

1.4 Scope of the Project

1. The project is limited to structural analysis and design only. The design of following structural elements is done:
   a. Slab
   b. Beam
   c. Column
   d. Staircase
   e. Mat Foundation
   f. Shear Wall
2. The bearing capacity of soil is not examined but assumed.
3. Design and layout of building services like electricity, sanitary system, water system etc. is omitted.
4. Economic considerations are not included.
5. Landscape design and parking detail design are not connected with this project.
6. Environmental and social attributes are not taken into consideration.

1.5 Code of Practice

1. **IS 456:2000** [Design of RCC Structures]
2. **IS 1893 (Part 1): 2016** [Earthquake-Resistant Design Criteria]
3. **NBC 105:2020** [Earthquake-Resistant Design Criteria]
4. **IS 13920:2016** [Ductile detailing of RCC structures subjected to earthquake load]
5. **IS 875 (Part 1): 1987** [To access dead load]
6. **IS 875 (Part 2): 1987** [To access live load]
7. **IS 1904: 1986** [For foundation design]
8. **IS 2950.1.1981 [**For foundation design]
9. **SP 16 and SP 34:** [Design aids and hand book]

1.6 Assumptions and Idealization in the Analysis of Earthquake Resistant RCC Building:

For simplicity, practicality, safety and ease, various assumptions and idealizations are adopted in the analysis and design process of this project. They are as follows:

1. Tensile strength of concrete is negligible.

2. Shrinkage and temperature strength are negligible.

3. Adhesion between concrete and steel is adequate enough to develop full strength.

4. Seismic load and wind load do not occur simultaneously.

5. The centerlines of beams, columns and shear walls are concurrent everywhere.

6. The values of elastic modulus of materials are taken from static analysis unless specified.

7. Resonance visualized under steady-state conditions for an earthquake may not occur.

8. Buildings are designed for frequently occurring earthquake loads rather than rare ones.

9. Direction of earthquake can be any, but in design, three components of acceleration i.e. one vertical and two horizontal components are considered.

10. Horizontal inertial forces due to earthquakes are dominant.

11. All the lateral forces generated by an earthquake are assumed to be transferred to the base as base shear and then transferred to the soil via foundation.

1.7 Characteristics of Earthquake Resistant Structures:

1. **Symmetricity of building:** Symmetrical buildings avoid concentration of loads at certain points, improving load transfer mechanism.

2. **Uniformity:** Shapes, distribution of beams, storey height and length of columns along the building affect the reaction of the building in an earthquake.

3. **Stiffness:** Structure should be stiff enough to withstand the load applied. Soft stories should be avoided.

4. Ductility: Its capacity to undergo large deformations under severe earthquake shaking even after yielding without failure. It improves design and detailing strategies.

5. Dimensions of building: Plinth area, height, width, and length determine the response of the building under load. Too long, too tall, or too wide buildings should be avoided.

6. Path of load transfer: Hurdles in load transfer like hanging columns, changes in dimension size along the structure, setbacks, and mass irregularity should be avoided in earthquake resistant buildings by providing concentration-free load transfer to the foundation and then soil.

1.8 SALIENT FEATURES OF THE PROJECT

Project name: Earthquake Resistant Design of Irregular Multi-Storeyed RCC Commercial Building

Physical Properties:

- Location: Ncell Tower, Kathmandu
- Total Number of Storey: 2 Basement, 1 Semi Basement and 12 Floor
- Total Height: 52.2 m
- Floor to Floor Height: 3 m for Basement and 3.6 m for floor
- Longest Length: 53.83m
- Longest Breadth: 37.58m
- Plinth Area: 1506 sq.m^2 (basement) and 640 sq m^2
- Earthquake Zone: V

Structural Properties:

- Total Number of Column: 489
- Sections of Columns: Circ. 600, Rect. 850*850, Rect. 1000*1000
- Sections of Beam: 525*700, 400*600
- Depth of Slab: 130 mm
- Foundation Depth: 2220 mm

- Geometry   Irregular

Some Values Adopted:

- Bearing Capacity of soil: 250 KN/m^2
- Unit Weight of concrete: 25 KN/m^3
- Unit weight of masonry: 19.6 KN/m^3
- Live load on staircase: 4 KN/m^2
- Live load on floor: 2.5 KN/m^2
- Live load on roof: 0 KN/m^2
- Seismic zone factor, Z: 0.35
- Response reduction factor   5 For SMRF

2. LITERATURE REVIEW

2.1. Background

An earthquake is a sudden and violent shaking of the ground caused by the release of accumulated stress energy in elastically strained rocks (tectonic plates) in the form of seismic waves. These plates, which are massive slabs of rock, move slowly and can become locked together due to friction. When the tension builds up and the plates suddenly shift, the energy released can create seismic waves that cause the ground to shake. Causes of earthquakes may be natural, like plate movements, rupture along weak rocks, or volcanic eruptions; they may also be non-natural, like the collapse of dams, huge landslides, mining, tunneling, explosions, nuclear weapons testing, etc. Plate tectonic theory or elastic rebound theory are used to describe the occurrence of earthquakes.

Earthquakes can occur anywhere in the world-intraplate or interplate earthquakes, but are most common in areas where tectonic plates meet, such as along the Pacific Ring of Fire. The severity of an earthquake is measured using the Richter scale, which assigns a numerical value to the amount of energy released by the quake, called the magnitude of earthquake. The scale ranges from 1 to 10, with each increase in value representing a tenfold increase in energy. Other scales like moment magnitude, body wave magnitude and surface wave magnitude are used as well. The intensity of earthquakes, i.e., the effect of earthquakes for different reasons is also measured using scales like the mmi scale.

The effects of an earthquake can be devastating. In addition to shaking the ground, earthquakes can trigger primary hazards like the collapse of structures, breakage of pipes, sewage laid on the ground, and other secondary hazards like landslides, tsunamis, fire, and geographical changes. Buildings, roads, and other infrastructure can be damaged or destroyed, and people can be injured or killed.

To reduce the risk of earthquake damage, engineers design buildings and other structures to be earthquake-resistant. This involves using materials and techniques that can absorb the energy of an earthquake without collapsing, and ensuring that buildings are properly anchored to the ground. Governments and emergency responders also prepare emergency plans and conduct

drills to help people prepare for earthquakes and respond quickly to their aftermath. In design, an approach is taken with codes for loads, acceleration coefficients, etc.

The philosophy of seismic design recognizes that earthquakes are inevitable and that the design of buildings should prioritize the safety of occupants in the event of an earthquake. The goal of ductile design is not to prevent damage to a building during large and rarest earthquakes but to ensure that the building remains standing and functional, protecting the lives of its occupants and minimizing the cost of repairs. **This fact suggests that the structure must be designed for frequently occurring earthquakes. For this, the structure must have the strongest vertical load bearing member i.e., a column which provides basic support to the structure. Hence, we are applying a strong column-weak beam concept.**

2.2 Design Philosophy

There are three philosophies for the design of reinforced concrete:

- ☐ Limit State Method
- ☐ Working Stress Method
- ☐ Ultimate Load Method

We have adopted a Limit state method for the design of the structural elements.

Limit State Method

The limit state method is a design approach used in structural engineering to ensure that a structure can withstand the loads and stresses it is likely to encounter over its intended lifetime. The method involves defining two types of limit states: ultimate limit state (ULS) and serviceability limit state (SLS).

The ultimate limit state refers to the state at which, if further load is applied, the structure is unable to withstand it and failure occurs. The design is based on the assumption that this state should not be reached during the intended lifespan of the structure. This involves calculating the maximum loads and stresses that a structure is likely to face and designing the structure to withstand these loads with an adequate safety margin.

The serviceability limit state refers to the state in which a structure is still able to support the loads and stresses placed upon it but may experience excessive deformation or deflection, which may affect the comfort or functionality of the structure. The design is based on the assumption that this state will be reached during the intended lifespan of the structure. This involves ensuring that the structure remains functional and serviceable even when subjected to the maximum anticipated loads and stresses.

The limit state method is widely used in the design of buildings, bridges, and other structures. It allows engineers to design structures that are safe, reliable, and efficient, and that can provide a long service life, making design economic.

"Following are the provisions and assumptions for Limit state design mentioned in IS 456:2000, Section 38 to Section 42"

Limit State of Collapse in Flexure

The limit state of collapse in flexure is a critical design consideration in structural engineering, particularly for reinforced concrete structures. In this limit state, the ultimate moment capacity of a beam or slab is reached, leading to excessive deformation, cracking, or failure.

The following assumptions are typically made for the limit state of collapse in flexure:

1. Linear-elastic behavior: The behavior of the structure up to the limit state is assumed to be linear-elastic, meaning that stress and strain are proportional and reversible.

2. Plane sections remain plane: It is assumed that the cross-section of the beam or slab remains plane even after deformation, meaning that any deformation occurs only in the direction perpendicular to the plane of the section.

3. Strain distribution: The strain distribution in the reinforced concrete cross-section is assumed to be linear and symmetric about the neutral axis.

4. Yielding of steel reinforcement: It is assumed that the steel reinforcement has yielded completely before the concrete reaches its ultimate compressive strain. This ensures that the full strength of the reinforcement is utilized.

5. No bond slip: It is assumed that there is no slip between the steel reinforcement and the surrounding concrete. This ensures that the reinforcement is fully effective in resisting the applied bending moment.

6. No buckling of compression reinforcement: It is assumed that the compression reinforcement does not buckle under compression. This is ensured by providing sufficient reinforcement and maintaining an adequate spacing between the reinforcement bars.

7. No shear failure: It is assumed that the structure does not fail in shear before reaching the limit state of collapse in flexure. This is ensured by designing the structure to resist both flexure and shear, and by ensuring that the shear reinforcement is adequately provided.

(a) Section (b) Strain diagram (c) Stress diagram
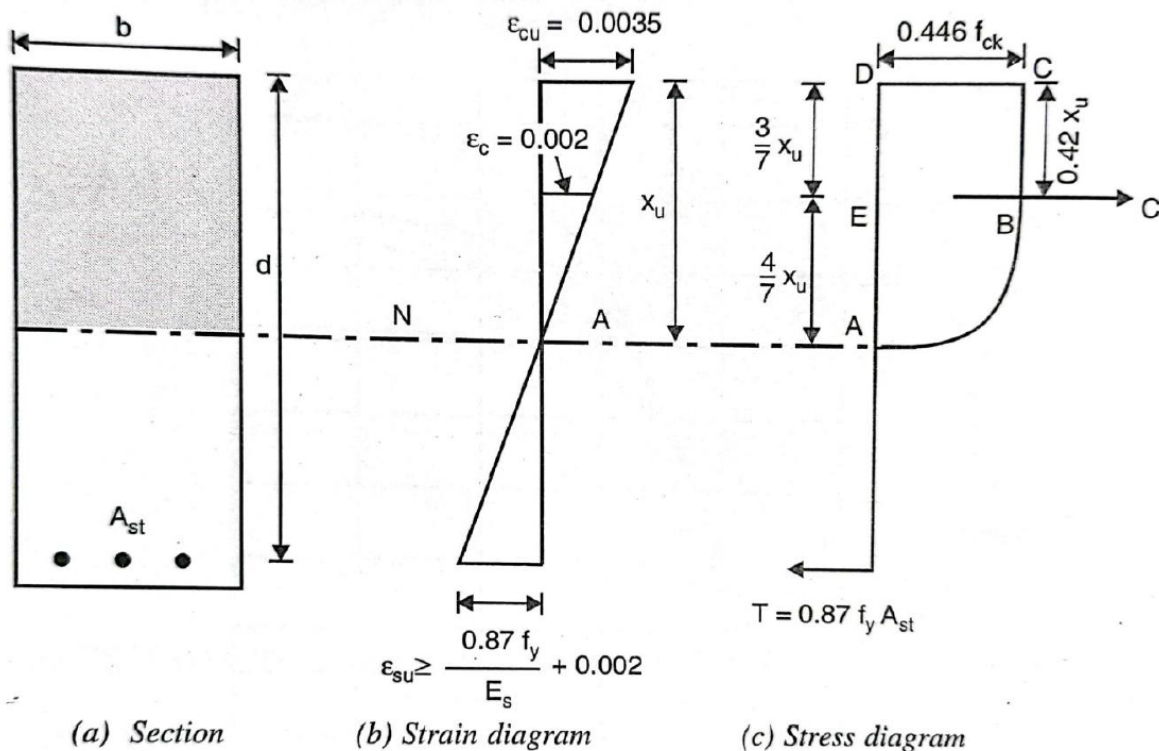
Source: IS 456:2000

Fig: 2.1 Stress-Strain Block diagram
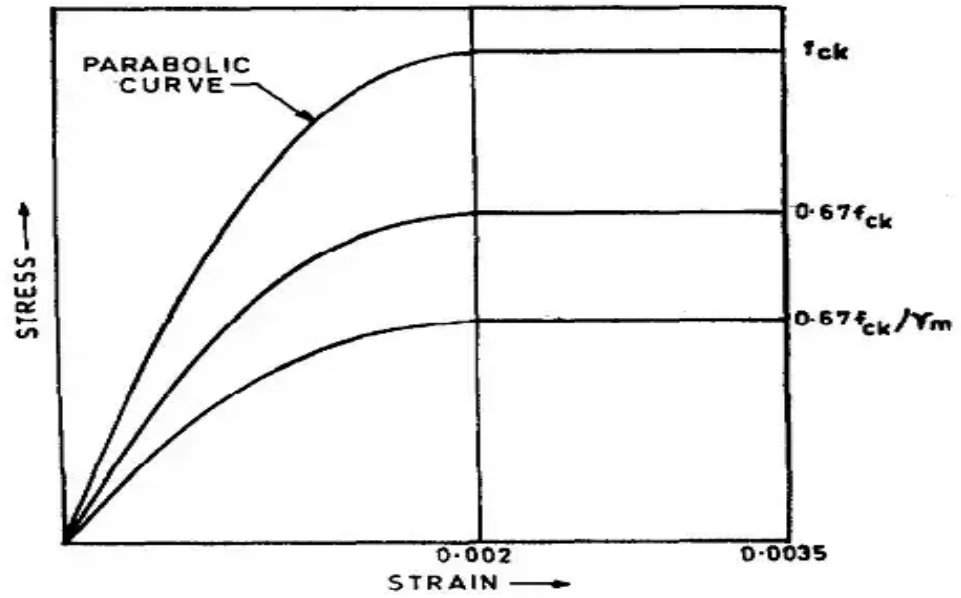
Fig 2.2: Stress- Strain curve for concrete

The Limit State of Collapse in Compression

The limit state of collapse in compression is a critical design consideration in structural engineering, particularly for columns and other compression members. In this limit state, the ultimate compressive strength of the member is reached, leading to excessive deformation, buckling, or failure.

The following assumptions are typically made for the limit state of collapse in compression:

1. Linear-elastic behavior: The behavior of the structure up to the limit state is assumed to be linear-elastic, meaning that stress and strain are proportional and reversible.

2. Perfectly straight and initially true: It is assumed that the member is initially straight and true before any load is applied. Any deviation from this is considered an initial imperfection, which may reduce the load-carrying capacity of the member.

3. Uniform and concentric load: It is assumed that the load on the member is uniform and concentric. Any eccentricity in the load or the member may cause bending and additional stresses, reducing the compressive strength of the member.

4. No material imperfections: It is assumed that the material properties of the member, including strength, stiffness, and ductility, are consistent and uniform. Any defects or inhomogeneities in the material may reduce the load-carrying capacity of the member.

5. No lateral instability: It is assumed that the member does not buckle laterally under compression before reaching the ultimate strength. This is ensured by designing the member to have a sufficient slenderness ratio, which is the ratio of its effective length to its lateral dimension.

6. Adequate confinement: It is assumed that the concrete in the member is adequately confined, either by the surrounding reinforcement or by other means. This ensures that the concrete can withstand the compressive stresses and does not crush or spall.

7. Sufficient reinforcement: It is assumed that the reinforcement in the member is sufficient to resist the tensile stresses caused by the compressive forces. This is ensured by designing the member to have a minimum amount of reinforcement, typically expressed as a percentage of the cross-sectional area.

The limit state of serviceability

The limit state of serviceability is a design consideration in structural engineering that ensures that a structure remains functional and meets certain performance criteria under normal service loads and conditions, without causing discomfort or inconvenience to its users. This limit state is concerned with the ability of the structure to perform its intended function while satisfying certain serviceability requirements.

The following are some examples of serviceability limit states:

Deflection: The deflection of a structure, such as a beam or a slab, should be limited to a certain maximum value to ensure that it remains functional and does not cause discomfort to its users. Excessive deflection may also affect the appearance and aesthetics of the structure.

Vibration: The vibration of a structure, such as a bridge or a tall building, should be limited to a certain frequency and amplitude to ensure that it does not cause discomfort or inconvenience to its users. Excessive vibration may also affect the safety and stability of the structure.

Cracking: The cracking of a structure, such as a concrete wall or a pavement, should be limited to a certain width and spacing to ensure that it remains functional and aesthetically acceptable. Excessive cracking may also affect the durability and long-term performance of the structure.

Service loads: The service loads on a structure, such as dead loads, live loads, wind loads, and earthquake loads, should be within certain limits to ensure that the structure remains stable and does not suffer from excessive deformation or stress. Excessive loads may also affect the safety and serviceability of the structure.

Durability: The durability of a structure, such as its resistance to corrosion, degradation, and environmental effects, should be ensured to ensure that it remains functional and serviceable over its intended design life. Insufficient durability may affect the maintenance and repair costs of the structure and reduce its overall service life.

The limit state of serviceability is usually checked using analytical calculations or computer simulations, based on appropriate design codes and standards. It is important to ensure that a structure meets the serviceability requirements to ensure its long-term performance, user satisfaction, and overall functionality.

A. **Control of deflection**

The deflection of a structure or part thereof shall not adversely affect the appearance or efficiency of the structure, finishes, or partitions. Two methods are given in the 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
- The calculation of deflection given in Appendix C of IS 456-2000 is to be followed in special cases**.**

B. Control of Cracking

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

- Expression for crack width and spacing (Annex F of IS: 456-2000).
- Allowable crack widths under different service conditions with due consideration to corrosion and durability of concrete (clause no. 35.l3.3 of IS: 456-2000).
- Unless the calculation of crack widths shows that a greater spacing is acceptable, for the flexure members in normal internal or external conditions of exposure, the maximum distance between bars in tension shall not exceed the value as given in IS: 456-2000, clause no. 26.3.3.
- Cracks due to bending in compression member subjected to design axial load > 0.2 f_{ck} * A need not be checked. For flexural members (a member that is subjected to a design load 0.2 f_{ck} * Ac), if greater spacing of reinforcement as given in clause 26.3.3, IS 456-2000 is required, the expected crack width should be checked by formula given in Annex F of IS 456-2000.

C. Control of Vibration

A dynamic load is any load of which the magnitude, direction or position vary with time, and almost any RCC structural system may be subjected to one form or another loading during its lifetime. Similarly, the structural response, i.e., The resulting stresses or deflections, is also time varying or dynamic and is expressed in terms of displacements. The limit state concept of designing reinforced concrete structures takes into account the probabilistic and structural variation in the material properties, loads, and safety factors.

D. Ductility

Ductility is the ability of a material or a structure to undergo significant deformation or plastic deformation without failing or breaking. In structural engineering, ductility is an important property that helps to ensure the safety and reliability of a structure in the event of an extreme loading condition, such as an earthquake or a windstorm.

The requirement for ductility in a structure can be explained as follows:

1. Energy Dissipation: During an extreme loading condition, a ductile structure can absorb a significant amount of energy through plastic deformation, which helps to reduce the impact of the loading and prevent catastrophic failure. Without sufficient ductility, a structure may experience brittle failure, which can lead to collapse or severe damage.

2. Redundancy: A ductile structure with redundant elements can provide multiple load paths for the transfer of loads and help to avoid sudden collapse or failure. In contrast, a brittle structure with no redundancy may experience sudden and catastrophic failure when one element fails.

3. Post-Earthquake Performance: In earthquake-prone regions, a ductile structure can provide better post-earthquake performance by resisting lateral loads and avoiding progressive collapse. A brittle structure, on the other hand, may suffer severe damage or collapse during an earthquake.

4. Construction Tolerance: Ductility also provides construction tolerance, as small imperfections and variations in the construction process can be absorbed by the structure without causing significant damage or failure.

5. Flexibility: A ductile structure can also provide greater flexibility in the design and construction process, as it allows for more freedom in the selection of materials and the design of elements. It also allows for greater adaptation to changing environmental conditions and loading scenarios."

2.3 Loads

Loads refer to the forces, weights, and other external agents that act on a structure, developing internal stresses, strain in them which influences its behavior. Loads are a vital consideration in the design of any structure, as they determine the size, strength, and materials required for the various structural components.

Some common types of loads that structural engineers must consider include:

1. Dead loads: These are the permanent, static loads due to the self-weight of the structure itself, fixed equipment, and other permanent features.

2. Live loads: These are the dynamic loads that can vary in magnitude and location over time, such as the weight of people, furniture, vehicles, and other movable objects.

3. Wind loads: These are the drag and lift loads due to wind flow against a structure, with varying magnitude depending on factors such as wind speed, direction, and topography.

4. Seismic loads: These are the loads created due to seismic activities, such as earthquakes, which can cause shaking and movement in a structure.

5. Snow loads: These are the loads created by the weight of snow on a structure, which can vary depending on factors such as snowfall rate, depth, and moisture content.

6. Thermal loads: These are the loads created by changes in temperature, which can cause expansion or contraction in a structure.

7. Impact loads: These are the loads created by sudden impacts, such as those caused by vehicles, falling objects, or explosions.

Structural engineers must carefully consider all of these loads when designing a structure to ensure that it can safely withstand the expected forces and stresses over its intended lifespan.

2.3.1 Design Loads

Using code IS 456-17 and IS 875-8.1, combinations of loads given are used to determine the size of the structural element. Buildings are subjected to a number of loads, forces and effects during their service life. In our project we have considered:

a) Dead load (DL)

b) Imposed load (IL)

c) Earthquake load (EL)

The following are the causes that generally cause internally equilibrated stresses forming crack in structure, but not collapse.

a) Foundation movement,

b) Axial elastic shortening,

c) Shrinkage,

d) Temperature changes, etc.

Beside the above- mentioned loads, the effect of following loads should also be considered in the design of structure.

a) Fatigue

b) Construction loads

c) Accidental loads

d) Impact and collision

e) Explosions

f) Fire, etc.

2.3.2 Load Assessment

Load assessment is an essential part of the structural design process. It helps to ensure that a structure is safe, stable, and able to withstand the forces that acting on it over its lifetime. The prime purpose of load assessment is to determine the magnitude and distribution of loads that a structure is likely to experience over its lifetime, and to ensure that the structure is designed to withstand these loads without failure or excessive deformation. Once the loads have been identified, we then analyze the structure using various mathematical models and computer simulations to determine the stresses and deformations that the structure will experience under each load condition. Use a combination of analytical tools such as finite element analysis (FEA) and structural analysis software to perform these calculations.

Load assessment typically involves a combination of theoretical calculations, computer modeling, and physical testing. The process begins with an evaluation of the various types of loads that a structure is likely to encounter. Each type of load is evaluated separately to determine its expected magnitude, direction, and frequency of occurrence.

i. **Dead Load**

According to the IS 875:1987(Part I), the dead load in a building shall comprise the weights of all walls, partitions, beam, column, floors, and roofs and shall include the weight of all other permanent features in the building. The details of the material chosen for design of structural members and their unit weight, total loads on each structural member are tabulated below:

S.N.	Material Used	Unit Weight	Type Member
1.	Cement Concrete for RCC	25 kN/m³	Beams, Columns, Slabs
2.	Common Burnt Clay Bricks	19.6 kN/m³	Walls
3.	Floor Finishing	1 kN/m²	Floors

Table 2.1 Unit weight for each material used

Estimation of dead load: The weight of those elements of building whose dimensions are fixed from functional considerations is calculated and can be worked out carefully. These are generally non-structural elements such as parapets, rooftops, railings, etc.

1. From pre-design, weight of structural elements such as beam, column, slab etc. are calculated.

2. All loads are systematically put on sketches, say plan-wise, showing their gravity lines with reference to column centerlines.

ii. **Live Load**

Live loads refer to dynamic or moving loads that can vary in magnitude and location over time. Live loads are temporary and also depend on the use of the building. Live loads can be further classified into two types:

Uniformly distributed loads: These are loads that are evenly distributed over a specific area or length of a structure. For example, the weight of people on a floor is generally considered to be a uniformly distributed load.

Concentrated loads: These are loads that are concentrated at a specific point or area of a structure. For example, the weight of a large piece of equipment on a floor might be considered a concentrated load.

Codes such as IS 875: 1987 (Part 2) are used for various occupancies where required. These codes permit certain modifications in the load intensities where large

contributory areas are involved, or when the building consists of many stories. The details of the load (adopted from NBC) as:

- Ramp Slab: 5 KN/m^2
- Staircase: 4 KN/m^2
- Slab: 2.5 KN/m^2

Estimation of live load:

Selection of live load intensity occupancy-wise as an applicable for slabs and beams from the code is done and written on plan. Their reduction of live load intensities for the number of storey in columns and that for calculating earthquakes loads may be considered in the calculations later.

iii. Moment Due to Eccentric Vertical Load

When a vertical load is applied to a beam or column that is not centered on the longitudinal axis, it creates an eccentric load, which causes a moment to be developed in the structure. The moment due to eccentric vertical loads is the product of the eccentricity and the magnitude of the applied load.

Mathematically, the moment due to an eccentric vertical load can be expressed as follows:

$$M = P * e$$

Where M is the moment due to the eccentric load, P is the magnitude of the vertical load, and e is the distance between the applied load and the longitudinal axis of the structure.

The moment due to eccentric vertical loads can have significant effects on the behavior of a structure, as it can cause bending and torsional stresses that can lead to deformation and failure if not properly accounted for in the design. Structural engineers must carefully evaluate the potential for eccentric loading and design the structure accordingly to ensure that it can safely withstand the expected forces and stresses over its intended lifespan.

When transferring the loads from parapets, partition walls, cladding walls, and façade walls etc. to the supporting beams or columns, the eccentricity with these loads should be properly considered in the case of rigid frames of reinforced concrete. Such eccentricities will produce externally applied joint moments similar to these arising from projecting cantilevers and these should be included in frame design This may involve selecting stronger materials, increasing the size or number of structural members, or modifying the overall design of the structure to better distribute the loads.

iv. **Seismic Load**

Seismic load refers to the force or energy generated by an earthquake that can affect structures such as buildings, bridges, and other man-made structures. Seismic loads can cause significant damage and failure to structures if they are not designed to withstand the shaking and ground motions that occur during an earthquake. Nobody knows

the exact timing and magnitude of such loads. Seismic loads are to be determined essentially to produce an earthquake resistant design.

Seismic loads are typically measured in terms of ground acceleration, which is the rate of change of velocity over time. The intensity of seismic loads is determined by various factors, including the magnitude and duration of the earthquake, the distance from the earthquake epicenter, the type of soil or rock beneath the structure, and the building's height and weight.

We use seismic load calculations and building codes to ensure that structures are built to withstand seismic loads. We can incorporate design features such as seismic bracing,

base isolation systems, and dampers to reduce the effects of seismic loads on buildings and other structures.

Since the **probable maximum earthquake occurrence is not frequent, designing buildings for such earthquakes isn't practical as well as economically unsound. Thus, the philosophy of seismic design is to obtain a no-collapse structure rather than no-damage structure**.

Seismic Design Criteria:

While designing seismic structures, it should meet following criteria:

a. Minor earthquake: It should not cause any structural or non-structural components of structure. Deflection of structure should be controlled providing stiffness.

b. Moderate earthquake: No damage to structural elements should occur, minor cracks may appear mainly elastic. Here elements yield should be prevented providing strength.

c. Severe Catastrophic Earthquake: Structure should not collapse and no life should be hurt. Here structures enter inelastic range and absorb energy by providing ductility.

Estimation of Seismic Loads:

IS: 1893-2016 is used to estimate seismic loads which specifies two methods which are:

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.

a. Equivalent Static method

According to this method, first base shear V_B is designed for building as whole, then distributed to each storey corresponding to its mass center. Then consider the floor diaphragm system, and detail structure analysis is distributed to each element.

ESM can be applied when:

From IS 1893 (Part 1): 2016:

✔   Regular structures have an approximate natural frequency of less than 0.4 s.

✔   Regular buildings with a height less than 15m in seismic zone II.

Steps for estimation of seismic loads using Equivalent Static Method:

1.   Seismic zone factor (Z): It is the value of peak ground acceleration considered for the standard design of structures located in each seismic zone. For Kathmandu, where our project building is located, Z = 0.36. [Table 3, IS 1893 (Part 1)]

2.   Importance factor (I): It is the factor used to estimate the design seismic force depending on the functional use of the structure and is characterized by hazardous consequences if the building fails. I = 1.5 for our project building. [Table 8, IS 1893 (part 1)]

3.   Response reduction factor (R): It is the representation of the maximum responses of a spectrum of idealized SDOF of different natural periods but having the same damping under the action of the same earthquake ground motion at their bases. For SMRF R=5 [IS 1893 (Part 1), Table 9]. It accounts for the nonlinear behavior of buildings during strong shaking.

4.   Approximate natural frequency is estimated as:

- Bare MRF (No infill walls): $Ta = 0.075h^{0.75}$[IS 1893 (Part 1), CL. 7.6.2 a]

- With infill walls: $Ta = \frac{0.09h}{\sqrt{d}}$[IS 1893 (Part 1), CL. 7.6.2 c]

Acceleration coefficient: Sa/g is the factor denoting the normalized design acceleration spectrum value to be considered for the design of structure subjected to earthquake. It depends upon the type of soil and natural frequency. [IS 1893 (Part1), CL. 6.4.2 a]

5.   Design horizontal seismic coefficient $A_h = \dfrac{\frac{z}{2} \times \frac{s}{g}}{\frac{R}{I}}$

6.   Seismic weight is calculated using plan, elevation of building. [Table 10 is referred to as coefficient] i.e.W

7.   Base Shear $V_B = A_h$ x W [IS 1893 (Part1), CL. 7.6.1]

8.   Vertical distribution of base shear is done to different floor levels:

$$Q_i = \frac{Wih_i^2}{\sum\limits_{j=1}^{n} Wjh_j^2} \quad \text{[IS 1893 (Part1), CL. 7.6.3]}$$

From NBC 105:2020

"HORIZONTAL BASE SHEAR COEFFICIENT"

☐ Ultimate Limit State

For the ultimate limit state, the horizontal base shear coefficient (design coefficient), Cd (T1), shall be given by:

$$Cd(T1) = \frac{C(T1)}{R\mu \times \Omega\mu}$$

Where,

C (T1) = Elastic Site Spectra as per 4.1.1

Rμ = Ductility Factor as per 5.3

Ωu = Over strength Factor for ULS as per 5.4

☐ Serviceability Limit State

☐ For the serviceability limit state, the horizontal base shear coefficient (design

☐ coefficient), Cd (T1), shall be given by:

$$Cd(T1) = \frac{Cs(T1)}{\Omega s}$$

Where,

Cs(T1) = Elastic Site Spectra determined for Serviceability Limit State as per 4.2

Ωs = Over strength Factor for SLS as per 5.4

☐ HORIZONTAL SEISMIC BASE SHEAR

The horizontal seismic base shear, V, acting at the base of the structure, in the direction being considered, shall be calculated as:

$$V = Cd (T1) W$$

Where,

Cd (T1) = Horizontal base shear coefficient as per 6.1

W = Seismic Weight of the structure as per 5.2

☐ VERTICAL DISTRIBUTION OF SEISMIC FORCES

The lateral seismic force (Fi) induced at each level 'i' shall be calculated as:

$$Fi = \frac{wi(hi^k)}{\sum\limits_{i}^{n} Wi(hi^k)}$$

Where,

Wi = seismic weight of the structure assigned to level 'i';

hi= height (m) from the base to level 'i';

n= total number of floors/levels

V= horizontal seismic base shear calculated as per 6.2

k= an exponent related to the structural period as follows:

• for structure having time period T≤0.5sec, k=1

• for structure having time period T≥2.5sec, k=2

• for structure having period between 0.5 sec and 2.5 sec, k

shall be determined by linear interpolation between 1 and 2."

Response Analysis Method:

Response spectrum is the representation of max value of responses of SDOF system for different natural frequency under same damping and earthquake motion.
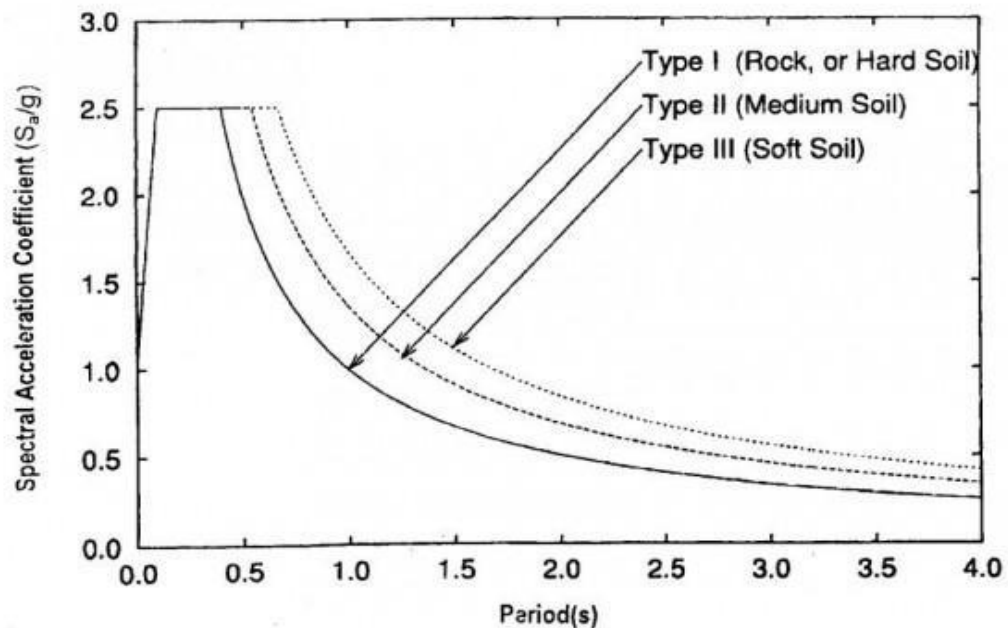
Source: IS 1893:2016

Fig: 2.3 Spectra for Response Spectrum Method

Steps for dynamic analysis:

1.   Seismic weight is calculated for each floor.

2.   Using the characteristics equation, natural frequency and modal shapes are determined.

$|[k]-w^2[m]|=0$

Where, $[k]$= stiffness matrix

   $[m]$= mass matrix

Now from[IS 1893 (part1), Cl 7.7.5.4]

3.   Modal mass is calculated:

$$Mk = \frac{[\sum_{i=1}^{n} Wi\phi i]^2}{g \sum_{i=1}^{n} Wi(\phi ik)^2}$$

4.   Mode participation factor Pk is calculated:

$$Pk = \frac{\sum_{i=1}^{n} Wi\phi i}{\sum_{i=1}^{n} Wi(\phi ik)^2}$$

5.   Calculation of Ak is done using Fig 2.3 ie response spectrum.

6.   Design lateral force for each floor. Qik= Ak ϕik Pk Wi

7.   Storey shear is calculated: Vik= $\sum_{j=i+1}^{n} Qik$

8.   Lateral forces in each floor after all modes considered.

$F_{roof}= V_{roof}$

$Fi=Vi-V_{i+1}$

Time History Method:

Time history method is analysis of buildings based on appropriate ground motion in desired range of natural frequencies and it is based on accepted principles of earthquake dynamics.

7. Modal Response Spectrum Method

It is based on NBC 105: 2020

"7.1 ULTIMATE LIMIT STATE

For the ultimate limit state, the horizontal base shear coefficient for each mode, Cd(Ti), shall as given by:

$Cd(Ti) = C(Ti) R\mu \times \Omega u$

Where,

$C(Ti)$ = Elastic Site Spectra at period Ti as per 4.1.1

Ti = fundamental period of the ith mode of vibration

$R\mu$ = Ductility Factor as per 5.3

Ωu = Over strength Factor for ULS as per 5.4

7.2 CALCULATION OF BASE SHEAR FORCE FOR EACH MODE

$Vi = Cd(Ti) \times Wi$

Where,

Wi = Effective modal gravity load of ith mode of vibration

$$= \frac{[\sum_{j=1}^{n} Wj\phi ji]^2}{\sum_{j=1}^{n} Wj(\phi ji)^2}$$

Wj = portion of gravity load at story level j

ϕji = mode shape coefficient at floor j in mode i

The modal force for ith mode of vibration at each of the story level j is determined as follows:

$$Fji = \frac{wj\phi ji}{\sum_{j=1}^{n} \phi jiwj} \times Vi$$

where,

---

wj = portion of gravity load at story level j

ɸji = mode shape coefficient at floor j in mode i

- A sufficient number of modes shall be included in the analysis to include at least 90% of the total seismic mass in the direction under consideration.
- The combination of modal effects (such as story shear, moment, drift, displacements) shall be carried out using an established method such as Square Root of the Sum of the Squares (SRSS) or the Complete Quadratic Combination (CQC) method or any other generally accepted combination methods.
- Modes shall be considered to be closely spaced if their frequencies are within 15%. For such modes, if the SRSS combination method is used, the modal action effects from any modes shall be first combined by direct summation ignoring any signs.
- When the design base shear (VR) obtained by combining the modal base shear forces is less than the base shear (V) calculated using Equivalent Static Method; the member forces, story shear forces & base reactions obtained from the MRS method shall be multiplied by V/VR.

    Where, V = Base Shear determined from Equivalent Static Method

    VR = Base Shear determined from Modal Combination

2.3.3 Load Combinations:

The load combination adopted for the design considering all load cases in accordance with NBC: 105 : 2020 Cl 3.6

"3.6.1 Load Combinations for Parallel Systems

Where seismic load effect is combined with other load effects, the following

load combination shall be adopted.

1.2DL + 1.5LL

DL + λLL + E

Where, λ = 0.6 for storage facilities

= 0.3 for other usage

3.6.2 Load Combinations for Non- Parallel Systems

 When lateral load resisting elements are not oriented along mutually orthogonal horizontal directions, structure shall be designed for the simultaneous effects due to full design earthquake load in one direction plus 30 percent of design earthquake load along the other horizontal direction. In this case, the following load combination shall be adopted.

1.2DL + 1.5LL

DL + λLL + (Ex + 0.3Ey)

DL + λLL + (0.3Ex + Ey)

Where, λ = 0.6 for storage facilities

= 0.3 for other usage"

2.4. Modifiers:

As time passes members subjected to load tend to crack and their effective stiffness in shear, torsion and flexure reduces. Modification factors are used to model cracked behavior of concrete sections. ETABS provides options for Flexure, Shear, Torsion and Mass/Weight modifiers. The modifier values were taken as given for the cracked section. The usual recommended procedure is to first analyze the model with uncracked section modifiers then check the tensile stress values on the wall. If those values exceed the tensile strength of the concrete, cracked section modifiers are used.

NBC 105 (Table 3-1) provides the effective stiffness of different components at a cracked state.

Component	Flexural Stiffness	Shear Stiffness
Beam	0.35EcIg	0.40EcAw
Columns	0.70EcIg	0.40EcAw
Wall- Cracked	0.50EcIg	0.40EcAw
Wall-Uncracked	0.80EcIg	0.40EcAw

Table: 2.2 Modifiers

3. IDEALIZATION

The process of replacing an actual structure with a simple system conducive to analysis is called "structural idealization" Most often, lines that are located along the centerlines of the components represent the structural components. While transforming a physical structure to its equivalent idealized form, the following characteristics of the idealized structure shall be selected as close as possible to the actual structure: Stiffness, span length, supports, loads, etc.

Structural idealization is meant for the introduction of necessary constraints and restraints in the real structure so that we can address the design of the structure with available design philosophies. In the design of RCC structures, idealization on following aspect is needed;

3. 1. Idealization of load

Idealization of loads acting on the structure is assessed with respect to the application of load and transfer of load in the elements separately. Uniform shell wall is assigned per m2 area on each slab.

3.2. Idealization of structure

This idealization imposes restraints/constraints to those variables which we are unable to address properly otherwise. Imploring the details of these idealizations, we need to start at the elemental level. Thus, we proceed with idealization of supports, slab elements, staircase element, beam and column element and the entire structural system.

3.2.1 Idealization of supports:

It deals with the assessment of fixity of structure at the foundation level. As we have designed mat foundation as substructure, idealizations for RCC framed structure supported over the mat are used. This idealization is used in defining the fixed support of the building frame for modeling and analysis. For buildings with basement walls or underground storey, the idealization depends upon the connection of the basement wall with the superstructure. In our case the basement wall is rigidly connected with a beam on the upper floor, and with the columns on the side.

3.2.2 Idealization of Slab

Idealization of the slab element is done so as to perform as a rigid floor diaphragm. Shell problems generally fall into one of two categories: thin shell problems and thick shell problems. Thick shell problems assume that the effects of transverse shear deformation are important to the solution. Thin shell problems, on the other hand, assume that transverse shear deformation is small enough to be neglected. A shell made of a single isotropic material with a thickness to span ratio > 1/15 is considered "thick" and considered "thin" if the ratio is less than 1/15. These estimates are approximate. For our analysis, idealization is done for slab to behave as a thin shell element subjected to out-of-plane bending. 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.

3.2.3 Idealization of Staircase

Staircase is simply idealized as an inclined slab. The loads from the staircase are transferred to the supports as vertical reactions and moments.

3.2.4 Idealization of Beam and Columns

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, perfect rigid joints do not exist. Another idealization is addressing the section of main beam as rectangular in shape despite being integrally connected with the slabs.

3.2.5 Idealization of Structural System

After idealizing individual elements, we idealize the structural system in its entirety to behave as our theoretical approximation for first order linear analysis and corresponding design. The building is idealized as an unbraced space frame. This 3D space framework is modeled in ETABS for analysis. Loads are modeled into structure in several load cases and load combinations. The building is then subjected to gravity and lateral loads are analyzed for necessary structural responses to design the members.

4. METHODOLOGY

4.1 Structural System

A structural system refers to a combination of structural elements (load carrying elements) and non-structural elements. It is the mechanism in which the load of a building or structure is supported and distributed. The structural system is a critical aspect of a building's design, as it ensures that the building is stable and able to withstand the forces that act upon it.

There are various types of structural systems used in building construction, including:

a. Beam and column: This is a common type of structural system that uses vertical columns and horizontal beams to distribute the load of the building.

b. Braced frame: In this type of structural system, diagonal braces are used to provide stability and support to the building.

c. Truss: A truss is a structural system made up of interconnected triangles. This type of system is often used in roofing and bridge construction.

d. Cable: Cable structures use tensioned cables to support the load of the building. This type of system is often used in suspension bridges and other types of large-span structures.

e. Arch: In an arch system, the load of the building is distributed along an arch-shaped structure. This type of system is often used in bridge construction and other types of large-span structures.

The choice of structural system depends on a number of factors, including the size and shape of the building, the materials used, and the loads that the building will be subjected to. Our project is based on the Beam and Column structural system. They transfer loads acting to ground without significantly disturbing the geometry, integrity and serviceability of the structure.

4.1.1 Data Collection

All the data required are taken from different literatures. Design parameters have been taken from Indian Standard code of Practice.

4.1.2 Load Calculation

Load calculation has been done using the IS 875-1987 (Part 1 & Part 2) as reference. The exact value of unit weight of the materials used in the building for calculation for weight has been taken from code. Thickness or depth of materials or section was fixed as per requirement and according to specifications in code.

- Vertical Load
    a. Live and Dead load on slabs
    b. Self-weight of beam
    c. Load transferred from slab

- Lateral Load

    Here a lateral load earthquake is only considered as Nepal lies in seismic zone V and is prone to earthquake over wind loads. Seismic load of a building depends upon its geographic location, lateral stiffness, soil upon which it is erected and it is reversible. Thus this effect is considered along both axes of building taken one at the time.

4.1.3 Preliminary Design

We performed initial structural calculations to ensure that the selected systems and materials can support the anticipated loads and meet applicable building codes and standards. Beam, Slab, Column design were done so that they act as guideline in analysis and aid in making final design safe and economical.

4.1.4 Modelling and Analyzing

Load testing of real structure to determine the responses is not possible all the time due to economical, technical and environmental constraints. This fact leads to the computer modeling of the complex structure by dividing it into simple elements of known solutions. An appropriate model is a prerequisite for an appropriate design. Nodal coordinates, elemental nodal connectivity, constraints conditions and loads are the data required for the preparation of a structural model.

The structural analysis of the model is required to get the approximate response of structure under the prevailing loading conditions. This response is then used to derive the

element internal forces and stresses, upon which the design of the element is based. Analysis can be done by:

i. **Manual Analysis**

It is done for small structures on a low budget. But most of the structures are statically indeterminate and multi-storeyed and tall manual calculation is nearly impossible, tedious, time consuming and possibility of errors.

ii. **Use of Standard software**

2D analysis or 3D analysis of structure can be done using standard software. For our project ETABS is used.

ETABS (Extended 3D Analysis of Building Systems) is a structural analysis and design software used for building structures. It is developed by Computers and Structures, Inc. (CSI) and is widely used by structural engineers for designing high-rise buildings, hospitals, schools, and other types of structures. ETABS uses a 3D model to perform static, dynamic, and seismic analysis of structures, as well as design various structural elements such as beams, columns, slabs, and walls. The software also includes features such as design optimization, code-based design, and the ability to generate detailed reports and drawings. ETABS is considered one of the most advanced software for building analysis and design in the industry. In this project the beams, columns were modeled as 3D frame elements and slab and shear walls as 3D shell elements.

Finite Element Method

Analysis is done using FEM. The Finite Element Method (FEM) is a numerical technique used to solve differential equations and other mathematical problems in engineering and science. It is used to find approximate solutions to complex problems that are difficult to solve analytically.

In FEM, the domain of the problem is divided into a finite number of smaller regions or elements. The behavior of these elements is then analyzed using numerical methods to find the solution to the problem. The analysis is performed by approximating the behavior of each element and then assembling them to create an overall model of the system being studied.
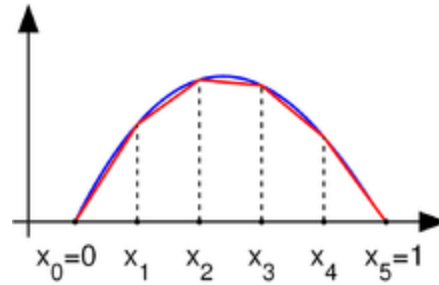
Fig:4.1 Discretization for FEM

Steps involved in FEM:
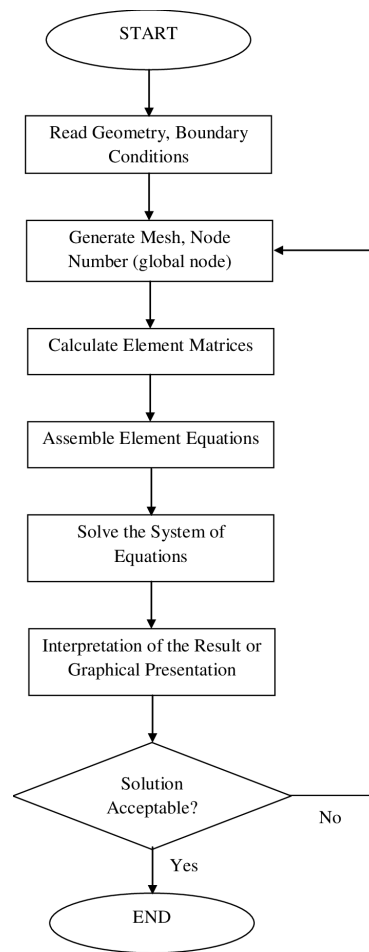
Fig:4.2 Steps Involved in FEM

Analysis Process

The finite element analysis program in ETABS involves three stages of activities:

In the pre-processing stage, nodal coordinates, connectivity, boundary conditions, loading and material information are defined and assigned to respective elements.

The processing stage involves the stiffness generation, stiffness modification and solutions of equations resulting in the evaluation of nodal variables. Other derived quantities such as gradients or stresses may be evaluated at this stage.

In the post processing stage presentation of results is dealt. Typically, deformed configuration, mode shapes, temperature and stress distributions are computed and displayed at this stage.

Interpretation of Analyzed Data

The maximum internal forces and stresses developed due to various load combinations and time history analysis was exerted using excel programs. The sample results are presented in a later part of this report.

4.1.5. Detail Design

With reference to preliminary design and ETABS modeling final dimensions for structural elements are fixed. Then using code all the elements are designed and all check provisions are completed in this step.

4.1.6 Drawing

As per codal provisions and detail design, reinforcement is fixed in slab, beam, column, shear walls and with curtailment, anchorage and splicing detail drawing is done. Ductility provisions are also considered.

4.1.7. Documentation

Along with theory, literature, designs, drawing, discussion on problems faced is recorded and presented to supervisors.

Summary:

The following provides a broad overview of the basic modeling, analysis, design, and detailing processes:

1. Select the Base Units and Design Codes
2. Set up Grid Lines
3. Define Story Levels
4. Define Section Properties
5. Draw Structural Objects
6. Select Objects
7. Assign Properties
8. Define Load Patterns
9. Assign Loads
10. Define Load Cases
11. Edit the Model Geometry
12. View the Model
13. Analyze the Model
14. Display Results for Checking
15. Design the structure
16. Generate Detail Documents
17. Output Results and Reports

5. PRELIMINARY DESIGN

Before proceeding with 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 for analysis, which are liable to be changed in the future after response evaluation. Preliminary design is an essential stage in the construction of any building structure. It involves defining the general layout, dimensions, and materials of the structure before moving on to detailed design and construction. Drawings received from the architect were thoroughly studied, and elements at the most exploited locations were chosen for preliminary design.

For the preliminary design of beams, the deflection criteria given in IS 456 23.2.1 can be used. And as specified in IS 456 Section 24.1, those criteria for beams can be applied to slabs too. Preliminary Design of slab was done following those criteria, and a depth of 179 mm was obtained. However, such thickness is unnecessary for carrying load and furthermore, it only increases the seismic mass of the structure, increasing the seismic demands. Hence, a slab depth of 130 mm was used and the slab span was reduced in half by introducing secondary beams.

To ensure that the final structure is safe, functional, and cost-effective, it is essential to be careful about the following things during the preliminary design stage:

1. Structural system: The appropriate structural system for the building based on the intended use, building height, location, and environmental factors should be chosen. The structural system should be selected based on its suitability to resist the expected loads and meet the desired performance objectives.

2. Site conditions: A thorough site analysis should be conducted to understand the soil conditions, groundwater table, and other geological and environmental factors that may affect the design of the structure. The soil conditions can significantly impact the foundation design, which in turn affects the entire structure.

3. Building geometry: The building geometry should be optimized to maximize the use of available space while minimizing the cost of construction. The building geometry should be designed to minimize irregularities that can affect the structural performance, such as mass irregularities, torsional irregularities, or soft storeys.

4. Material selection: The appropriate materials for each component of the structure should be selected based on the intended use, structural requirements, and environmental factors. The material selection should consider the durability, strength, and cost-effectiveness of the material.

5. Building codes: The preliminary design should comply with all relevant building codes, regulations, and standards. Building codes specify the minimum requirements for structural safety, seismic resistance, fire protection, and other key aspects of building design and construction.

5.1 Design of Slabs:

Longer length (L_y) = 8636 mm

Shorter span(L_x)= 7698 mm

L_y/L_x= 1.122 < 2

Therefore, a two-way slab is designed.

Calculation of depth (d) w.r.t. control of deflection ,

Span 10 m [Cl. 23.2, IS 456: 2000]

So, α= 26, β=1.

Let % of steel= 0.2

F_y=415 N/mm^2

F_s=0.58 x 415 x 0.2 / 0.2=240.7N/mm^2

From the graph for modification factor vs. % tension reinforcement graph,

γ=1.65

λ=1; (no compression reinforcement provided)

δ=1; (other than flanged beam)

$\gamma * \delta * \lambda * \alpha * \beta$= 42.9

L_x/d=42.9, i.e. d=179 mm >150 mm

Hence, a secondary beam is to be provided.

For secondary beam provided at mid-span:

Longer length (L_y)= 7698 mm

Shorter span(L_x)= 4318 mm

L_y/L_x= 1.783 < 2

Therefore, a two-way slab is designed.

Calculation of depth (d) w.r.t. control of deflection

Span 10 m [Cl. 23.2, IS 456: 2000]

So, α = 26, β = 1.

Let % of steel= 0.2

F_y=415 N/mm^2

F_s=0.58*415*0.2/0.2=240.7N/mm^2

From graph for modification factor vs % tension reinforcement graph

γ=1.65

λ=1; (no compression reinforcement provided)

δ=1; (other than flanged beam)

$\gamma * \delta * \lambda * \alpha * \beta$= 42.9

l_x/d=42.9, i.e. d=105 mm

Clear cover =15 mm

Let diameter of rebars =10 mm

So, overall depth(D)= 105+15+10/2=125 mm

Let us provide 130 mm overall depth.

5.2 Design of beams:

Dead load from slab =0.13 * 25=3.25 KN/m^2

Dead load from beam= 25 * 0.4 * (0.6-0.13) =4.7KN/m

[Taking size of beam 400 * 600mm]

Imposed load =4 KN/m^2

Floor finish=1 KN/m^2

Wall load= 20 * 0.11 * (3.6-0.6) = 6.6KN/m

 [Wall Thickness= 0.11m, height of floor= 3.6m, depth of beam= 0.6m]

Load distribution from slab to beam:

Dead load from slab + imposed load + floor finish W_1= 8.25KN/m^2

Load on secondary beam from trapezoidal portion:

W= 2 * [{W_1 * L_x/ 2} * {1-1/(3*β^2)}]=28.964 KN/m

Factored load on secondary beam= 1.5 * load on secondary beam from trapezoidal portion + dead load from beam

=1.5 * (28.964+4.7)

=50.496 KN/m

Reaction on each end= 50.496 * L_x/2=194.359KN

It acts as a point load on the primary beam at mid-span.

For Primary Beam:

W_1=8.25 KN/m^2

Load from Triangular portion,

W=W_1 * L_x * 2/3=8.25 * 4.318 * 2/3=23.749KN/m

Total Factored UDL load on primary beam=1.5* (23.75+4.7+6.6)=52.574KN/m

Point load on center of primary beam =2 * 194.36=388.718KN

Maximum moment(M_u)= wl^2/12 +PL/8=746.37KNm

Depth from moment criteria :

M_u=0.138 * f_{ck} * b * d_{eff}^2

f_{ck}=30MPa

d_{eff}=0.671m > 600mm so again,

So, new depth d=700mm

For d/b=1.5, b=450mm

Repeating calculations:

DL from beam=6.413KN/m

Wall load =6.38KN/m

W_1= 3.25+4+1=8.25KN/m^2

W=28.964kN/m

Total load=1.5x(28.964+6.413)=53.066KN/m

Reaction on each end=53.066 * 7.698/2=204.25KN

For Primary beam:

W_1=8.25KN/m^2

W=23.749KN/m

Total factored load=1.5*(23.749+6.413+6.38)=54.813KN/m

Point load on center of primary beam=2*204.25=408.502KN

Maximum moment (M_u)=(54.813*8.636^2)/12+408.502*8.636/8=781.643KNm

Depth from moment criteria

M_u=0.138 * f_{ck} * b * d^2, so d= 648mm

Adopting d_{eff}=650mm

L/d_{eff}=α*β*γ*δ*λ, for economic design keeping ratio between 12 to 15 [Cl.23.2, IS456,2000]

i.e depth=8636/15=575.733mm

Then actual depth= 650+ clear cover (50) =700mm

For secondary beam:

Dead load from slab =0.13*25=3.25 KN/m^2

Dead load from beam= 25*0.4* (0.3-0.13) =2.025KN/m

[Taking size of beam 3000x400mm]

Imposed load =4KN/m^2

Floor finish=1KN/m^2

Load distribution from slab to beam:

Dead load from slab + imposed load + floor finish W_1= 8.25KN/m^2

Load on secondary beam from trapezoidal portion:

W= 2* [{W_1*L_x/2}*{1-1/(3*β^2)}]=28.964KN/m

Factored load on secondary beam= 1.5 * load on secondary beam from trapezoidal portion plus dead load from beam

=1.5 * (28.964+2.025)

=46.4835 KN/m

Maximum moment $(M_u)=(46.485.636^2)/12=229.54$ KNm

Depth from moment criteria

b = 300(assume)

$M_u=0.138*f_{ck}*b*d^2$, so d= 429.9mm

Adopting d_{eff}=450mm

Assume clear cover as 50 mm

Then Actual depth =500 mm

The preliminary design for secondary beam is 300*500 mm

5.3 Design of column:

Assume preliminary column size of 700 mm*700 mm

Influential Area= 45.485 m^2

Floor	self wt slab	self wt primary beam	self wt secondary beam	self wt partition wall	self wt column	floor finish	Staircase load	Total Dead load	Imposed load	Sum
Basement 2	147.83	86.46	44.10	10.00	36.75	45.49	0.00	370.63	113.71	484.34
Basement 1	147.83	86.46	44.10	119.45	36.75	45.49	0.00	480.07	113.71	593.78
Semi Basement 2	147.83	86.46	44.10	65.66	36.75	45.49	0.00	426.28	113.71	540.00
Ground	147.83	86.46	44.10	0.00	44.10	45.49	15.00	382.98	90.97	473.95
Floor 1st	93.24	72.62	44.10	0.00	44.10	28.69	47.90	330.65	57.38	388.03
Floor 2nd	147.83	86.46	44.10	56.86	44.10	45.49	65.80	490.63	90.97	581.60
Floor 3rd	117.37	86.46	44.10	56.86	44.10	36.11	65.80	450.80	72.23	523.03
Floor 4th	117.37	86.46	44.10	56.86	44.10	36.11	65.80	450.80	72.23	523.03
Floor 5th	147.83	86.46	44.10	56.86	44.10	45.49	65.80	490.63	90.97	581.60
Floor 6th	117.37	86.46	44.10	56.86	44.10	36.11	65.80	450.80	72.23	523.03
Floor 7th	117.37	86.46	44.10	56.86	44.10	36.11	65.80	450.80	72.23	523.03
Floor 8th	147.83	86.46	44.10	56.86	44.10	45.49	65.80	490.63	90.97	581.60
Floor 9th	117.37	86.46	44.10	56.86	44.10	36.11	65.80	450.80	72.23	523.03
Floor 11th	147.83	86.46	44.10	56.86	44.10	45.49	32.90	457.73	90.97	548.70
Floor 12th	147.83	86.46	44.10	56.86	44.10	45.49	0.00	424.83	90.97	515.81
Floor 13th	147.83	86.46	44.10	56.86	44.10	45.49	0.00	424.83	90.97	515.81

Fig: Different types of load (all in KN) in each floor level to be carried by column

Total load=8420.392 KN

Factored load =12630.59 KN

For axially loaded short column $=0.4*f_{ck}*A_c+0.67*f_y*A_s$

$P_u=.4*f_{ck}*.97*A_g+.67*f_y*.03*A_g$(assume 3% rebar)

A_g obtained $=.5823$ m^2

For square column,L=0.7631 m

Assumed section as 800 mm * 800 mm

6. MODAL ANALYSIS

6.1 STOREY DRIFT CALCULATION FROM ETABS ANALYSIS

Once all the loads are computed and all possible load combinations are developed, we can model these loads in the building. Analysis is done, and the value of inter-storey drift for serviceability conditions is computed from absolute displacements for earthquake loads in both horizontal directions.

Story	Elevation(m)	Location	X-Direction	Y-Direction	<0.025
18 Floor 12th	52.2	Top	0.002173	0.000068	ok
17 Floor 11th	48.6	Top	0.002191	0.000077	ok
16 Floor 10th	45	Top	0.002319	0.000147	ok
15 Floor 9th	41.4	Top	0.002472	0.000187	ok
14 Floor 8th	37.8	Top	0.002545	0.000223	ok
13 Floor 7th	34.2	Top	0.002589	0.000255	ok
12 Floor 6th	30.6	Top	0.002579	0.000277	ok
11 Floor 5th	27	Top	0.002498	0.000286	ok
10 Floor 4th	23.4	Top	0.002328	0.000276	ok
9 Floor 3rd	19.8	Top	0.002151	0.000242	ok
8 Floor 2nd	16.2	Top	0.002114	0.000178	ok
7 First Floor	12.6	Top	0.001481	0.000495	ok
6 Ground floor	9	Top	0.001088	0.000465	ok
5 Parking inlet floor	7.8	Top	0.000682	0.000232	ok
4 Semi basement	6	Top	0.000465	0.000146	ok
3.1 Ramp floor	5.142	Top	0.000348	0.000115	ok
3 Parking floor	3.98	Top	0.000279	0.000124	ok
2 basement-1	3	Top	0.000249	0.00006	ok
1.1 Rramp floor	2.142	Top	0.000141	0.000042	ok
Basement-2	0	Top	0	0	ok

Table 6.1: Storey Drift for ULS Condition Along X direction
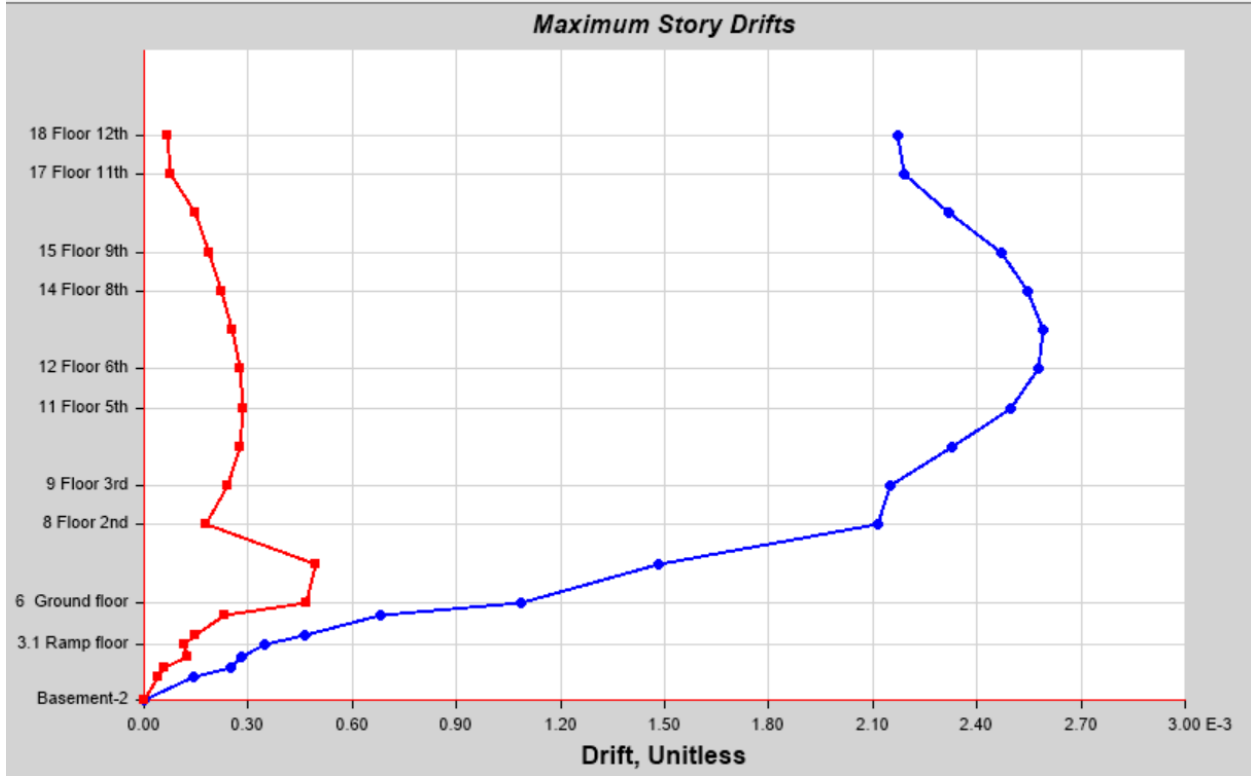
Fig: 6.1 Max Storey Drift for ULS condition along X- direction

Story	Elevation(m)	Location	X-Direction	Y-Direction	<0.025
18 Floor 12th	52.2	Top	0.000085	0.001857	ok
17 Floor 11th	48.6	Top	0.000133	0.001914	ok
16 Floor 10th	45	Top	0.000202	0.001971	ok
15 Floor 9th	41.4	Top	0.000232	0.002011	ok
14 Floor 8th	37.8	Top	0.000228	0.002031	ok
13 Floor 7th	34.2	Top	0.000227	0.002022	ok
12 Floor 6th	30.6	Top	0.000218	0.001971	ok
11 Floor 5th	27	Top	0.000202	0.001877	ok
10 Floor 4th	23.4	Top	0.000186	0.001732	ok
9 Floor 3rd	19.8	Top	0.00013	0.001524	ok
8 Floor 2nd	16.2	Top	0.000145	0.001266	ok
7 First Floor	12.6	Top	0.000529	0.000965	ok

6 Ground floor	9	Top	0.000202	0.00084	ok
5 Parking inlet floor	7.8	Top	0.000124	0.000379	ok
4 Semi basement	6	Top	0.000111	0.000234	ok
3.1 Ramp floor	5.142	Top	0.000136	0.000159	ok
3 Parking floor	3.98	Top	0.00011	0.000117	ok
2 basement-1	3	Top	0.000085	0.000139	ok
1.1 Rramp floor	2.142	Top	0.000043	0.000106	ok
Basement-2	0	Top	0	0	ok

Table 6.2: Storey Drift for ULS condition along Y direction
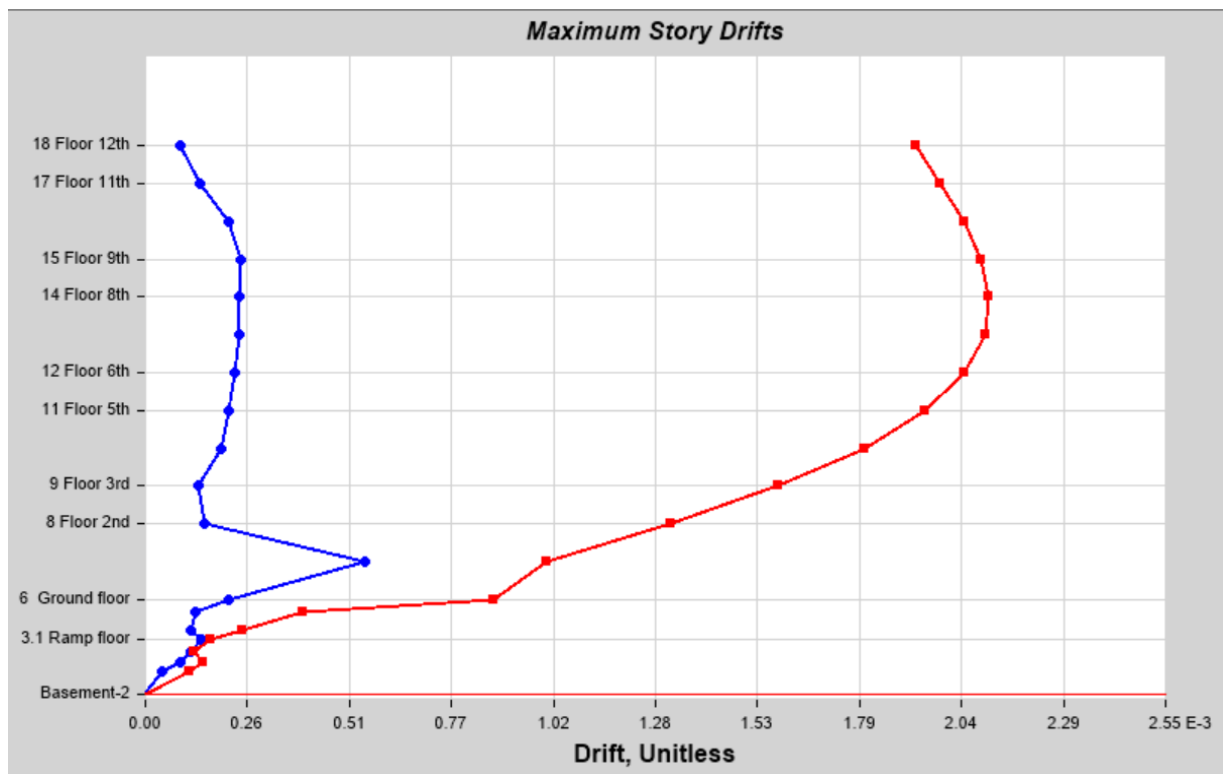

Fig: 6.2 Maximum Storey Drift for ULS condition along Y axis

Table 6. 3: Storey Drift for SLS condition along X direction

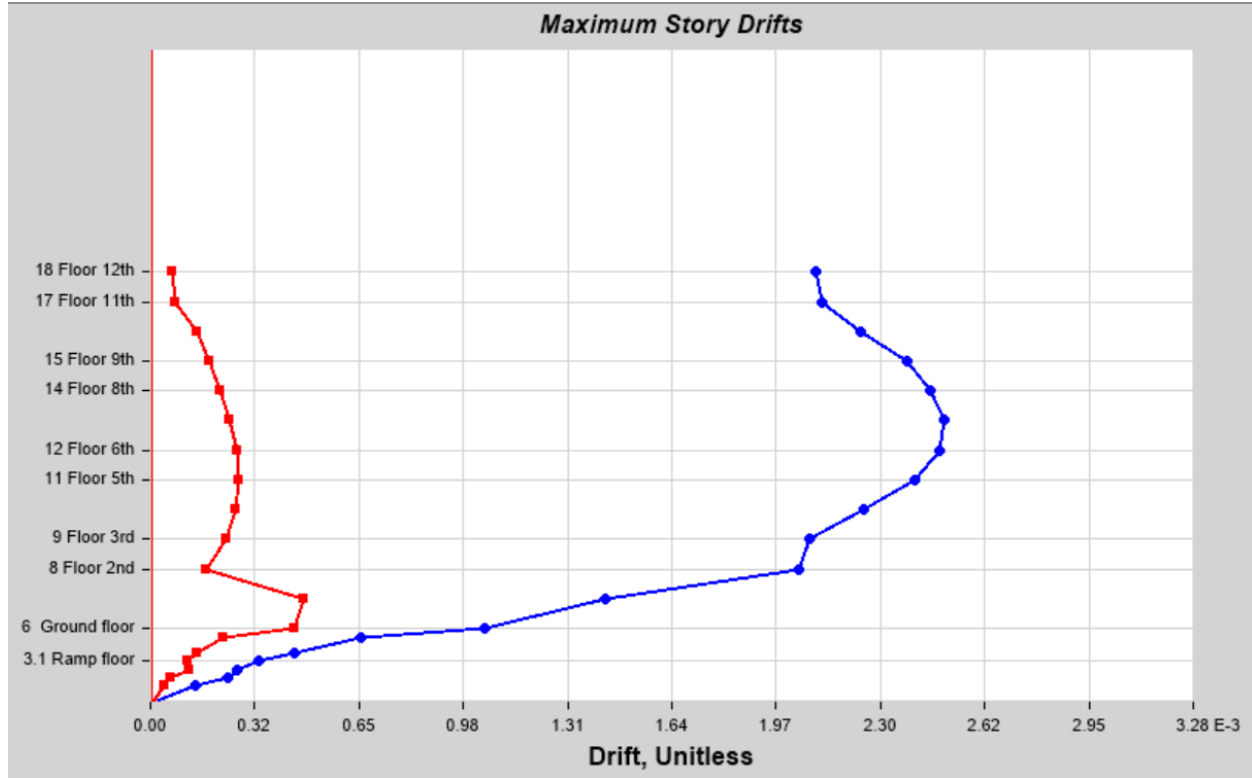| Story	Elevation(m)	Location	X-Direction	Y-Direction	<0.006
18 Floor 12th	52.2	Top	0.002094	0.000065	ok
17 Floor 11th	48.6	Top	0.002111	0.000074	ok
16 Floor 10th	45	Top	0.002234	0.000142	ok
15 Floor 9th	41.4	Top	0.002381	0.00018	ok
14 Floor 8th	37.8	Top	0.002452	0.000215	ok
13 Floor 7th	34.2	Top	0.002495	0.000246	ok
12 Floor 6th	30.6	Top	0.002484	0.000267	ok
11 Floor 5th	27	Top	0.002407	0.000276	ok
10 Floor 4th	23.4	Top	0.002242	0.000266	ok
9 Floor 3rd	19.8	Top	0.002072	0.000234	ok
8 Floor 2nd	16.2	Top	0.002037	0.000172	ok
7 First Floor	12.6	Top	0.001427	0.000476	ok
6 Ground floor	9	Top	0.001048	0.000448	ok
5 Parking inlet floor	7.8	Top	0.000657	0.000223	ok
4 Semi basement	6	Top	0.000448	0.00014	ok
3.1 Ramp floor	5.142	Top	0.000336	0.000111	ok
3 Parking floor	3.98	Top	0.000269	0.000119	ok
2 basement-1	3	Top	0.00024	0.000057	ok
1.1 Rramp floor	2.142	Top	0.000135	0.00004	ok
Basement-2	0	Top	0	0	ok

Fig:6. 3: Maximum Storey Drift for SLS condition along X direction

Story	Elevation(m)	Location	X-Direction	Y-Direction	<0.006
18 Floor 12th	52.2	Top	0.000085	0.001857	OK
17 Floor 11th	48.6	Top	0.000133	0.001914	OK
16 Floor 10th	45	Top	0.000202	0.001971	OK
15 Floor 9th	41.4	Top	0.000232	0.002011	OK
14 Floor 8th	37.8	Top	0.000228	0.002031	OK
13 Floor 7th	34.2	Top	0.000227	0.002022	OK
12 Floor 6th	30.6	Top	0.000218	0.001971	OK
11 Floor 5th	27	Top	0.000202	0.001877	OK
10 Floor 4th	23.4	Top	0.000186	0.001732	OK
9 Floor 3rd	19.8	Top	0.00013	0.001524	OK
8 Floor 2nd	16.2	Top	0.000145	0.001266	OK

7 First Floor	12.6	Top	0.000529	0.000965	OK
6 Ground floor	9	Top	0.000202	0.00084	OK
5 Parking inlet floor	7.8	Top	0.000124	0.000379	OK
4 Semi basement	6	Top	0.000111	0.000234	OK
3.1 Ramp floor	5.142	Top	0.000136	0.000159	OK
3 Parking floor	3.98	Top	0.00011	0.000117	OK
2 basement-1	3	Top	0.000085	0.000139	OK
1.1 Rramp floor	2.142	Top	0.000043	0.000106	OK
Basement-2	0	Top	0	0	OK

Table 6. 4: Storey Drift for SLS condition along Y direction
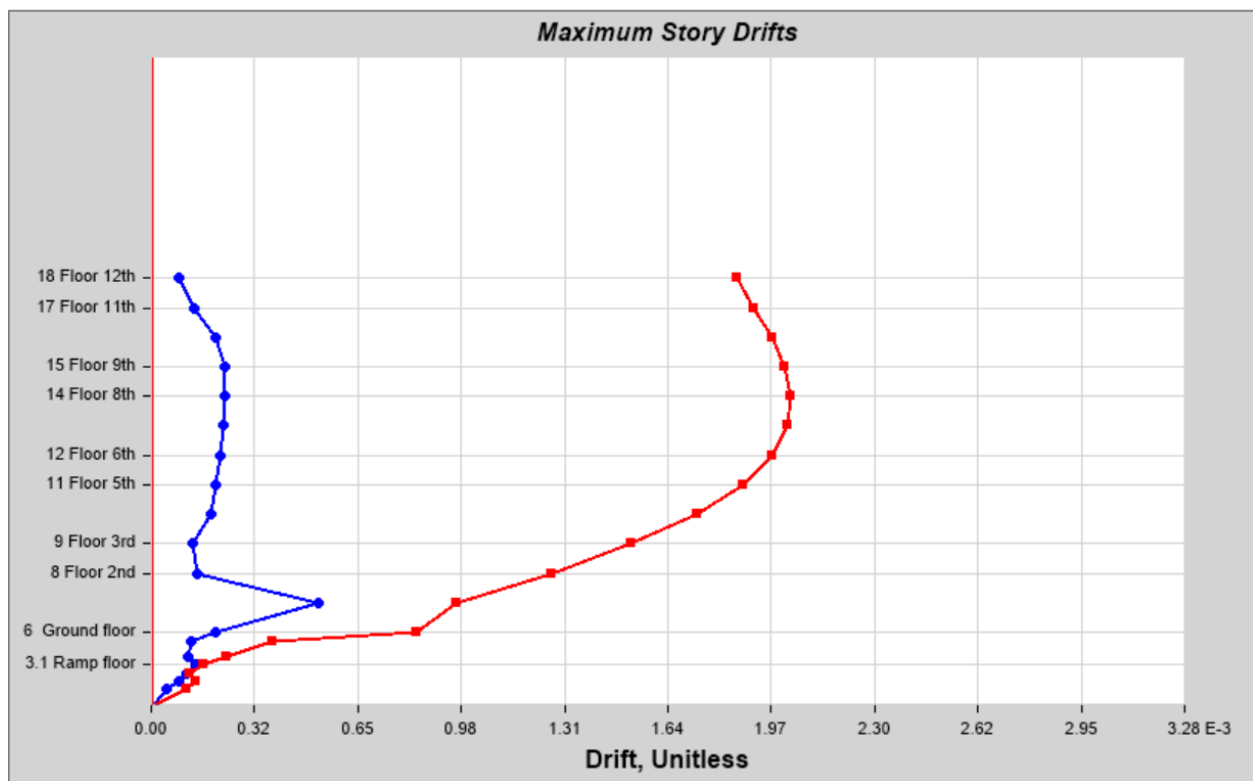

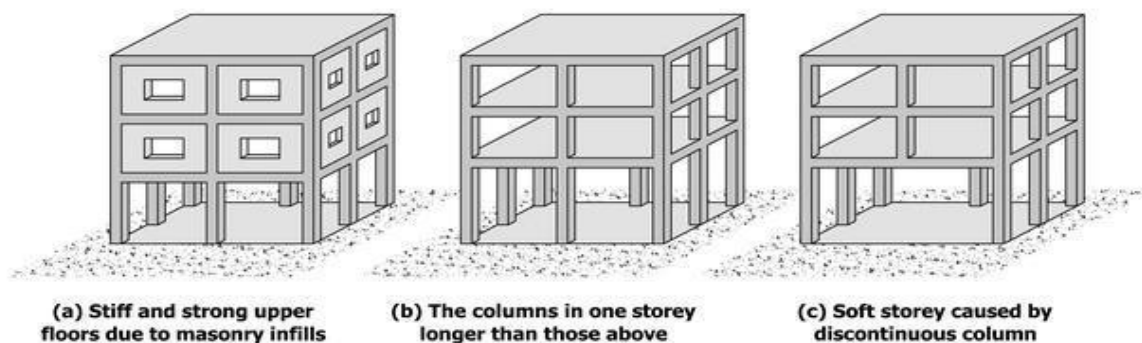Fig 6.4 : Maximum Storey Drift for SLS condition along Y direction

6.2 STIFFNESS IRREGULARITY COMPUTATION FROM ETABS ANALYSIS

With reference to Pg. 17, IS 1893(Part 1), 2016:

6.2.1 Soft Storey: A soft storey in a building structure refers to a floor level that is significantly weaker or less rigid than the floors above it. Typically, a soft storey has a reduced strength due to various reasons, such as less reinforcement, less concrete volume, or fewer walls compared to the floors above it. Mainly, due to hanging columns, higher storey height in some regions or discontinuity in load transfer path causes soft storey.

Soft storeys can be dangerous during seismic activity because they can collapse more easily than the rest of the building, causing significant damage or even complete collapse. This is because the lower floors are subjected to higher lateral forces during earthquakes than upper floors, and the soft storey is unable to withstand these forces.

To avoid the risk of soft storey collapse during earthquakes, it is important to ensure that buildings are designed and constructed with sufficient strength and stiffness. This may involve using special construction techniques, reinforcing the lower floors, or limiting the height of the building.

(a) Stiff and strong upper floors due to masonry infills

(b) The columns in one storey longer than those above

(c) Soft storey caused by discontinuous column

Source: IIT, Kanpur

Fig:6.5 Soft Storey

Story	X-Direction (kN/m)	Stiffness ratio (Ki+1/Ki)	<1	Y-Direction (kN/m)	Stiffness ratio (Ki+1/Ki)	<1
18 Floor 12th	84488.406			136744.503		
17 Floor 11th	373622.253	0.226	Ok	347135.132	0.394	OK
16 Floor 10th	762004.352	0.490	Ok	594267.894	0.584	OK
15 Floor 9th	1087377.791	0.701	Ok	791918.738	0.750	OK
14 Floor 8th	1367946.690	0.795	Ok	954697.370	0.829	OK
13 Floor 7th	1602293.885	0.854	Ok	1088510.489	0.877	OK
12 Floor 6th	1825842.385	0.878	Ok	1220911.951	0.892	OK
11 Floor 5th	2069319.796	0.882	Ok	1373658.248	0.889	OK
10 Floor 4th	2378129.471	0.870	Ok	1571533.409	0.874	OK
9 Floor 3rd	2767556.593	0.859	Ok	1872574.038	0.839	OK
8 Floor 2nd	3161043.548	0.876	Ok	2488050.242	0.753	OK
7 First Floor	6158504.585	0.513	Ok	3561585.892	0.699	OK
6 Ground floor	15616148.463	0.394	Ok	7416062.857	0.480	OK
5 Parking inlet floor	33189776.388	0.471	Ok	20293271.135	0.365	OK
4 Semi basement	101002413.00	0.329	Ok	53293152.907	0.381	OK
3.1 Ramp floor	97989686.419	1.031	Not Ok	56237726.052	0.948	OK
3 Parking floor	139749393.00	0.701	Ok	59151781.095	0.951	OK
2 basement-1	184956356.00	0.756	Ok	114077205.00	0.519	OK
1.1 Rramp floor	119634969.00	1.546	Not Ok	65746174.376	1.735	Not OK
Basement-2	0			0		

Table 6. 5:Storey irregularities along X and Y directions

6.2.2 Weak Storey: A weak storey in a building structure refers to a floor level that is significantly weaker than the floors above it and below it. This weakness may be due to a variety

of factors, such as lower material strength, reduced reinforcement, or the presence of openings that compromise the structural integrity of the floor.

In a multi-storey building, a weak storey is particularly concerning during seismic activity. During an earthquake, the forces acting on the building can cause significant lateral movement, which can lead to the failure of weaker floors, including weak storeys.

To avoid the risk of weak storey collapse during seismic activity, it is important to ensure that all floors of a building have similar levels of strength and stiffness. This can be achieved by designing the building with appropriate structural elements and reinforcement, and by ensuring that all floors are constructed to the same standards. Building codes often specify minimum strength requirements for all floors of a building to help prevent weak storey collapse during earthquakes.

6.2.3 MASS IRREGULARITY COMPUTATION FROM ETABS ANALYSIS

Mass irregularity is a condition in a building structure where there is a significant difference in mass between adjacent storeys or lateral load-resisting elements. When there exists a seismic weight of any storey 150% greater than adjacent storeys except roof it is said to be mass irregularity. This can be caused by a variety of factors, including changes in building geometry, structural materials, or floor-to-floor height.

Mass irregularities in a building structure can have serious consequences during seismic activity. When a building is subjected to lateral forces during an earthquake, the difference in mass between storeys can result in a torsional response, leading to building deformation or even collapse.

To avoid the risk of mass irregularity, it is important to ensure that the mass distribution in a building is as uniform as possible. This can be achieved by designing the building to have a consistent floor-to-floor height, using similar structural materials throughout the building, and avoiding abrupt changes in building geometry. Additionally, it is important to ensure that the building is designed to withstand the lateral forces that are likely to occur during seismic activity, taking into account the mass distribution and other structural factors. Building codes often specify requirements for mass distribution to help prevent mass irregularities in buildings.

For our building, the plan area for top storey is much lesser than the immediate down storey which has resulted in mass irregularity for top storey as indicated in the table below.
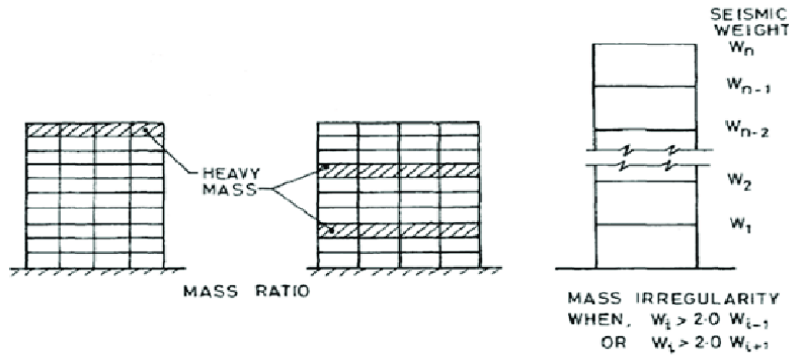

Fig 6.6 Mass irregularity

Story	Diaphragm	Mass (kg)	ABS(Mi+1-Mi)/Mi	<0.5
18 Floor 12th	D1	128162.19		
17 Floor 11th	D1	566032.18	3.4165	not ok
16 Floor 10th	D1	857141.88	0.5143	OK
15 Floor 9th	D1	821497.9	0.0416	OK
14 Floor 8th	D1	881079.89	0.0725	OK
13 Floor 7th	D1	821497.9	0.0676	OK
12 Floor 6th	D1	821497.9	0.0000	OK
11 Floor 5th	D1	881078.79	0.0725	OK
10 Floor 4th	D1	821497.9	0.0676	OK
9 Floor 3rd	D1	754005.52	0.0822	OK
8 Floor 2nd	D1	1055248.82	0.3995	OK
7 First Floor	D1	1094714.01	0.0374	OK
6 Ground floor	D1	775913.56	0.2912	OK
4 Semi basement	D1	984530.08	0.2689	OK
2 basement-1	D1	912813.83	0.0728	OK

Table 6.6 :Mass irregularity

6.2.4 TORSIONAL IRREGULARITY COMPUTATION FROM ETABS ANALYSIS

Torsional irregularity in a building structure refers to a condition where the building's plan or layout creates an uneven distribution of lateral forces during an earthquake. This can occur when the building's mass center is not aligned with its stiffness center or when the building has an asymmetric plan or elevation.

Torsional irregularity can lead to severe structural damage or even collapse during seismic activity. This is because uneven lateral forces can cause torsion or twisting motion, which can lead to building deformation and failure of structural elements.

To prevent torsional irregularity, building designs should be symmetric with regard to mass distribution, stiffness, and elevation. We can also use methods such as structural analysis, computer modeling, and seismic design codes to identify and address torsional irregularity in building designs. This needs to be considered when the floor diaphragms are rigid in their own plane in relation to the vertical structural elements that resist the lateral forces. Torsional irregularity is considered to exist when 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.
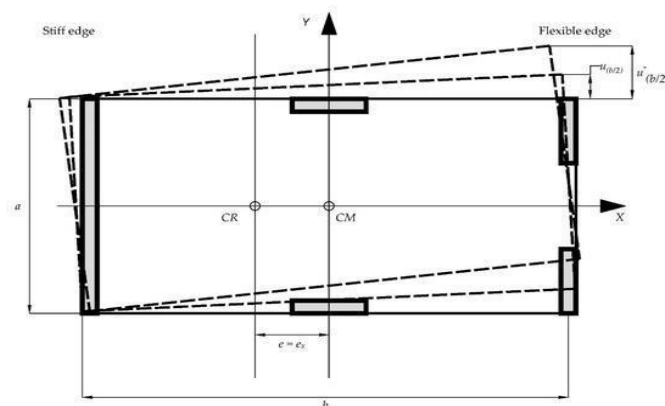

Fig:6.7 Torsional Irregularity

Story	Output Case	Max Drift	Avg Drift	Ratio	Ratio <1.5	Output Case	Max Drift	Avg Drift	Ratio	Ratio <1.5
18 Floor 12th	EQX ULS	0.0021	0.0020	1.048	OK	EQY ULS	0.0018	0.0017	1.058	OK
17 Floor 11th	EQX ULS	0.0023	0.0021	1.062	OK	EQY ULS	0.0020	0.0018	1.101	OK
16 Floor 10th	EQX ULS	0.0025	0.0023	1.093	OK	EQY ULS	0.0020	0.0018	1.101	OK
15 Floor 9th	EQX ULS	0.0024	0.0023	1.04	OK	EQY ULS	0.0021	0.0019	1.107	OK
14 Floor 8th	EQX ULS	0.0027	0.0024	1.103	OK	EQY ULS	0.0021	0.0019	1.113	OK
13 Floor 7th	EQX ULS	0.0027	0.0024	1.107	OK	EQY ULS	0.0022	0.0019	1.119	OK
12 Floor 6th	EQX ULS	0.0027	0.0024	1.11	OK	EQY ULS	0.0021	0.0019	1.126	OK
11 Floor 5th	EQX ULS	0.0026	0.0023	1.112	OK	EQY ULS	0.0021	0.0018	1.132	OK
10 Floor 4th	EQX ULS	0.0021	0.0021	1.001	OK	EQY ULS	0.0019	0.0017	1.136	OK
9 Floor 3rd	EQX ULS	0.0021	0.0019	1.109	OK	EQY ULS	0.0017	0.0015	1.136	OK
8 Floor 2nd	EQX ULS	0.0019	0.0017	1.13	OK	EQY ULS	0.0014	0.0013	1.148	OK
7 First Floor	EQX ULS	0.0014	0.0012	1.172	OK	EQY ULS	0.0009	0.0008	1.142	OK
6 Ground floor	EQX ULS	0.0010	0.0008	1.228	OK	EQY ULS	0.0010	0.0007	1.439	OK
4 Semi basement	EQX ULS	0.0004	0.0003	1.466	OK	EQY ULS	0.0002	0.0001	1.764	NOT OK
2 basement-1	EQX ULS	0.0002	0.0001	1.662	NOT OK	EQY ULS	0.0002	0.0001	1.658	NOT OK

Table 6.7: Torsion irregularity check in both direction

Since basement levels are below natural ground, torsional irregularity in these stories are not concerned with the structural response of the building.

6.3 Dynamic Analysis

We have taken into consideration the provision for dynamic analysis made in IS 1893 (Part 1):2016. It states that dynamic analysis is needed to be done for regular buildings above 15m height in the seismic zone V. So, we have also defined earthquake load for response spectrum function. The scale factor as stated in code is provided to balance the base reaction for response spectrum earthquake load to that of linear static earthquake.

Story	Elevation(m)	Location	X-Direction	Y-Direction	<0.025
18 Floor 12th	52.2	Top	0.00215	0.000839	ok
17 Floor 11th	48.6	Top	0.002275	0.000876	ok
16 Floor 10th	45	Top	0.002636	0.000948	ok
15 Floor 9th	41.4	Top	0.002816	0.001	ok
14 Floor 8th	37.8	Top	0.002905	0.00104	ok
13 Floor 7th	34.2	Top	0.002966	0.001066	ok
12 Floor 6th	30.6	Top	0.002967	0.001069	ok
11 Floor 5th	27	Top	0.002889	0.001042	ok
10 Floor 4th	23.4	Top	0.002706	0.000977	ok
9 Floor 3rd	19.8	Top	0.002519	0.000863	ok
8 Floor 2nd	16.2	Top	0.002516	0.000693	ok
7 First Floor	12.6	Top	0.001433	0.000516	ok
6 Ground floor	9	Top	0.00108	0.000544	ok
5 Parking inlet floor	7.8	Top	0.000677	0.000229	ok
4 Semi basement	6	Top	0.000497	0.000173	ok
3.1 Ramp floor	5.142	Top	0.000368	0.000123	ok
3 Parking floor	3.98	Top	0.00029	0.000144	ok
2 basement-1	3	Top	0.000247	0.00008	ok
1.1 Rramp floor	2.142	Top	0.000144	0.000055	ok
Basement-2	0	Top	0	0	ok

Table 6.8: Storey Drift for Response Spectrum along X direction
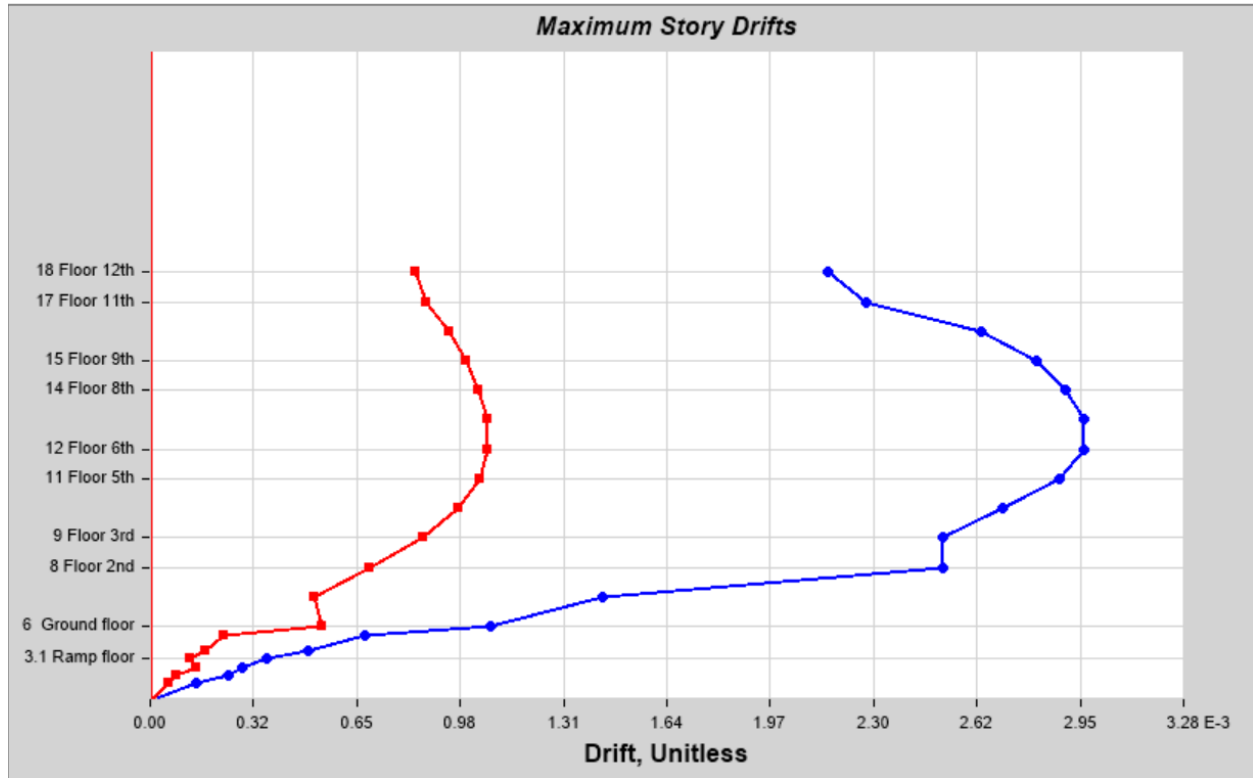

Fig: 6.8 Maximum Storey Drift for Response Spectrum along X direction

Story	Elevation(m)	Location	X-Direction	Y-Direction	<0.025
18 Floor 12th	52.2	Top	0.000685	0.002339	ok
17 Floor 11th	48.6	Top	0.000699	0.002413	ok
16 Floor 10th	45	Top	0.000852	0.00249	ok
15 Floor 9th	41.4	Top	0.000908	0.002542	ok
14 Floor 8th	37.8	Top	0.000927	0.002572	ok
13 Floor 7th	34.2	Top	0.000936	0.002567	ok
12 Floor 6th	30.6	Top	0.000926	0.002512	ok
11 Floor 5th	27	Top	0.000892	0.002402	ok
10 Floor 4th	23.4	Top	0.000825	0.002224	ok
9 Floor 3rd	19.8	Top	0.000758	0.001965	ok

8 Floor 2nd	16.2	Top	0.000803	0.001633	ok
7 First Floor	12.6	Top	0.000701	0.001245	ok
6 Ground floor	9	Top	0.000315	0.001051	ok
5 Parking inlet floor	7.8	Top	0.000228	0.000467	ok
4 Semi basement	6	Top	0.000167	0.000286	ok
3.1 Ramp floor	5.142	Top	0.000192	0.000195	ok
3 Parking floor	3.98	Top	0.000157	0.000145	ok
2 basement-1	3	Top	0.000125	0.000165	ok
1.1 Rramp floor	2.142	Top	0.000067	0.000131	ok
Basement-2	0	Top	0	0	ok

Table 6.9: Storey Drift for Response Spectrum along Y direction
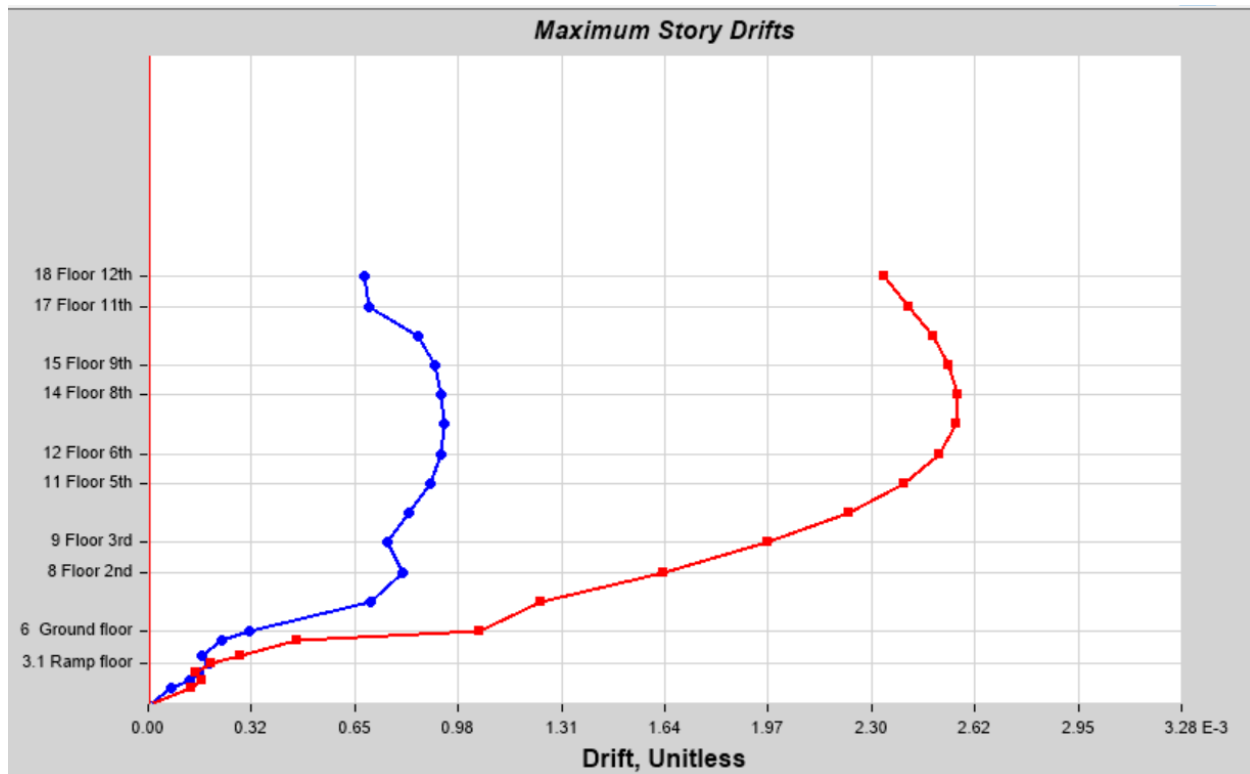
Fig: 6.9 Maximum Storey Drift for Response Spectrum along Y direction

6.4. Modal Data (Time Period and Mass Participation Ratio)

A building is torsionally irregular when the natural period corresponding to the fundamental torsional mode of oscillation is more than those of two translation modes of oscillation along each principal plan design.

As per NBC 105 Cl. 7.3, the minimum number of modes to include 90% of total seismic mass along both directions is 30 for our building.

Mode	Period(sec)	SumUX	SumUY	SumRZ
1	1.001	0.4982	0.0116	0.0053
2	0.876	0.5146	0.4821	0.0168
3	0.753	0.5215	0.5024	0.3638
4	0.245	**0.6672**	0.5104	0.3651
5	0.205	0.6829	0.6465	0.3699
6	0.172	0.6878	0.6541	0.4762
7	0.115	0.7509	0.663	0.4805
8	0.105	0.7977	0.6786	0.4805
9	0.09	0.7981	**0.7183**	0.4818
10	0.086	0.845	0.7231	0.5542
11	0.079	0.8473	0.7414	0.6066
12	0.068	0.856	0.7426	0.6082
13	0.063	0.8561	0.7596	0.6175
14	0.063	0.8562	0.7757	0.6178
15	0.062	0.8589	0.776	0.6187
16	0.06	0.8648	0.7817	0.6254
17	0.059	0.8649	0.7817	0.6258
18	0.057	0.8672	0.7941	0.6261
19	0.055	0.8672	0.7973	0.6264
20	0.052	0.8697	0.8369	0.6378
21	0.051	0.87	0.8654	0.6496
22	0.049	0.8914	0.8769	0.6575

23	0.047	0.9057	0.8772	0.6576
24	0.044	0.9065	0.8846	0.6581
25	0.043	0.9066	0.8936	0.6989
26	0.042	0.9067	0.8942	0.6989
27	0.04	0.9278	0.8959	0.6997
28	0.04	0.9281	0.8962	0.7009
29	0.038	0.9285	0.8982	0.7009
30	0.038	**0.9327**	**0.9085**	0.7009

Table 6.10: Time period and mass participation ratio

6.5 CENTER OF MASS AND CENTER OF RIGIDITY

Centre of mass of each story 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. Drawing of one floor of both blocks is included in drawings (Annex B). 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.

Table 6.11: Center of mass and center of rigidity:

Story	XCM(m)	YCM(m)	XCR(m)	YCR(m)
18 Floor 12th	32.8146	37.9147	29.1413	34.3286
17 Floor 11th	24.0156	36.1555	20.8253	33.9166
16 Floor 10th	21.4788	34.9583	20.8444	33.9387
15 Floor 9th	21.4741	35.0538	20.8467	33.9664
14 Floor 8th	21.5904	34.7666	20.8633	34.0072
13 Floor 7th	21.4741	35.0538	20.8784	34.0577
12 Floor 6th	21.4741	35.0538	20.8797	34.108
11 Floor 5th	21.5904	34.7666	20.8482	34.1449
10 Floor 4th	21.4741	35.0538	20.7617	34.1464
9 Floor 3rd	21.8251	35.1483	20.6015	34.0648
8 Floor 2nd	23.5815	35.1506	20.1518	33.9537
7 First Floor	22.4126	38.6263	19.4117	34.0539
6 Ground floor	19.5864	34.8682	16.9995	33.8838
4 Semi basement	21.342	40.1289	18.4184	42.6227
2 basement-1	21.9339	40.1395	18.413	41.9883

6.6 CALCULATION OF SEISMIC COEFFICIENT

Calculation of seismic coefficient as per IS 1893 (Part 1), 2016:

Seismic coeff. $Ah = (\frac{Z}{2} * \frac{Sa}{g})/(\frac{R}{I})$

Here, Seismic coeff. Factor (Z)=0.36, For Zone V   [Table 3]

    Importance Factor (I)=1.2, For business building   [Table 8]

    Response Reduction Factor=5, For SMRF   [Table 9]

    Height of building (h)= 44.4

    Considering no in fill wall then

    Natural Period of Vibration (Ta)= $0.075h^{0.75}$ = 1.29 s   [Cl.7.6.2 a]

    Average response acceleration coefficient (Sa/g) = 1.67/1.29=1.29   [Cl.6.4.2 a]

Seismic coefficient ,Ah= .056

Base shear is calculated as,Vb= AhxW where W= Seismic weight.

From Etabs analysis, we get Base shear =21073 KN

The Design Base Shear can be distributed along the building storey as:

$$Qi = Vb * (Wi * hi^{2})/\Sigma(Wi * hi^{2})$$

Where, Qi= Design lateral force at i[th] floor.

Calculation of seismic coefficient as per NBC 105: 2020

Location of building: Kathmandu

Seismic zoning factor:(Table 4.5)   Z=   0.35

Importance factor: (Table 4.6)   I=   1.25

Height of building: h=   44.4m

Method of analysis: Equivalent static method

Soil type: (Table 4.4)   D

PERIOD OF VIBRATION

For reinforced Moment Resisting Frame:(Clause 5.1.2 & 5.1.3)

T1=1.25*.075*h^0.75=1.6125

lower period of flat part of spectrum: Ta=   0

Upper part of flat part of spectrum: Tc=   2

peak spectral acceleration normalized by PGA: $\boldsymbol{a}$=2.25

Coeff. that controls the descending branch of spectrum:
K=   0.8

Calculation of Spectral shape factor: Ch(t)

Ta<=T1<=Tc   Ch(t) =   2.25

Ductility Factor for ULS, R_u =   4

Overstrength Factor for ULS , $\boldsymbol{\Omega}_u$ =   1.5

Overstrength Factor for SLS , $\boldsymbol{\Omega}_s$ =   1.25

Elastic site spectra for horizontal loading
C(T)=Ch(T)ZI=   0.984

Elastic site spectra for SLS,   Cs=0.2*C(T)=0.197

HORIZONTAL BASE SHEAR COEFFICIENT

FOR ULS

$C_{du}=C(T)/(R_u * \Omega_u)=$ 0.164

For SLS

$C_{ds}=C_s/\Omega_s$ =   0.158

Exponent related to structural period(linear interpolation)

K   =   1.556

Output Case	FX(KN)	FY(KN)
EQX ULS	-21073.7589	0
EQY ULS	0	-21073.6919
EQY SLS	0	-20302.7031
EQX SLS	-20302.7677	0
RSx	11307.7326	0
RSy	0	10740.7109

Table 6.12: Base shear obtained for ETABS Analysis

Scaling factor for x direction: 1.8637

Scaling factor for y direction: 1.9621

The base shear from the response spectrum is scaled to match the base shear obtained from static analysis.

Detail Design

7. BEAM DESIGN

In building, a beam is a horizontal or inclined structural element that is capable of withstanding loads primarily through internal shear forces and bending moments. Beams are typically used to support loads over relatively long spans. Beams can be made from various materials, including wood, steel, concrete, and composite materials. The shape and size of a beam depend on the loads it needs to support. Common types of beams include rectangular, circular, and I-shaped beams.

Beams are designed to resist bending or flexure, which is the main type of stress that they experience. This bending stress is caused by the loads that are applied to the beam, which create a moment that tries to bend the beam. Beams are also designed to resist shear forces, which are forces that act perpendicular to the length of the beam and tend to slide one part of the beam relative to another.

Design Steps of Beam:

1. Determination of loads: Determine the loads that the beam will be required to support. This includes dead loads (the weight of the building materials), live loads (the weight of people and furniture), and any other loads that may be applicable.

2. Selection of materials: Choose the appropriate material for the beam based on factors such as cost, strength, durability, and ease of construction. Common materials used for beams include wood, steel, and reinforced concrete.

3. Calculation of beam size and shape: Calculate the size and shape of the beam based on the loads it will be required to support. This involves determining the maximum bending moment, shear force, and deflection that the beam will experience, and selecting a size and shape that will meet the required strength and stiffness criteria.

4. Placement of beam: Determine the location of the beam within the building structure. This may involve considerations such as clearance requirements, support conditions, and architectural design.

5. Design of beam connections: Design the connections between the beam and other structural elements such as columns, walls, and foundations. This includes selecting

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appropriate fasteners, brackets, and other components to ensure that the beam is securely and safely connected to the other elements.

6. Detailing and drawing of beam: Prepare detailed drawings for longitudinal, transverse reinforcement also considering ductile detailing and specifications for the beam, including dimensions, materials, and construction details.

7. Analysis of the beam design: Verify the strength and stability of the beam design using appropriate structural analysis methods. This includes checking the beam for factors such as shear, bending, deflection, and buckling to ensure that it will perform safely and reliably over its expected lifetime.

7.1 Primary Beam Connected to Shear Wall

Beam ID $= B46$

Unique Name $= 1450$

Floor $= 2nd$

Concrete Grade $= M30$

Steel Grade $= Fe\, 500\, (Longitudinal)$

$\qquad = Fe\, 415\, (Transverse)$

Design of longitudinal bars for Moment

1. Known data
   a. Overall Depth, D $= 750\ mm$
   b. Width of Beam, B $= 525mm$
   c. Nominal Cover $= 20\ mm$
   d. $\o = 25\ mm$

   Effective Depth, d $= 750 - 20 - 25 - \left(\frac{25}{2}\right)$

   $\qquad = 692.5\ mm$
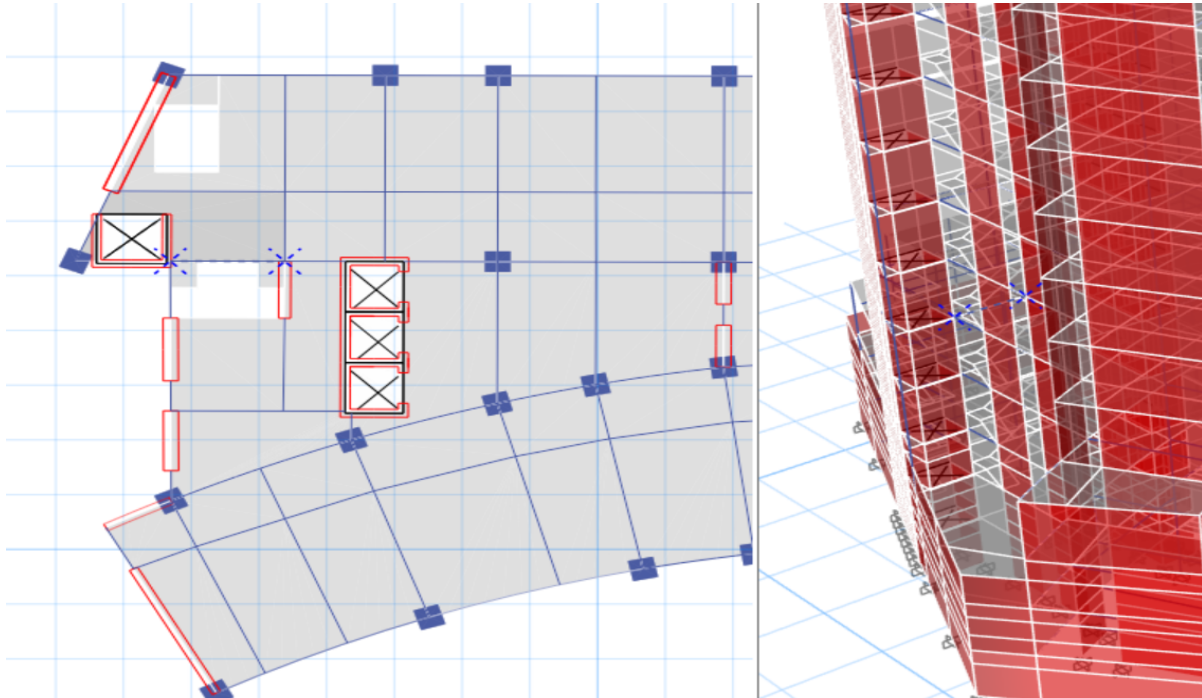
Fig 7.1 Primary Beam Connected to Shear Wall

2. Check for axial stress

    a. $Axial\ force\ =\ 0$

    b. $Axial\ stress\ =\ 0$

3. Check for member size

    a. $\frac{B}{D} = \frac{525}{750} = 0.7\ >\ 0.3$ **(OK)**

Width of Beam $= 525\ mm\ >\ 200\ mm$ OK

Clear length (L) $= 3805 - 400/2 - 300/2 = 3455mm$

$\frac{L}{D} = \frac{3455}{750} = 4.61\ >\ 4$ **(OK)**

Minimum Area Required (Ast, min)

    a. $A_{st,min}\ =\ \frac{0.85}{fy} \times b \times d$

$$= \frac{0.85}{500} \times 525 \times 692.5$$

$$= 618.056\ mm^2 (IS\ 456:2000)$$

b. $A_{st,min} = 0.24 \sqrt{\frac{fck}{fy}} \times b \times d$

$$= 0.24 \sqrt{\frac{30}{500}} \times 525 \times 692.5$$

$$= 955.83 \, mm^2 \, (IS \, 13920: 2016)$$

Adopt max $A_{st,min} = 955.83 \, mm^2$

4. Maximum area required:

   a. Tension steel $A_{st,max} = 0.04 \times b \times d$

   $$= 0.04 \times 525 \times 692.5$$

   $$= 14542.5 \, mm^2$$

   b. Compression steel, $A_{sc,max} = 0.04 \times b \times d$

   $$= 0.04 \times 525 \times 692.5 \, mm^2$$

   $$= 14542.5 \, mm^2$$

5. Limiting value of depth of Neutral Axis:

   $$\frac{X_{u,max}}{d} = \frac{0.035}{0.0055 + \frac{f_y}{1.155E_s}}$$

   $$\therefore \frac{X_{u,max}}{d} = 0.46$$

6. Limiting Moment $\left(M_{u,lim} \right)$

$$M_{u,lim} = 0.36 \times \left(\frac{x_y}{d} \right) \times \left\{ 1 - 0.4 \left(\frac{x_y}{d} \right) \right\} \times bd^2 f_{ck}$$

$$= 0.133 \times f_{ck} \times bd^2$$

$$= 0.133 \times 30 \times 525 \times 692.5^2$$

$$\therefore M_{u,lim} \, 1004.55 \, KNm$$

Moment Diagram (kN-m)

(-)Moment	-29.4168	-93.3317	-192.7093
(+)Moment	27.446	148.0724	302.162

Shear Diagram (kN)

Shear	238.4022	231.9403	234.6117

Torsion Diagram (kN-m)

Torsion	4.4077	1.821	1.4256
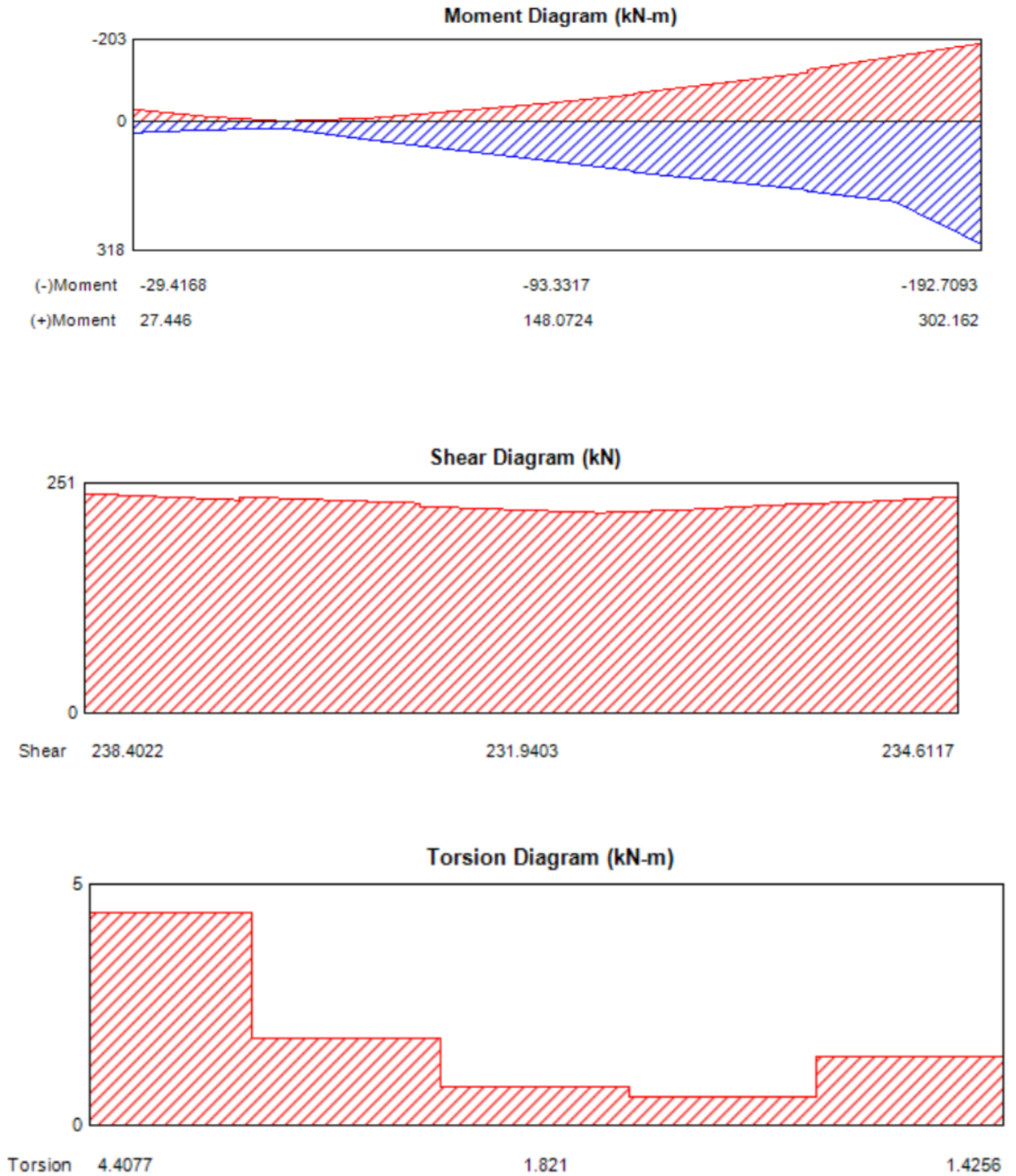
Fig 7.2: Moment, Shear and Torsional Diagram

7. Position I: Left (Top)

Governing combination = Envelop

Factored Moment, $M_u = -29.4168\ KNm$

Torsional moment, $T_u = -4.4077\ KNm$

Moment due to Torsion, $M_t = T_u \dfrac{\left(1+\frac{D}{B}\right)}{1.7}$

$$= 4.4077 \dfrac{\left(1+\frac{750}{525}\right)}{1.7}$$

$$= 6.397\ KNm$$

Required design Bending Moment $(M) = M_u + M_t = 35.7135$

$\therefore M < M_{u,lim} \Rightarrow designed\ on\ reinforced\ section$

$$M = 0.87\ f_y A_{st}\ d\left(1 - \frac{f_y A_{st}}{f_{ck}}bd\right)$$

$or,\ 35.7135 \times 10^6 = 0.87 \times 500 \times A_{st} \times 692.5\left(1 - \frac{500 \times A_{st}}{30 \times 525 \times 692.5}\right)$

$\therefore A_{st} = 119.207\ mm^2 < A_{st,min}$

$\therefore Provide\ A_{st} = 955.83\ mm^2\ (Top)$

$Percentage\ of\ Tension\ steel\ (\%) = \frac{955.83}{525 \times 750} \times 100\ 0 = 0.243\%$

$Area\ of\ compression\ steel = 50\%\ of\ A_{st}\ or\ A_{st,min}$

$Therefore,\ A_{sc} = 955.83\ mm^2\ (Top)$

$Percentage\ of\ compression\ steel\ (\%) = 0.243\%$

8. Position I: Left (Button)

Governing Combination $= Envelop$

Factored Moment, $M_u = 27.446\ KNm$

Torsional Moment, $T_u = 0$

Required Design Bending Moment $(M) = 27.446\ KNm$

$\therefore M < M_{u,lim}$ Singly reinforced

$$M = 0.87\ f_y A_{st}\ d(1 - \frac{f_y\ A_{st}}{f_{ck}\ bd})$$

$$or,\ 27.446 \times 10^6 = 0.87 \times 500 \times Ast \times 692.5(1 - \frac{500 \times A_{st}}{30 \times 525 \times 692.5})$$

$$or,\ A_{st} = 91.49\ mm^2 < A_{st,min}$$

$$\therefore,\ provide\ A_{st} = 955.83\ mm^2 (Bottom)$$

$\%\ of\ tension\ steel = 0.243\ \%$

$$A_{sc} = 955.83\ mm^2\ (Bottom)$$

9. Position: Middle (Top)

Governing Combination $= Envelop$

Factored Moment, $M_u = 93.3317\ KNm$

Torsional Moment, $T_u = 1.821$ KN-m

Moment due to Torsion $(M_t) = \dfrac{T_u\ (1+\frac{D}{B})}{1.7}$

$$= 2.601\ KNm$$

Required Design Bending Moment (M) $= M_u + M_t$

$$= 95.93 \ KNm$$

$\therefore M < M_{u,lim} \Rightarrow Singly \ reinforced \ section$

$$M = 0.87 f_y A_{st} \ d \left(1 - \frac{f_y A_{st}}{f_{ck}} bd\right)$$

$A_{st} = 323.243 \ mm^2 < A_{st,min}$

$\therefore A_{st} = 955.83 \ mm^2$

$\% \ of \ tension \ steel = 0.243\%$

$\therefore A_{sc} = 955.83 \ mm^2$

$\% \ of \ compression \ steel = 0.243 \ \%$

10. Position Middle (Bottom)

Governing combination $= Envelop$

Factored Moment, $M_u = 148.0724 \ KNm$

Torsional Moment, $T_u = 0 \Rightarrow M_t = 0$

Required Design Bending Moment, $M = M_u + T_u = 148.072 \ KNm$

$$\therefore M < M_{u,lim} \Rightarrow singly \ reinforced$$

$$M = 0.8t f_y A_{st} \ d \ (1 - \frac{f_y A_{st}}{f_{ck} \ bd})$$

$A_{st} = 503.151 \ KNm < A_{st,min}$

$\therefore A_{st} = 955.83 \ mm^2 (Bottom)$

$\% \ of \ tension \ steel = 0.243 \ \%$

$\therefore A_{sc} = 955.83 \ mm^2 \ (Top)$

$\% \ of \ compression \ steel = 0.243 \ \%$

11. Position: Right (Top)

Factored Moment, $M_u = 192.7093 \, KNm$

Torsional Moment, $T_u = 1.4256 \, KNm$

Moment due to Torsion, $M_t = \dfrac{T_u(1+\frac{D}{B})}{1.7} = 2.036 \, KNm$

Required Design Bending Moment, $M = M_u + M_t = 194.746 \, KNm$

$\therefore M < M_{u,lim} \Rightarrow singly \ reinforced$

$M = 0.87 \, f_y A_{st} \, d \left(1 - \dfrac{f_y A_{st}}{f_{ck} \, bd}\right)$

$A_{st} = 666.874 \, mm^2 < A_{st,min}$

$\therefore A_{st} = 955.83 \, mm^2 \, (Top)$

$\%\ of\ tension\ steel = 0.243\ \%$

$\therefore A_{sc} = 955.83 \, mm^2 (Bottom)$

$\%\ of\ compression\ steel = 0.243\ \%$

12. Position: Right (Bottom)

Factored Moment, $M_u = 302.1618 \, KNm$

Torsional Moment, $T_u = 0 \ \ Mt = 0$

Design BM, $M = M_u + M_t = 302.1618 \, KNm$

$\therefore M < M_{u,lim} \Rightarrow Singly \ reinforced$

$M = 0.87 \, f_y A_{st} \, d \left(1 - \dfrac{f_y A_{st}}{f_{ck} \, bd}\right)$

$A_{st} = 1053.995 \, mm^2 > A_{st,min}$

$Provide\ A_{st}\ =\ 1054\ mm^2\ (Bottom)$

$\%\ Tension\ steel\ =\ (\frac{1054}{525\ \times 750}) \times 100\ =\ 0.267\ \%$

$A_{sc}\ =\ 955.83\ mm^2\ (Top)$

$\%\ Compression\ steel\ =\ 0.243\ \%$

13. Longitudinal Detailing:

Position	Manually calculated area of steels (mm^2)	Bars	Area provided (mm^2)	% Provided
Left (Bottom) sag.	955.83	$2-25\emptyset$	981.75	0.243
Left (Top) hag.	955.83	$2-25\emptyset$	981.75	0.243
Mid (Bottom) sag.	955.83	$2-25\emptyset$	981.75	0.243
Mid (Top)	955.83	$2-25\emptyset$	981.75	0.243
Right (Bottom)	1054	$3-25\emptyset$	1472.62	0.267
Right (Top)	955.83	$2-25\emptyset$	981.75	0.243

14. Design for shear

a. From ETABS:-

At left end: $V_u = 238.4021$ KN

At mid-span: $V_u = 231.9402$ KN

At right end: $V_u = 234.6116$ KN

b. Shear force due to formation of plastic hinge at both ends of the beam plus factored load on the span.

For sway to right:

$$v_{u,a} = v_{u,a}^{D+L} - 1.4 \frac{M_u^{As}+M_u^{Bh}}{L_{AB}}$$

$$v_{u,b} = v_{u,b}^{D+L} + 1.4 \frac{M_u^{As}+M_u^{Bh}}{L_{AB}}$$

For sway to left:

$$v_{u,a} = v_{u,a}^{D+L} - 1.4 \frac{M_u^{Ah}+M_u^{Bs}}{L_{AB}}$$

$$v_{u,b} = v_{u,b}^{D+L} + 1.4 \frac{M_u^{Ah}+M_u^{Bs}}{L_{AB}}$$

$$M_u = 0.87 \, fy \, A_{st} \, d \left(1 - \frac{fy \, Ast}{fck \, bd}\right)$$

After calculation:

$$M_u^{As} = 282.43 \, KNm < M_{u,lim}$$

$$M_u^{Bs} = 441.81 \, KNm < M_{u,lim}$$

$$M_u^{Ah} = 282.43 \, KNm < M_{u,lim}$$

$$M_u^{Bh} = 282.43 \, KNm < M_{u,lim}$$

From analysis,

$$v_{u,a}^{D+L} = 57.9506 \, KN$$

$$v_{u,b}^{D+L} = 56.5464 \, KN$$

For sway to right:

$$v_{u,a} = -170.936 \, KN$$

$$v_{u,b} = 285.436 \, KN$$

For sway to left:

$$v_{u,a} = -235.52 \, KN$$

$$v_{u,b} = 350.0164 \, KN$$

Taking maximum of obtained from analysis and formation of plastic hinges.

Design forces at left end, $(V_u)_{left} = 238.4021 \, KN$

Design forces at right end, $(V_u)_{right} = 350.0164 \, KN$

Design forces at mid span, $(V_u)_{mid} = 231.94 \, KN$

15. Design of shear reinforcement

   Spacing of stirrups

   a. At left end:

   $$V_u = 234.402 \, KN$$

   $$T_u = 4.4077 \, KNm$$

$$V_e = 238.402 + 1.6 \, \frac{T_u}{b}$$

$$= 251.835 \, KN$$

Maximum permissible shear stress in concrete: $\tau_{c,max} = 3.5 \frac{N}{mm^2}$

Shear stress, $\tau_{ue} = \frac{V_e}{bd} = \frac{251.835 \times 10^3}{525 \times 692.5} = 0.69 \frac{N}{mm^2}$

Tensile steel provided at end = 0.25 %

Permissible shear strength of concrete:

$$\tau_c = 0.37 \frac{N}{mm^2} < \tau_{c,max} < \tau_{ue} \textbf{ (OK)}$$

Required capacity of shear reinforcement:

$$V_{us} = V_e - \tau_c \times b \times d$$

$$= 251.835 - (0.37 \times 525 \times 692.5 \times 10^{-3}$$

$$= 117.317 \ KN$$

Provide 2-legged $10mm\emptyset$ stirrups.

$$A_{sv} = 157.08 \ mm^2$$

Spacing of stirrups: $\qquad S_v = \dfrac{0.87 \, f_y \, A_{sv} \, d}{V_{us}}$

$$= 403.337 \ mm$$

Minimum spacing: $\qquad S_{v,min} = \dfrac{0.87 \, f_y \, A_{sv} \, d}{0.4b}$

$$= 325.214 \ mm$$

Maximum spacing: $\qquad = 0.75 \, d = 519.375 \ mm \ or \ 300mm$

So, over the distance of: $\qquad 2 \times d = 1385 \ mm$

$$S_v \le \frac{d}{4} = 173.125 \ mm$$

$$S_v \le \frac{d}{4} = 200 \ mm$$

$$\therefore S_v \le 100 \ mm$$

Hence, Provide 2- legged 10mm stirrups at a spacing of 90 mm over the distance of 1385 mm.

b. At right end:

$$V_u = 350.0164\ KN$$

$$T_u = 1.4256\ KNm$$

$$V_e = 350.0164 + 1.6\ \frac{1.4256}{0.525}$$

$$= 350.0164\ KNm$$

Maximum permissible shear stress in concrete: $\tau_{c,max} = 3.5\frac{N}{mm^2}$

Shear stress, $\tau_{ue} = \frac{V_e}{bd} = \frac{350.0164\times10^3}{525\times692.5} = 0.96\frac{N}{mm^2}$

Tensile steel provided at end = 0.25 %

Permissible shear strength of concrete:

$$\tau_c = 0.37\frac{N}{mm^2} < \tau_{c,max} < \tau_{ue}\ \textbf{(OK)}$$

Required capacity of shear reinforcement:

$$V_{us} = V_e - \tau_c\times b\times d$$

$$= 350.0164 - (0.37\times525\times692.5\times10^{-3}$$

$$= 215.50\ KN$$

Provide 2-legged $10mm\emptyset$ stirrups.

$$A_{sv} = 157.08\ mm^2$$

Spacing of stirrups: $\quad\quad S_v = \dfrac{0.87\, f_y\, A_{sv}\, d}{V_{us}}$

$= 219.46\; mm$

Minimum spacing: $\quad\quad S_{v,\,min} = \dfrac{0.87\, f_y\, A_{sv}\, d}{0.4b}$

$= 325.214\; mm$

Maximum spacing: $\quad\quad = 0.75\, d = 519.375\; mm\; or\; 300mm$

So, over the distance of: $\quad 2 \times d = 1385\; mm$

$S_v \leq \dfrac{d}{4} = 173.125\; mm$

$S_v \leq \dfrac{d}{4} = 200\; mm$

$\therefore S_v \leq 100\; mm$

Hence, Provide 2- legged 10mm stirrups at a spacing of 90 mm over the distance of 1385 mm.

c. At mid span:

$V_u = 231.94\; KN$

$T_u = 1.4256\; KNm$

$V_e = 231.94 + 1.6\,\dfrac{1.4256}{b}$

$= 236.285vKN$

Maximum permissible shear stress in concrete: $\tau_{c,\,max} = 3.5\dfrac{N}{mm^2}$

Shear stress, $\tau_{ue} = \dfrac{V_e}{bd} = \dfrac{236.285 \times 10^3}{525 \times 692.5} = 0.65 \dfrac{N}{mm^2}$

Tensile steel provided at end = 0.37 %

Permissible shear strength of concrete:

$\tau_c = 0.43 \dfrac{N}{mm^2} < \tau_{c,max} < \tau_{ue}$ **(OK)**

Required capacity of shear reinforcement:

$V_{us} = V_e - \tau_c \times b \times d$

$= 236.285 - (0.43 \times 525 \times 692.5 \times 10^{-3}$

$= 79.95 \, KN$

Provide 2-legged $10 mm\emptyset$ stirrups.

$A_{sv} = 157.08 \, mm^2$

Spacing of stirrups: $\qquad S_v = \dfrac{0.87 f_y A_{sv} d}{V_{us}}$

$= 403.337 \, mm$

Minimum spacing: $\qquad S_{v,min} = \dfrac{0.87 f_y A_{sv} d}{0.4b}$

$= 325.214 \, mm$

Maximum spacing: $\qquad = 0.75 \, d = 519.375 \, mm \text{ or } 300mm$

So, over the distance of: $\quad 2 \times d = 1385 \, mm$

$$S_v \leq \frac{d}{4} = 173.125 \, mm$$

$$S_v \leq \frac{d}{4} = 200 \, mm$$

$$\therefore S_v \leq 100 \, mm$$

Hence, Provide 2- legged 10mm stirrups at a spacing of 90 mm over the distance of 1385 mm.

16. Design of side Reinforcement

   Since, d > 450 mm provide 0.1% of bD of steel.

   Reinforcement along vertical sides

$$A_{sr} = 0.001 \times 525 \times 750 = 393.75 \, mm^2$$

   Provide 3-16 mm Ø bar along vertical sides.

$$Actual \, A_{sr} = 461.81 \, mm^2 > A_{sr} \, (OK)$$

17. Check for deflection

   Clear span = $3455 \, mm$

   Width of support = $300mm$

   1/12 of clear span = $287.92 \, mm$

   Width of support > $\frac{1}{12}$ of clear span so L$_x$ is taken as clear span

   $\propto = 26$ (For continuous)

   $\beta = 1$ (span less than 10m)

   For λ,

$$F_s = 0.58 \, f_y \, \frac{Area \, of \, steel \, req}{Area \, of \, steel \, provided}$$

$$= 0.58 \times 500 \times \frac{955.83}{987.75}$$

$$= 282.34 \, N/mm^2$$

Therefore, % Ast = 0.25

$\lambda = 1.36$

$\gamma = 1.07$

$\delta = 1$

So, $\propto \beta \gamma \delta \lambda = 37.83$

$\frac{Lx}{d} = \frac{3455}{692.5} = 4.989 < 37.83$ **(OK)**

18. Check for development length

$$L_d = \frac{\emptyset \rho_s}{4\tau_{bd}}$$

$$= \frac{25 \times 0.87 \times 500}{4 \times 1.5 \times 1.6} = 1132.81 \ mm \text{ (For tension)}$$

$$= \frac{25 \times 0.87 \times 500}{4 \times 1.5 \times 1.25} = 1450 \ mm \text{ (For compression)}$$

Also,

$$L_d \leq 1.3 \frac{M}{V} + L_o \quad \text{where} \quad M = 0.87 f_y A_{st,prov} \left(d - \frac{f_y A_{st,prov}}{f_{ck} * b} \right)$$

$$A_{st,prov} = 1472.62 \ mm$$

$$\therefore M = 413.66 \ KNm$$

$$V = 231.94 \ KN$$

$$Lo = \frac{330}{2} - 20 = 130 \ mm$$

$$1.3 \frac{M}{V} + Lo = 2448.52 \ mm > L_d \text{ (OK)}$$

7.2 Primary Beam

Beam ID $=\ B13$

Unique Name $=\ 1714$

Floor $=\ 2nd$

Concrete Grade $=\ M30$

Steel Grade $=\ Fe\ 500\ (Longitudinal)$

$\qquad\ =\ Fe\ 415\ (Transverse)$
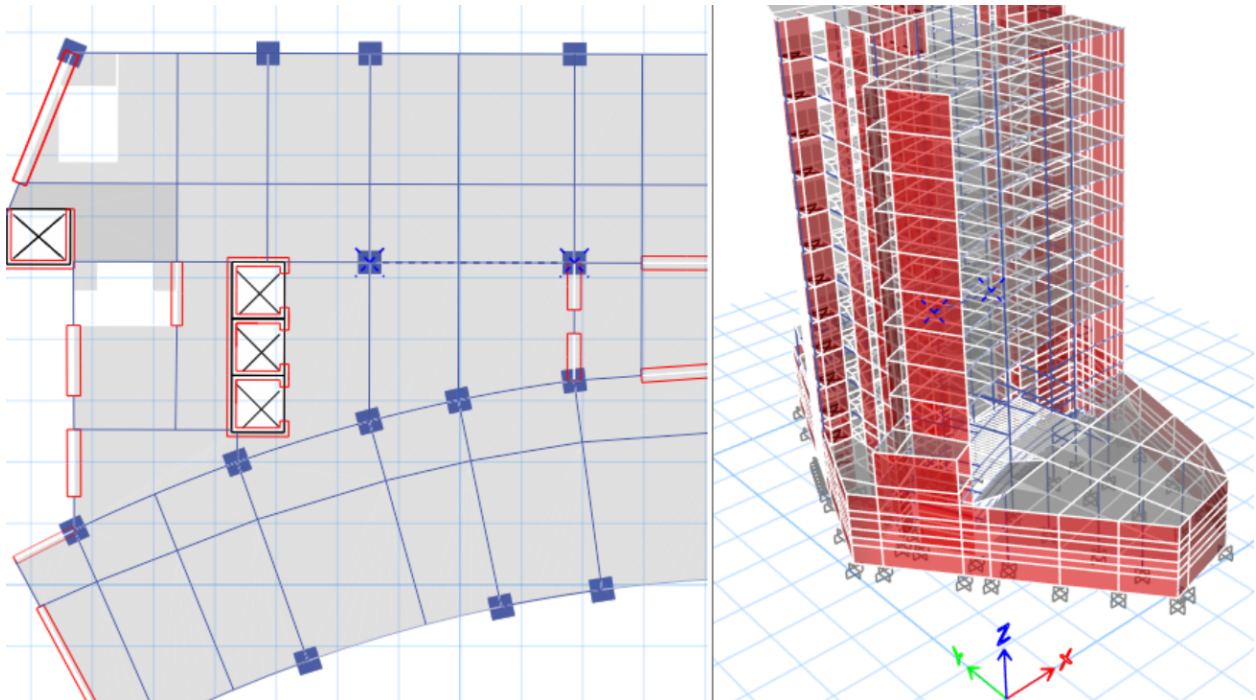
Fig: 7.3 Primary Beam

Design of longitudinal bars for moment

1. Known data

    a. Overall Depth, D $= 800\ mm$

    b. Width of Beam, B $= 650mm$

    c. Nominal Cover $= 20\ mm$

    d. ø $= 25\ mm$

    Effective Depth, d $=\ 800\ -\ 20\ -\ \left(\frac{25}{2}\right)$

    $\qquad =\ 767.5\ mm$

2. Check for axial stress

   Axial force from ETABS = 0

   Axial stress $= \; 0 \times \dfrac{10^3}{800 \times 650} \; = \; 0$ **(OK)**

3. Check for member size

   $\dfrac{B}{D} \; = \; 650 \,/\, 800 \; = \; 0.8125$

   Clear length, $l \; = \; 7512.3 - \dfrac{850}{2} - \dfrac{850}{2} = \; 6662.3 \; mm$

   $L/D \; = \; \dfrac{6662.3}{800} = \; 8.33 \; > 4$ **(OK)**

4. Minimum Area Required ($A_{st,min}$)

   $A_{st,min} = \dfrac{0,85}{f_y} \; b \times d \; = \dfrac{0.85}{500} \times 690 \times 767.5 \; = \; 848.0875 \; mm^2$

   $A_{st,min} = \dfrac{0.24 \times \sqrt{f_{ck}}}{f_y} \times b \times d \; = \; \dfrac{0.24 \times \sqrt{30}}{500} \times 650 \times 767.5 \; = \; 1311.57 \; mm^2$

   Adopt maximum $A_{st,min} \; = \; 1311.57 \; mm^2$

   Maximum area required,

   a. Tension steel, $A_{st,max} \; = \; 0.04 \times b \times d$

   $= \; 0.04 \times 650 \times 767.5$

   $= \; 19955 \; mm^2$

   b. Compression steel, $A_{sc,max} = 0.04 \times b \times d$

$= \; 19955 \; mm^2$

5. Limiting value of depth of Neutral Axis:

$$\frac{X_{u,max}}{d} = \frac{0.0035}{0.0055 + \frac{f_y}{1.155E_s}}$$

$$\therefore \frac{X_{u,max}}{d} = 0.46$$

6. Limiting Moment $\left(M_{u,lim}\right) = 0.36 \times \left(\frac{x_y}{d}\right) \times \left\{1 - 0.4\left(\frac{x_y}{d}\right)\right\} \times bd^2 f_{ck}$

$$= 0.36 \times 0.46 \times \{1 - 0,4 \times 0.46\} + 650 \times 767.5^2 \times 30$$

$$\therefore M_{u,lim} = 1552.18 \, KNm$$

Moment Diagram (kN-m)

(-)Moment	-741.0241	-42.9197	-784.012
(+)Moment	140.5619	676.0154	178.5352

Shear Diagram (kN)

Shear	468.1016	379.3773	450.1446
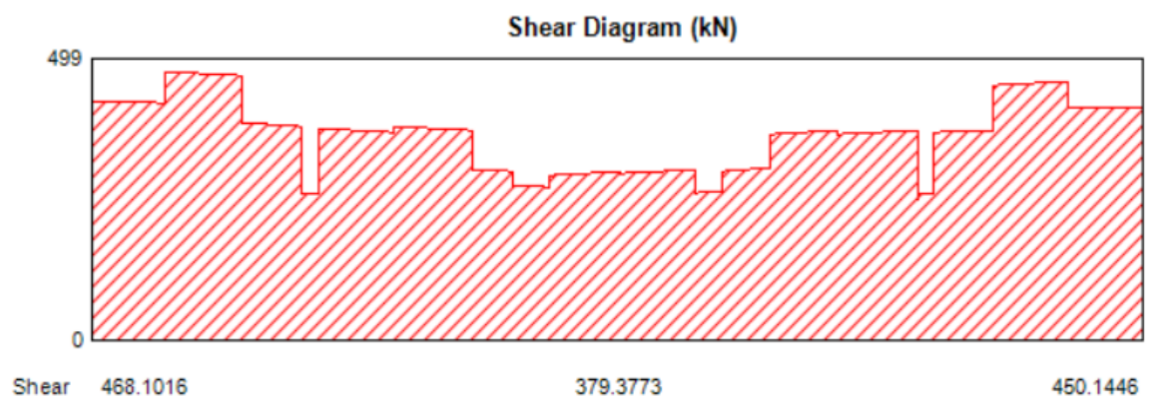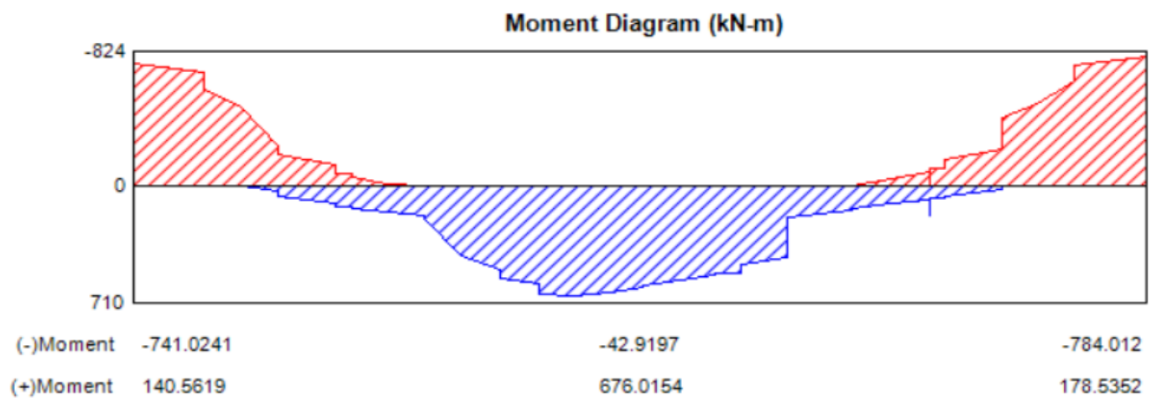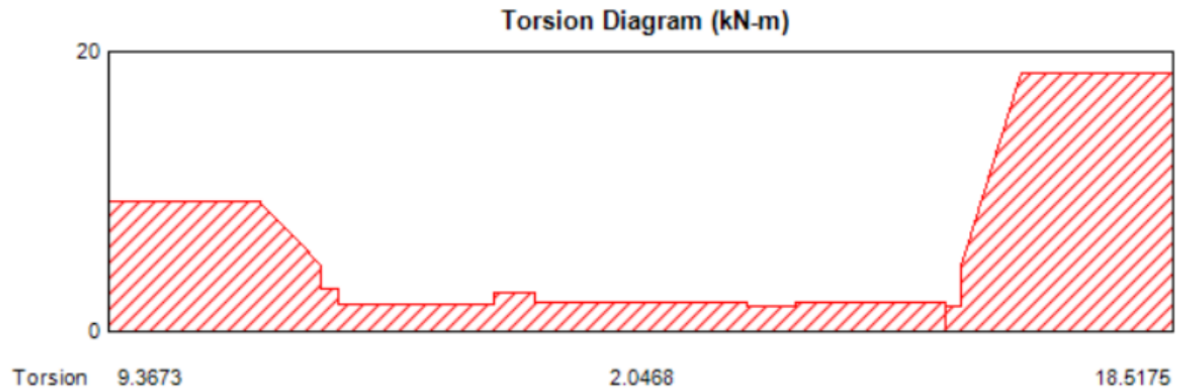
Fig 7.4: Moment, Shear and Torsion diagram for Primary Beam

7. Position: I Left end:

Governing envelope combination $=$ *envelope*

Factored moment, $M_u = 741.0241 \, KNm$

Torsional moment, $T_u = 9.9673 \, KNm$

Moment due to torsion $\left(M_t\right) = T_u \dfrac{\left(1+\frac{D}{b}\right)}{1.7}$

$$= 12.29 \, KNm$$

Required design bending moment (M)

$$M = M_u + M_t = 759.914 \, KNm$$

Since, M<$M_{u,lim}$ design as singly reinforced section

design as singly reinforced section

$$M = 0.87 f_y \times A_{st} \times d \left(1 - \frac{f_y A_{st}}{f_{ck} bd}\right)$$

$\left(A_{st}\right)_{left} = 2458.25 mm^2 \, (Bottom) > Ast, min \, \textbf{(OK)}$

$\% \, tension \, of \, steel = 0.47\%$

Area of compression steel $\left(A_{sc}\right) = 1289.125mm^2 < A_{st,min}$ **(OK)**

So, $\left(A_{sc}\right)_{left} = 1311.57mm^2$

And % *of compression steel* $= 0.23\%$

8. Position: II Mid-Span:

Governing envelope combination $= envelope$

Factored moment, $M_u = 741.0241\ KNm$

Torsional moment, $T_u = 9.9673\ KNm$

Moment due to torsion $\left(M_t\right) = T_u \dfrac{\left(1+\frac{D}{b}\right)}{1.7}$

$\qquad\qquad\qquad = 12.29\ KNm$

Required design bending moment (M)

$M = M_u + M_t = 759.914\ KNm$

Since, M<$M_{u,lim}$ design as singly reinforced section

 design as singly reinforced section

$M = 0.87\ f_y \times A_{st} \times d\left(1 - \dfrac{f_y A_{st}}{f_{ck}bd}\right)$

$\left(A_{st}\right)_{mid} = 2193.64\ mm^2 > A_{st,min}$ **(OK)**

% *tension of steel* $= 0.42\%$

Area of compression steel $\left(A_{sc}\right)_{mid} = 0.5 \times\left(A_{st}\right)_{mid}$

$= 1096.82\ mm^2$

9. Position: III Right end:

   Governing envelope combination $=$ *envelope*

   Factored moment, $M_u = 784.012\ KNm$

   Torsional moment, $T_u = 18.5175\ KNm$

   Moment due to torsion $\left(M_t\right) = T_u\dfrac{\left(1+\frac{D}{b}\right)}{1.7}$

   $$= 24.3\ KNm$$

   Required design bending moment (M)

$$M = M_u + M_t = 808.312\ KNm$$

   Since, M<$M_{u,lim}$ design as singly reinforced section

   $$M = 0.87\,f_y \times A_{st} \times d\left(1 - \frac{f_y A_{st}}{f_{ck}\,bd}\right)$$

   $\left(A_{st}\right)_{right} = 2620.718\ mm^2 > A_{st,min}$

   $\%\ tension\ of\ steel = 0.5\%$

   Area of compression steel $\left(A_{sc}\right)_{right} = 1310.36\ mm^2$

10. Longitudinal Detailing

Position	Manually calculated area of steels (mm^2)	Bars	Area provided (mm^2)	% Provided
Left (Top)	1311.57	$3 - 25\emptyset$	1472.62	0.28
Left (Bottom)	2458.25	$6 - 25\emptyset$	2945.24	0.56
Mid (Top)	1096.82	$3 - 25\emptyset$	1472.62	0.28
Mid (Bottom)	2193.64	$5 - 25\emptyset$	2454.37	0.47
Right (Top)	1310.36	$3 - 25\emptyset$	1472.62	0.28
Right (Bottom)	2620.72	$6 - 25\emptyset$	2945.24	0.56

11. Design for shear

a. From ETABS:-

At left end: V_u = 468.1016 KN

At mid-span: V_u = 379.377 KN

At right end: V_u = 450.1446 KN

b. Shear force due to formation of plastic hinge at both ends of the beam plus factored load on the span.

For sway to right:

$$v_{u,a} = v_{u,a}^{D+L} - 1.4\,\frac{M_u^{As}+M_u^{Bh}}{L_{AB}}$$

$$v_{u,b} = v_{u,b}^{D+L} + 1.4\,\frac{M_u^{As}+M_u^{Bh}}{L_{AB}}$$

For sway to left:

$$v_{u,a} = v_{u,a}^{D+L} - 1.4 \frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}}$$

$$v_{u,b} = v_{u,b}^{D+L} + 1.4 \frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}}$$

$$M_u = 0.87 \, fy \, Ast \, d \left(1 - \frac{fy \, Ast}{fck \, bd}\right)$$

$$M_u^{As} = 886.55 \, KNm < M_{u,lim}$$

$$M_u^{Bs} = 886.55 \, KNm < M_{u,lim}$$

$$M_u^{Ah} = 467.464 \, KNm < M_{u,lim}$$

$$M_u^{Bh} = 467.464 \, KNm < M_{u,lim}$$

From analysis,

$$v_{u,a}^{D+L} = 504.0628 \, KN$$

$$v_{u,b}^{D+L} = 464.4283 \, KN$$

For sway to right:

$$v_{u,a} = 503.78 \, KN$$

$$v_{u,b} = 464.713 \, KN$$

For sway to left:

$$v_{u,a} = 504.35 \, KN$$

$$v_{u,b} = 464.144 \, KN$$

Taking maximum obtained from analysis and formation of plastic hinges.

Design forces at left end, $(V_u)_{left} = 504.35\ KN$

Design forces at right end, $(V_u)_{right} = 464.713\ KN$

Design forces at mid span, $(V_u)_{mid} = 379.377\ KN$

12. Design of shear reinforcement

Providing 2-legged 10 mmm phi stirrups,

Maximum spacing = 0.75d = 0.75 * 767.5 = 567.645mm or 300m (whichever is smaller)

Spacing of reinforcements;

a. At left end:

$$V_u = 504.35\ KN$$

$$T_u = 9.3673\ KNm$$

$$V_e = 504.35 + 1.6\ \frac{T_u}{b}$$

$$= 527.41\ KN$$

Maximum permissible shear stress in concrete: $\tau_{c,max} = 3.5\ \frac{N}{mm^2}$

Shear stress, $\tau_{ue} = \frac{V_e}{bd} = \frac{527.41 \times 10^3}{767.5 \times 692.5} = 1.057\ \frac{N}{mm^2}$

Tensile steel provided at end = 0.56 %

Permissible shear strength of concrete:

$$\tau_c = 0.5216\ \frac{N}{mm^2} < \tau_{c,max} < \tau_{ue}\ \textbf{(OK)}$$

Required capacity of shear reinforcement:

$$V_{us} = V_e - \tau_c \times b \times d$$

$$= 527.41 - (0.5216 \times 767.5 \times 650 \times 10^{-3})$$

$$= 267.197 \, KN$$

Provide 2-legged $10mm\emptyset$ stirrups.

$$A_{sv} = 157.08 \, mm^2$$

Spacing of stirrups: $\qquad S_v = \dfrac{0.87 \, f_y \, A_{sv} \, d}{V_{us}}$

$$= 403.337 \, mm$$

Minimum spacing: $\qquad S_{v,min} = \dfrac{0.87 \, f_y \, A_{sv} \, d}{0.4b}$

$$= 196.27 \, mm$$

Maximum spacing: $\qquad = 0.75 \, d = 575.625 \, mm \, or \, 300mm$

For spacing of links [IS 13920:2016]

So, over the distance of: $\qquad 2 \times d = 1535 \, mm$

$$S_v \le \frac{d}{4} = 191.875 \, mm$$

$$S_v \le \frac{d}{4} = 200 \, mm$$

$$\therefore S_v \le 100 \, mm$$

Design spacing is less than the required minimum spacing as per ductile detailing. Hence, Provide 2- legged 10mm stirrups at a spacing of 90 mm over the distance of 1535 mm.

b. At right end:

$$V_u = 464.713 \ KN$$

$$T_u = 18.5175 \ KNm$$

$$V_e = 464.713 + 1.6 \frac{18.5175}{0.600}$$

$$= 510.29 \ KNm$$

Maximum permissible shear stress in concrete: $\tau_{c,max} = 3.5 \frac{N}{mm^2}$

Shear stress, $\tau_{ue} = \frac{V_e}{bd} = \frac{510.29 \times 10^3}{650 \times 767.5} = 1.046 \frac{N}{mm^2}$

Tensile steel provided at end = 0.56 %

Permissible shear strength of concrete:

$$\tau_c = 0.5216 \frac{N}{mm^2} < \tau_{c,max} < \tau_{ue} \ \textbf{(OK)}$$

Required capacity of shear reinforcement:

$$V_{us} = V_e - \tau_c \times b \times d$$

$$= 510.29 - (0.5216 \times 650 \times 767.5 \times 10^{-3})$$

$$= 250.07 \ KN$$

Provide 2-legged $10mm\emptyset$ stirrups.

$$A_{sv} = 157.08 \ mm^2$$

Spacing of stirrups: $S_v = \dfrac{0.87\, f_y\, A_{sv}\, d}{V_{us}}$

$= 209.713\ mm$

Minimum spacing: $S_{v,min} = \dfrac{0.87\, f_y\, A_{sv}\, d}{0.4b}$

$= 325.214\ mm$

Maximum spacing: $= 0.75\, d = 519.375\ mm\ or\ 300mm$

So, over the distance of: $2 \times d = 1385\ mm$

$S_v \leq \dfrac{d}{4} = 173.125\ mm$

$S_v \leq \dfrac{d}{4} = 200\ mm$

$\therefore S_v \leq 100\ mm$

Hence, Provide 2- legged 10mm stirrups at a spacing of 90 mm over the distance of 1535 mm.

c. At mid span:

$V_u = 379.377\ KN$

$T_u = 2.0468\ KNm$

$V_e = 379.377 + 1.6\,\dfrac{2.0468}{0.650}$

$= 384.415\ KN$

Maximum permissible shear stress in concrete: $\tau_{c,max} = 3.5\,\dfrac{N}{mm^2}$

Shear stress, $\tau_{ue} = \dfrac{V_e}{bd} = \dfrac{384.415 \times 10^3}{650 \times 767.5} = 0.77 \dfrac{N}{mm^2}$

Tensile steel provided at end $= 0.47 \%$

Permissible shear strength of concrete:

$\tau_c = 0.4844 \dfrac{N}{mm^2} < \tau_{c,max} < \tau_{ue}$ **(OK)**

Required capacity of shear reinforcement:

$V_{us} = V_e - \tau_c \times b \times d$

$$= 384.415 - (0.4844 \times 650 \times 767.5 \times 10^{-3})$$

$= 142.76 \ KN$

Provide 2-legged $10 mm\emptyset$ stirrups.

$A_{sv} = 157.08 \ mm^2$

Spacing of stirrups: $\qquad S_v = \dfrac{0.87 f_y A_{sv} d}{V_{us}}$

$= 367.35 \ mm$

Minimum spacing: $\qquad S_{v,min} = \dfrac{0.87 f_y A_{sv} d}{0.4b}$

$= 325.214 \ mm$

Maximum spacing: $\qquad = 0.75 \ d = 519.375 \ mm \ or \ 300mm$

So, over the distance of: $\qquad 2 \times d = 1385 \ mm$

$S_v \le \frac{d}{4} = 173.125\ mm$

$S_v \le \frac{d}{4} = 200\ mm$

$\therefore S_v \le 100\ mm$

Hence, Provide 2- legged 10mm stirrups at a spacing of 90 mm over the distance of 1535 mm.

13. Design of side Reinforcement

Since, d > 450 mm provide 0.1% of bD of steel.

Reinforcement along vertical sides

$A_{sr} = 0.001 \times 767.5 \times 650 = 498.875\ mm^2$

Provide 4-14 mm Ø bar along vertical sides.

$Actual\ A_{sr} = 615.752\ mm^2 > A_{sr}\ (OK)$

14. Check for deflection

Clear span $= 6662.3\ mm$

Width of support $= 850\ mm$

1/12 of clear span $= 555.19\ mm$

Width of support $> \frac{1}{12}$ of clear span so L_x is taken as clear span:

$L_x = 6662.3\ mm$

$\propto = 26$ (For continuous)

$\beta = 1$ (span less than 10m)

For λ,

$F_s = 0.58\ f_y\ \frac{Area\ of\ steel\ req}{Area\ of\ steel\ provided}$

$$= 0.58 \times 500 \times \frac{2193.64}{2454.37}$$

$$= 259.193 \; N/mm^2$$

Therefore, % Ast = 0.47%

$\lambda = 1.19$

$\gamma = 1.08$

$\delta = 1$

So, $\propto \beta \gamma \delta \lambda = 33.4152$

$\frac{Lx}{d} = \frac{6662.3}{767.5} = 8.6 < 33.4152$ **(OK)**

15. Check for development length:

$$L_d = \frac{\varnothing \rho_s}{4\tau_{bd}}$$

$$= \frac{25 \times 0.87 \times 500}{4 \times 1.5 \times 1.6} = 1132.81 \; mm \text{ (For tension)}$$

$$= \frac{25 \times 0.87 \times 500}{4 \times 1.5 \times 1.6 \times 1.25} = 906.25 \; mm \text{ (For compression)}$$

Also,

$$L_d \leq 1.3 \frac{M}{V} + L_o$$

$$M = 0.87 f_y A_{st,prov} \left(d - \frac{f_y A_{st,prov}}{f_{ck} * bd} \right)$$

$$= 0.87 \times 500 \times 2945.54 \times 767.5 \left(1 - \frac{500 \times 2945.54}{30 \times 650 \times 767.5} \right)$$

$\therefore M = 886.63 KNm$

$V = 464.713 \; KN, \; 90° \; bend, \; so$

$Lo = 8 \times \varnothing = 200 \; mm$

$1.3 \frac{M}{V} + Lo = 2680.28 \; mm > L_d$ **(OK)**

7.3 Secondary Beam

Steel Grade = Fe 500

Concrete Grade = M 90

Secondary Id = Sec Beam 600 × 400

Length: 3250 4260.9

Beam ID: B31 B32

Design of Longitudinal bare for moment

1. Known Data:

    Overall depth of beam, $D = 600mm$

    Width of beam, $B = 400\ mm$

    Nominal cover $= 20\ mm$

    Diameter of the bar, $\emptyset = 20\ mm$

    Considering bars in a single layer,

    Effective depth, $d = 600 - 20 - \frac{20}{2} = 570mm$

2. Check for axial stress:

    Axial force from ETABS $= 0KN$

    Axial stress $= 0$ **(OK)**

3. Check for member size:

    $\frac{B}{D} = \frac{400}{600} = 0.667 > 0.3$ **(OK)**

4. Clear length:

    i) For B31

$$L1 = 3250 - \frac{525}{2} * - \frac{490}{2} = 2787.5 \ mm$$

$$\frac{L1}{D} = \frac{2789.5}{600} = 4.64 > 4 \ \textbf{(OK)}$$

ii) For B32

$$L2 = 4260.3 - \frac{525}{2} * - \frac{400}{2} = 3797.8 \ mm$$

$$\frac{L2}{D} = \frac{3797.8}{600} = 6.33 > 4 \ (OK)$$

5. Minimum area required $(A_{st,min})$

   i. $A_{st,min} = \frac{0.85}{f_y} \times b \times d = \frac{0.85}{500} \times 400 \times 570 = 387.6 \ mm^2$

   ii. $A_{st,min} = 0.24 \times \sqrt{\frac{f_{ck}}{f_y}} \times b \times d$

$$= 0.24 \times \sqrt{\frac{\sqrt{30}}{500}} \times 400 \times 570$$

$$= 599.43 \ mm^2$$

Adopt max $A_{st,min} = 599.43 \ mm^2$

6. Maximum are required $(A_{st,max})$

   iii.   Tension steel $(A_{st,max}) = 0.04 \times b \times d$

$$= 0.04 \times 400 \times 570.$$

$$= 9120 \ mm^2$$

  b) Compression steel $(A_{sc,max}) = 0.04 \times b \times d$

$$= 0.04 \times 400 \times 570.$$

$$= 9120 \ mm^2$$

6. For Beam B31:

a) For position I left

Governing combination $=$ *Envelop*

Factored Moment, $M_u = 19.5128\ KNm$

Torsional Moment, $T_u = 0.6823\ KNm$

Moment due to torsion $(M_t) = T_u \dfrac{\left(1 + \frac{D}{b}\right)}{1.7}$

$= \dfrac{0.6623}{1.7} * \left(1 + \dfrac{600}{400}\right)$

$= 0.9740\ KNm.$

Required design bonding Moment $(M) = M_u + M_t$

$= 19.5128 + 0.9740$

$= 20.4868 KNm$

As $M_u < M_{u,lim}$, it is designed as a singly reinforced section.

We have,

$$M = 0.87\,f_y \times A_{st} \times d\left(1 - \dfrac{f_y A_{st}}{f_{ck}\,bd}\right)$$

or, $20.48 \times 10^6 = 0.87 \times 500 \times A_{st} \times 570\left(1 - \dfrac{500 \times A_{st}}{30 \times 400 \times 570}\right)$

$\therefore A_{st} = 63.10 < A_{st,min}$

$\therefore Provide\ A_{st} = 599.43\ mm^2$

$\therefore A_{sc} = 599.43\ mm^2$

$\%\ of\ tension\ steel = 0.25\%$

$\% \, of \, compression \;=\; 0.25\%$

b) For position II (middle)

Governing combination $=\; Envelope$

Factored Moment, $M_u \;=\; 101.1357 KNm$

Torsional Moment, $T_u \;=\; 0.6623 \, KNm$

Moment due to torsion, $M_t \;=\; T_u \dfrac{1+\frac{D}{b}}{1.7}$

$=\; 0.6623 \;=\; 0.9740 \, KNm$

Required design bending moment (M) is

$M \;=\; 101.1357 \;+\; 0.9740 \;=\; 102.11 \, KNm$

Since, $M_u < M_{u,lim}$, it is designed as a singly reinforced beam.

$$M \;=\; 0.87 \, f_y \times A_{st} \times d \left(1 \;-\; \frac{f_y A_{st}}{f_{ck} bd}\right)$$

$or, \; 102.11 \times 10^6 \;=\; 0.87 \times 500 \times A_{st} \times 570 \times (1 - \frac{500 \times A_{st}}{30 \times 400 \times 570})$

$\therefore A_{st} \;=\; 425.022 \, mm^2 \; < A_{st,min}$

$\therefore Provide \, A_{st} \;=\; 599.43 \, mm^2$

$\therefore A_{sc} \;=\; 599.43 \, mm^2$

$\% \, of \, tensile \, steel \;=\; 0.25\%$

$\% \, of \, compression \, steel \;=\; 0.25\%$

c. For position III (right)

Governing Combination $=\; Envelope$

Factored Moment, Mu $=\; 160.4501 \, KNm$

Torsional Moment, $T_u = 0.6866\ KNm$

Moment due to torsion, $M_t = T_u \dfrac{\left(1+\frac{D}{b}\right)}{1.7} = 1.01\ KNm$

Design bending moment $(M) = 161.46\ KNm$

Since, $M_u < M_{u,lim}$, it is designed as a singly reinforced beam.

$$M = 0.87\ f_y \times A_{st} \times d \left(1 - \frac{f_y A_{st}}{f_{ck}\,bd}\right)$$

$or,\ 161.46 = 0.87 \times 500 \times A_{st} \times 570 \times \left(1 - \dfrac{500 \times A_{st}}{30 \times 400 \times 570}\right)$

$\therefore A_{st} = 685.53\ mm^2 > A_{st,min}$

$\therefore,\ Provide\ A_{st} = 685.53\ mm^2$

$\%\ of\ tensile\ steel = 0.29\%$

$\therefore Provide\ A_{sc} = 50\% * Ast = 342.76\ mm^2 < A_{sc,min}$

$\therefore Provide\ A_{sc} = 599.43\ mm^2$

$\%\ of\ compression\ steel = 0.25\ \%$

7. For Beam B32:

    a) For position I left

    Governing combination $= Envelop$

    Factored Moment, $M_u = 140.34\ KNm$

    Torsional Moment, $T_u = 0.7712\ KNm$

    Moment due to torsion $\left(M_t\right) = T_u \dfrac{\left(1+\frac{D}{b}\right)}{1.7}$

$= \dfrac{0.7712}{1.7} * \left(1 + \dfrac{D}{b}\right)$

$$= 0.9740 \, KNm$$

Required design bonding Moment $(M) = M_u + M_t$

$$= 140.32 + 1.134$$

$$= 141.47 \, KNm$$

As $M_u < M_{u,lim}$, it is designed as a singly reinforced section.

We have,

$$M = 0.87 \, f_y \times A_{st} \times d \left(1 - \frac{f_y A_{st}}{f_{ck} \, bd}\right)$$

$or,$ $141.47 \times 10^6 = 0.87 \times 500 \times A_{st} \times 570 \, (1 - \frac{500 \times A_{st}}{30 \times 400 \times 570})$

$$\therefore A_{st} = 596.57 \, mm^2 < A_{st,min}$$

$$\therefore Provide \, A_{st} = 599.43 \, mm^2$$

$$\% \, of \, tension \, steel = 0.25\%$$

$$\therefore Provide \, A_{sc} = 599.43 \, mm^2$$

$$\% \, of \, compression \, steel = 0.25\%$$

b) For position II (middle)

Governing combination $= Envelope$

Factored Moment, $M_u = 208.055 KNm$

Torsional Moment, $T_u = 0.6899 \, KNm$

Moment due to torsion, $M_t = T_u \frac{1+\frac{D}{b}}{1.7}$

$$= \frac{0.6889\left(1+\frac{D}{b}\right)}{1.7} = 1.01 \, KNm$$

Required design bending moment (M) is

$M = 208.06 + 1.01 = 209.07 \ KNm$

Since, $M_u < M_{u,lim}$, it is designed as a singly reinforced beam.

$$M = 0.87 \ f_y \times A_{st} \times d \left(1 - \frac{f_y A_{st}}{f_{ck} bd}\right)$$

$or, \ 209.07 \times 10^6 = 0.87 \times 500 \times A_{st} \times 570 \times (1 - \frac{500 \times A_{st}}{30 \times 400 \times 570})$

$\therefore A_{st} = 902.77 mm\text{\textasciicircum}2 \ < A_{st,min}$

$\therefore Provide \ A_{st} = 902.77 \ mm^2$

$\% \ of \ tensile \ steel = 0.25\%$

$\therefore Provide \ A_{sc} = 0.5 \ * \ 902.77 \ mm^2 = 451.385 \ mm^2 \ < \ A_{sc,min}$

$\therefore A_{sc} = 599.43 \ mm^2$

$\% \ of \ compression \ steel = 0.25\%$

c. For position III (right)

Governing Combination $= \ Envelope$

Factored Moment, $M_u = 13.01 KNm$

Torsional Moment, $T_u = 0.122 \ KNm$

Moment due to torsion, $M_t = T_u \frac{\left(1 + \frac{D}{b}\right)}{1.7} = 0.1794 \ Knm$

Design bending moment $(M) = 13.19 \ KNm$

Since, $M_u < M_{u,lim}$, it is designed as a singly reinforced beam.

$$M = 0.87 f_y \times A_{st} \times d \left(1 - \frac{f_y A_{st}}{f_{ck} bd}\right)$$

$$or, \ 13.19 \times 10^6 = 0.87 \times 500 \times A_{st} \times 570 \left(1 - \frac{500 \times A_{st}}{30 \times 400 \times 570}\right)$$

$$\therefore A_{st} = 53.40 \ mm^2 < A_{st,min}$$

$$\therefore Provide \ A_{st} = 599.43 \ mm^2$$

$$\therefore Provide \ A_{sc} = 599.43 mm^2$$

$$\% \ of \ tensile \ steel = 0.25\%$$

$$\% \ of \ compression \ steel = 0.25 \%$$

8. Longitudinal Detailing

   Beam B31:

	Position	Manually Calculated Area of Steel Bars	Bars	Area provided	Provided
sag	Left (bottom)	599.43	2-20	628.32	0.26
hog	Left (top)	599.43	2-20	628.32	0.26
sag	Mid (bottom)	599.43	2-20	628.32	0.26
hog	Mid (top)	599.43	2-20	628.32	0.26
Sag	Right (bottom)	685.53	3-20	942.48	0.39
Hog	Right (top)	599.43	2-20	628.32	0.26

Beam B32

	Position	Manually Calculated Area of Steel Bars	Bars	Area provided	Provided
sag	Left (bottom)	599.43	2-20	628.32	0.26
hog	Left (top)	599.43	2-20	628.32	0.39
sag	Mid (bottom)	902.77	3-20	942.48	0.26
hog	Mid (top)	599.43	2-20	628.32	0.26
sag	Right (bottom)	599.43	2-20	628.32	0.26
hog	Right (top)	599.43	2-20	628.32	0.26

8. Design of transverse bars for shear reinforcement

Design of shear forces:

a) For B31:

$$V_{ui} = 27.3872 \ KN$$

$$V_{umid} = 63.3373 \ KN$$

$$V_{uj} = 55.286 \ KN$$

Shear force due to formation of plastic hinge:

Grid	$M_s(KNm)$	$M_h(KNm)$	$V_g(KNm)$
i	148.63	148.63	27.3872
j	214.59	148.63	55.286

For sway to right:

$$V_{ui} = V_{gi} = 1.4\left[\frac{M_{uis}+M_{uju}}{L}\right]$$

$$V_{uj} = V_{gj} = 1.4\left[\frac{M_{uis}+M_{uju}}{L}\right]$$

For sway to left:

$$V_{ui} = V_{gi} = 1.4\left[\frac{M_{uis}+M_{uju}}{L}\right]$$

$$V_{uj} = V_{gj} = 1.4\left[\frac{M_{uis}+M_{uju}}{L}\right]$$

Computed V_{ui}

Right sway = 27.24 KN

Left sway = 27.57 KN

Computed V_{uj}

Right sway = 55.38 KN

Left sway = 55.04 KN

Maximum design shear force at end i = 27.57 KN

Maximum design shear force at end j = 55.38 KN

Torsional moment at end i (T_{ui}) = 0.66 KN

Torsional moment at end j (T_{uj}) = 0.69 KN

For end i:

a) Equivalent factored shear force: $V_e = V_{dui} + 1.6\frac{T_u}{b}$

$$= 27.57\ KN$$

b) Maximum permissible shear stress in concrete:

$$\tau_{c,max} = 3.5 \frac{N}{mm^2}$$

c) Shear stress, $\tau_{ue} = \frac{V_e}{bd} = \frac{27.57 \times 10^3}{400 \times 570} = 0.121 \frac{N}{mm^2}$

According to clause 6.3.4, IS 13920:2016, during the calculation of design shear force, concrete in the RC section is not considered.

Required capacity of shear reinforcement ($V_{us)} = 27.57 \, KN$

Provide 2-legged 8mmØ stirrups.

A_{sv} for 2-legged 8mmØ stirrups: $A_{sv} = 157.08 \, mm^2$

Spacing of stirrups: $\qquad S_v = \dfrac{0.87 \, f_y A_{sv} d}{V_{us}}$

$= 750.41 \, mm$

Maximum spacing, [IS 456:2000, Cl. 26.5.1.6]

i) $\quad S_{v,max} = \dfrac{0.87 \, f_y A_{sv}}{0.4b} = 226.85 \, mm$

ii) $\quad S_{v,max} = 0.45 \times d = 427.5 \, mm$

iii) $\quad S_{v,max} = 300 \, mm$

Maximum spacing, [IS 13920:2016, Cl. 6.3.5]

i) $\quad \dfrac{\emptyset}{4} = \dfrac{570}{4} = 142.5 \, mm$

ii) $\quad 8 \times \emptyset = 8 \times 20 = 160 \, mm$

iii) $\quad 100 \, mm \, over \, a \, length \, of \, 2d$

$\qquad$ Over a length of $2 \times d = 2 \times 570 = 1140 \, mm$

Hence, a maximum spacing of 100mm is provided for 2 legged 8mmØ stirrups over the length of 1140 mm.

For end j:

$(V_{ue)} = 55.38 \, KN$

Required capacity of shear reinforcement $(V_{us)} = 55.38 \, KN$

Provide 2-legged 8mmØ stirrups.

A_{sv} for 2-legged 8mmØ stirrups: $A_{sv} = 100.53 \, mm^2$

Spacing of stirrups: $\qquad S_v = \dfrac{0.87 \, f_y \, A_{sv} \, d}{V_{us}}$

$= 373.58 \, mm$

Maximum spacing, [IS 456:2000, Cl. 26.5.1.6]

   i) $\qquad S_{v, max} = \dfrac{0.87 \, f_y \, A_{sv}}{0.4b} = 226.85 \, mm$

   ii) $\qquad S_{v, max} = 0.45 \times d = 427.5 \, mm$

   iii) $\qquad S_{v, max} = 300 \, mm$

Maximum spacing, [IS 13920:2016, Cl. 6.3.5]

   i) $\qquad \dfrac{\emptyset}{4} = \dfrac{570}{4} = 142.5 \, mm$

   ii) $\qquad 8 \times \emptyset = 8 \times 20 = 160 \, mm$

   iii) $\qquad 100 \, mm \, over \, a \, length \, of \, 2d$

           $Over \, a \, length \, of \, 2 \times d = 2 \times 570 = 1140 \, mm$

Hence, a maximum spacing of 100mm is provided for 2-legged 8mmØ stirrups over the length of 1140 mm.

For mid-span:

$(V_{ue)} = 63.34 \, KN$

$(T_{ue)} = 0\ KNm$

Required capacity of shear reinforcement $(V_{us)} = 63.34\ KN$

Provide 2-legged 8mmØ stirrups.

A_{sv} for 2-legged 8mmØ stirrups: $A_{sv} = 100.53\ mm^2$

Spacing of stirrups: $\qquad S_v = \dfrac{0.87\ f_y\ A_{sv}\ d}{V_{us}}$

$= 326.63\ mm$

Maximum spacing, [IS 456:2000, Cl. 26.5.1.6]

i) $\quad S_{v,max} = \dfrac{0.87\ f_y\ A_{sv}}{0.4b} = 226.85\ mm$

ii) $\quad S_{v,max} = 0.45 \times d = 427.5\ mm$

iii) $\quad S_{v,max} = 300\ mm$

Maximum spacing, [IS 13920:2016, Cl. 6.3.5]

$S_v \le \frac{d}{2} = 285\ mm$

Hence, provide 225 mm spacing over the mid-span with 2 legged 8mmØ stirrups.

b) *For B32:*

$V_{ui} = 169.24\ KN$

$V_{umid} = 141.49\ KN$

$V_{uj} = 16.41\ KN$

Shear force due to formation of plastic hinge:

Grid	$M_s(KNm)$	$M_h(KNm)$	$V_g(KNm)$
I	148.63	148.63	150.83
J	148.63	148.63	16.41

For sway to right:

$$V_{ui} = V_{gi} = 1.4\left[\frac{M_{uis}+M_{uju}}{L}\right]$$

$$V_{uj} = V_{gj} = 1.4\left[\frac{M_{uis}+M_{uju}}{L}\right]$$

For sway to left:

$$V_{ui} = V_{gi} = 1.4\left[\frac{M_{uis}+M_{uju}}{L}\right]$$

$$V_{uj} = V_{gj} = 1.4\left[\frac{M_{uis}+M_{uju}}{L}\right]$$

Computed V_{ui}

Right sway $=$ 150.72 KN

Left sway $=$ 16.52 KN

Computed V_{uj}

Right sway $=$ 150.94 KN

Left sway $=$ 16.30 KN

Maximum design shear force at end i $=$ 169.24 KN

Maximum design shear force at mid $=$ 141.79 KN

Maximum design shear force at end j $=$ 16.52 KN

Torsional moment at end j (T_{ui}) $=$ 0 KNm

Torsional moment at end j (T_{uj}) = 0 KNm

For end i:

a) Equivalent factored shear force: V_e = V_{dui} + 1.6 $\frac{T_u}{b}$

$$= 169.24 \ KN$$

b) According to clause 6.3.4, IS 13920:2016, during the calculation of design shear force, concrete in the RC section is not considered.

c) Required capacity of shear reinforcement $(V_{us)}$ = 169.24 KN

Provide 2-legged 10mmØ stirrups.

A_{sv} for 2-legged 10mmØ stirrups: A_{sv} = 157.08 mm^2

Spacing of stirrups: $\qquad S_v = \frac{0.87 \ f_y \ A_{sv} \ d}{V_{us}}$

= 191.01 mm

Maximum spacing, [IS 456:2000, Cl. 26.5.1.6]

i) $\qquad S_{v,max} = \frac{0.87 \ f_y \ A_{sv}}{0.4b} = 354.46 \ mm$

ii) $\qquad S_{v,max} = 0.45 \times d = 427.5 \ mm$

iii) $\qquad S_{v,max} = 300 \ mm$

Maximum spacing, [IS 13920:2016, Cl. 6.3.5]

i) $\qquad \frac{\text{Ø}}{4} = \frac{570}{4} = 142.5 \ mm$

ii) $\qquad 8 \times \text{Ø} = 8 \times 20 = 160 \ mm$

iii) $\qquad 100 \ mm \ over \ a \ length \ of \ 2d$

$\qquad\qquad$ Over a length of 2 $\times d$ = 2 $\times 570$ = 1140 mm

---

Hence, a maximum spacing of 100mm is provided for 2 legged $10mm\emptyset$ stirrups over the length of 1140 mm.

For end j:

$(V_{ue)} = 16.52\ KN$

Required capacity of shear reinforcement $(V_{us)} = 16.52\ KN$

Provide 2-legged $8mm\emptyset$ stirrups.

A_{sv} for 2-legged $8mm\emptyset$ stirrups: $A_{sv} = 100.53\ mm^2$

Spacing of stirrups: $\qquad S_v = \dfrac{0.87\ f_y\ A_{sv}\ d}{V_{us}}$

$= 1252.36\ mm$

Maximum spacing, [IS 456:2000, Cl. 26.5.1.6]

    i) $S_{v,\,max} = \dfrac{0.87\ f_y\ A_{sv}}{0.4b} = 226.85\ mm$

    ii) $S_{v,\,max} = 0.45 \times d = 427.5\ mm$

    iii) $S_{v,\,max} = 300\ mm$

Maximum spacing, [IS 13920:2016, Cl. 6.3.5]

    i) $\dfrac{\emptyset}{4} = \dfrac{570}{4} = 142.5\ mm$

    ii) $8 \times \emptyset = 8 \times 20 = 160\ mm$

    iii) $100\ mm\ over\ a\ length\ of\ 2d$

        Over a length of $2 \times d = 2 \times 570 = 1140\ mm$

Hence, a maximum spacing of 100mm is provided for 2-legged $8mm\emptyset$ stirrups over the length of 1140 mm.

For mid-span:

$(V_{ue}) = 141.49 \ KN$

$(T_{ue}) = 0 \ KNm$

Required capacity of shear reinforcement $(V_{us}) = 141.49 \ KN$

Provide 2-legged 10$mm\emptyset$ stirrups.

A_{sv} for 2-legged 10$mm\emptyset$ stirrups: $A_{sv} = 157.07 \ mm^2$

Spacing of stirrups: $\qquad S_v = \dfrac{0.87 \ f_y \ A_{sv} \ d}{V_{us}}$

$= 227.98 \ mm$

Maximum spacing, [IS 456:2000, Cl. 26.5.1.6]

   i) $S_{v,max} = \dfrac{0.87 \ f_y \ A_{sv}}{0.4b} = 354.46 \ mm$

   ii) $S_{v,max} = 0.45 \times d = 427.5 \ mm$

   iii) $S_{v,max} = 300 \ mm$

Maximum spacing, [IS 13920:2016, Cl. 6.3.5]

   i) $\dfrac{\emptyset}{4} = \dfrac{570}{4} = 142.5 \ mm$

   ii) $8 \times \emptyset = 8 \times 20 = 160 \ mm$

   iii) $100 \ mm \ over \ a \ length \ of \ 2d$

             Over a length of $2 \times d = 2 \times 570 = 1140 \ mm$

Hence, a maximum spacing of 100mm is provided for 2-legged 10$mm\emptyset$ stirrups over the length of 1140 mm.

9. Design of side Reinforcement

Since, $d < 750\ mm$ side face reinforcement is not needed.

10. Check for deflection

[IS456 : 2000 Cl. 23.2.1]

a) *For Beam B31:*

Clear span $=\ 2787.5\ mm$

Width of support $=\ 462.5\ mm$

1/12 of clear span $=\ 32.29\ mm$

Width of support $>\ \frac{1}{12}$ of clear span so L_x is taken as clear span.

Effective length $(L_x) = 3250$ mm

$\propto\ =\ 20$ (For simply supported)

$\beta\ =\ 1$ (Since span is less than 10m)

For λ,

$$F_s =\ 0.58\ f_y\ \frac{Area\ of\ steel\ req}{Area\ of\ steel\ provided}$$

$$=\ 0.58 \times 500 \times \frac{599.53}{628.32}$$

$$=\ 276.71\ N/mm^2$$

Therefore, % Ast = 0.26%

$\lambda = 1.4$

$\gamma\ =\ 1.08$

$\delta\ =\ 1$

So, $\propto \beta\,\gamma\,\delta\,\lambda\ =\ 20 \times 1 \times 1.4 \times 1.08\ =\ 30.24$

$$\frac{Lx}{d}\ =\ \frac{3250}{570}\ =\ 5.7\ <\ 30.24\ \textbf{(OK)}$$

b) **For Beam B32:**

Clear span $=$ 3797. 3 mm

Width of support $=$ 462. 5 mm

1/12 of clear span $=$ 316. 48 mm

Width of support $>$ $\frac{1}{12}$ of clear span so L_x is taken as clear span.

Effective length (L_x) = 4260.3 mm

$\propto$ $=$ 20 (For simply supported)

β $=$ 1 (Since span is less than 10m)

For λ,

$$F_s = 0.58 f_y \frac{Area\ of\ steel\ req}{Area\ of\ steel\ provided}$$

$$= 0.58 \times 500 \times \frac{902.77}{942.48}$$

$$= 277.78\ N/mm^2$$

Therefore, % Ast = 0.39%

λ = 1.2

γ $=$ 1. 08

δ $=$ 1

So, $\propto \beta \gamma \delta \lambda$ $=$ 20×1×1. 2×1. 08 $=$ 25. 92

$$\frac{Lx}{d} = \frac{4260.3}{570} = 7.47 < 25.92 \textbf{(OK)}$$

11. Check for development length:

IS456-2000: Cl. 26.2.2

$$L_d = \frac{\varphi \rho_s}{4\tau_{bd}}$$

$$= \frac{25 \times 0.87 \times 500}{4 \times 1.5 \times 1.6} = 906.25 \, mm \text{ (For tension)}$$

$$= \frac{25 \times 0.87 \times 500}{4 \times 1.5 \times 1.25} = 1160 \, mm \text{ (For compression)}$$

Also,

$$L_d \leq 1.3 \frac{M}{V} + L_o$$

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

$$M = 142.12 \, KNm$$
$$V = 63.34 \, KN$$
$$Lo = \frac{462.5}{2} - 20 = 211.25 \, mm$$
$$1.3 \frac{M}{V} + Lo = 3128.14 \, mm > L_d \text{ (OK)}$$

7.4 CURVED BEAM DESIGN

Beam ID $= B21$

Unique Name $= 319$

Floor $= 6th$

Concrete Grade $= M30$

Steel Grade $= Fe\ 500\ (Longitudinal)$

$\qquad = Fe\ 415\ (Transverse)$
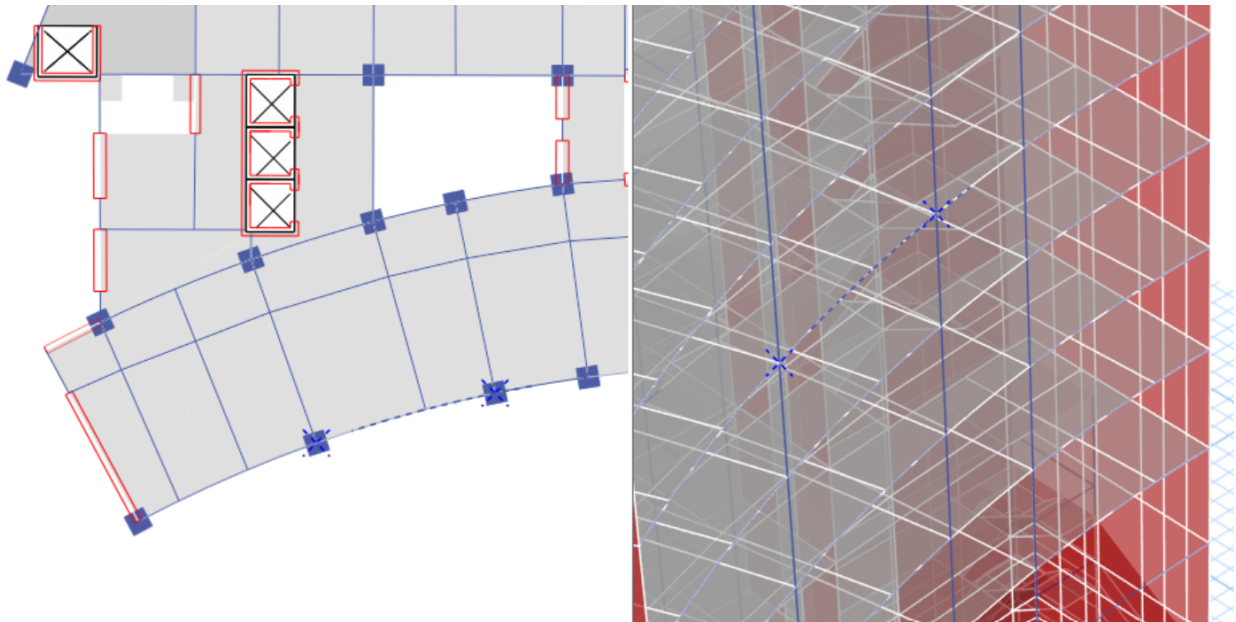
Fig: 7.5 Curved Beam

Design of longitudinal bars for moment

1. Known data

    a. Overall Depth, D $= 700\ mm$

    b. Width of Beam, B $= 525mm$

    c. Nominal Cover $= 25\ mm$

    d. ø $= 25\ mm$

Effective Depth, d $= 700 - 25 - \left(\frac{25}{2}\right)$

$$= 662.5 \ mm$$

$Angle \ Subtended = 12°$

Length= 7369.3mm

2. Check for axial stress

Axial force from ETABS = 22.75KN

Axial stress $= \ 22.75 \times \dfrac{10^3}{525 \times 700} \ = 0.069 N/ \ mm^2$

Permissible axial stress =0.08 x fck=0.08x30=2.4N/mm^2 > 2.4N/mm^2 **(OK)**

3. Check for member size

$\dfrac{b}{D} \ = \ 525 \ / \ 700 \ = \ 0.75 > 0.3$ ok

Clear length, $l \ = \ 7369.3 - \dfrac{850}{2} - \dfrac{850}{2} = \ 6519.3 \ mm$

$L/D \ = \ \dfrac{6519.3}{700} = \ 9.31 \ > 4$ **(OK)**

4. Minimum Area Required $(A_{st,min})$

$A_{st,min} = \dfrac{0,85}{f_y} \ b \times d \ = \dfrac{0.85}{500} \times 525 \times 662.5 \ = \ 591.28 \ mm^2$

$A_{st,min} = \ \dfrac{0.24 \times \sqrt{f_{ck}}}{f_y} \times b \times d \ = \ 914.42 \ mm^2$

Adopt maximum $A_{st,min} \ = \ 914.42 mm^2$

Maximum area required,

a. $A_{st,max} \ = \ 0.04 \times b \times d$

$$= \ 0.04 \times 525 \times 662.5$$

$$= \ 13912.5 \ mm^2$$

[Annex G]

5. Limiting Moment $\left(M_{u,lim}\right) = \ 0.133 \ bd^2 f_{ck}$

$$= \ 0.133 \times 30 \times 525 \times 662.5\text{^}2$$

$\therefore M_{u,lim} \ = \ 919.4 \ KNm$

Moment Diagram (kN-m)

(-)Moment	-428.7853	-82.3474	0
(+)Moment	0	126.6141	285.2135

Shear Diagram (kN)

Shear	225.7376	226.4029	246.2459

Torsion Diagram (kN-m)

Torsion	4.4126	1.9509	2.739

Fig 7.6: Moment, Shear and Torsion diagram for Curved Beam

5. Position: I Left end:

   Governing envelope combination $=$ *envelope*

   Factored moment, $M_u = 428.53 KNm$

   Torsional moment, $T_u = 4.41 KNm$

   Moment due to torsion $\left(M_t\right) = T_u \dfrac{\left(1+\frac{D}{b}\right)}{1.7}$

$$= 6.053 KNm$$

   Required design bending moment (M)

$$M = M_u + M_t = 434.6 \ KNm$$

   Since, M<$M_{u,lim}$ design as singly reinforced section

   design as singly reinforced section

$$M = 0.87 f_y \times A_{st} \times d\left(1 - \frac{f_y A_{st}}{f_{ck} bd}\right)$$

   $\left(A_{st}\right)_{left} = 1636.36 mm\char`^2 \ (Bottom) > Ast, min$ **(OK)**

   $\% \ tension \ of \ steel = 0.56\%$

   Area of compression steel $\left(A_{sc}\right) = 0$ **(OK)**

   So, $\left(A_{sc}\right)_{left} = 0$

   And $\% \ of \ compression \ steel = 0.56\%$

6. Position: II Mid-Span:

   Governing envelope combination $=$ *envelope*

   Factored moment, $M_u = 136.91 \ KNm$

   Torsional moment, $T_u = 1.95 KNm$

   Moment due to torsion $\left(M_t\right) = T_u \dfrac{\left(1+\frac{D}{b}\right)}{1.7}$

$$= 2.67 KNm$$

Required design bending moment (M)

$$M = M_u + M_t = 139.58 KNm$$

Since, M<$M_{u,lim}$ design as singly reinforced section

design as singly reinforced section

$$M = 0.87 f_y \times A_{st} \times d \left(1 - \frac{f_y A_{st}}{f_{ck} bd}\right)$$

$$(A_{st})_{mid} = 496.132 \ mm^2 < A_{st,min} \quad \text{Provide 914.42mm}^2$$

$$\% \ tension \ of \ steel = 0.28\%$$

7. Position: III Right end:

Governing envelope combination = *envelope*

Factored moment, $M_u = 284.93 \ KNm$

Torsional moment, $T_u = 2.735 \ KNm$

Moment due to torsion $(M_t) = T_u \frac{\left(1 + \frac{D}{b}\right)}{1.7}$

$$= 3.754 \ KNm$$

Required design bending moment (M)

$$M = M_u + M_t = 288.68 KNm$$

Since, M<$M_{u,lim}$ design as singly reinforced section

$$M = 0.87 f_y \times A_{st} \times d \left(1 - \frac{f_y A_{st}}{f_{ck} bd}\right)$$

$$(A_{st})_{right} = 1055.05 \ mm^2 > A_{st,min}$$

$\% \ tension \ of \ steel \ = \ 0.42\%$

8. Longitudinal Detailing

Position	Manually calculated area of steels (mm^2)	Bars	Area provided (mm^2)	% Provided
Left (Top)	1636.36	$4 - 25\emptyset$	1963.49	0.56
Left (Bottom)		$4 - 25\emptyset$	1963.49	0.56
Mid (Top)	914.42	$2 - 25\emptyset$	981.75	0.28
Mid (Bottom)		$2 - 25\emptyset$	981.75	0.28
Right (Top)	1055.05	$3 - 25\emptyset$	1472.62	0.42
Right (Bottom)		$3 - 25\emptyset$	1472.62	0.42

9. Design for shear

   a. From ETABS:-

   At left end: V_u = 225.62 KN

   At mid-span: V_u = 246.112 KN

   At right end: V_u = 226.27 KN

   b. Shear force due to formation of plastic hinge at both ends of the beam plus factored load on the span.

   For sway to right:

   $$v_{u,a} = v_{u,a}^{D+L} - 1.4 \frac{M_u^{As}+M_u^{Bh}}{L_{AB}}$$

$$v_{u,b} = v_{u,b}^{D+L} + 1.4 \frac{M_u^{As}+M_u^{Bh}}{L_{AB}}$$

For sway to left:

$$v_{u,a} = v_{u,a}^{D+L} - 1.4 \frac{M_u^{Ah}+M_u^{Bs}}{L_{AB}}$$

$$v_{u,b} = v_{u,b}^{D+L} + 1.4 \frac{M_u^{Ah}+M_u^{Bs}}{L_{AB}}$$

$$M_u = 0.87 \, fy \, Ast \, d \left(1 - \frac{fy \, Ast}{fck \, bd}\right)$$

$$M_u^{As} = 512.614 \, KNm < M_{u,lim}$$

$$M_u^{Bs} = 394.443 \, KNm < M_{u,lim}$$

$$M_u^{Ah} = 512.614 \, KNm < M_{u,lim}$$

$$M_u^{Bh} = 394.443 \, KNm < M_{u,lim}$$

From analysis,

$$v_{u,a}^{D+L} = 143.57 \, KN$$

$$v_{u,b}^{D+L} = 168.112 \, KN$$

For sway to right:

$$v_{u,a} = -51.218 \, KN$$

$$v_{u,b} = 168.31 \, KN$$

For sway to left:

$$v_{u,a} = -51.218 \, KN$$

$$v_{u,b} = 168.31 \, KN$$

Taking maximum obtained from analysis and formation of plastic hinges.

Design forces at left end, $(V_u)_{left} = 225.62\ KN$

Design forces at right end, $(V_u)_{right} = 246.112\ KN$

Design forces at mid span, $(V_u)_{mid} = 226.27\ KN$

10. Design of shear reinforcement

Providing 2-legged 10 mmm phi stirrups,

Maximum spacing = 0.75d = 0.75 * 767.5 = 567.645mm or 300m (whichever is smaller)

Spacing of reinforcements;

a. At left end:

$$V_u = 225.62\ KN$$

$$T_u = 4.41\ KNm$$

$$V_e = 225.62 + 1.6\ \frac{T_u}{b}$$

$$= 225.633\ KN$$

Maximum permissible shear stress in concrete: $\tau_{c,max} = 3.5\frac{N}{mm^2}$

Shear stress, $\tau_{ue} = \frac{V_e}{bd} = \frac{225.633\times10^3}{525\times662.5} = 0.648\frac{N}{mm^2}$

Permissible shear strength of concrete:

$\tau_c = 0.52\frac{N}{mm^2} < \tau_{c,max} < \tau_{ue}$ **(OK)**

Required capacity of shear reinforcement:

$$V_{us} = V_e - \tau_c \times b \times d$$

$$= 225.633 - (0.52{\times}525{\times}662.5 \times 10^{-3})$$

$$= 44.773\ KN$$

Provide 2-legged $10mm\emptyset$ stirrups.

$$A_{sv} = 157.08\ mm^2$$

Spacing of stirrups:
$$S_v = \frac{0.87\,f_y\,A_{sv}\,d}{V_{us}}$$

$$= 839.185mm$$

Minimum spacing:
$$S_{v,min} = \frac{0.87\,f_y\,A_{sv}\,d}{0.4b}$$

$$= 270\ mm$$

Maximum spacing:
$$= 0.75\,d = 496.875\ mm\ or\ 300mm$$

For spacing of links [IS 13920:2016]

So, over the distance of: $\quad 2 \times d = 1325\ mm$

$$S_v \le \frac{d}{4} = 331.25\ mm$$

$$S_v \le \frac{d}{4} = 200\ mm$$

$$\therefore S_v \le 100\ mm$$

Design spacing is less than the required minimum spacing as per ductile detailing. Hence, Provide 2- legged 10mm stirrups at a spacing of 100 mm over the distance of 1325 mm.

b. At right end:

$$V_e = 246.12 KN$$

Maximum permissible shear stress in concrete: $\tau_{c,max} = 3.5 \frac{N}{mm^2}$

Shear stress, $\tau_{ue} = \frac{V_e}{bd} = 0.707 \frac{N}{mm^2}$

Permissible shear strength of concrete:

$$\tau_c = 0.46 \frac{N}{mm^2} < \tau_{c,max} < \tau_{ue} \textbf{ (OK)}$$

Required capacity of shear reinforcement:

$$V_{us} = V_e - \tau_c \times b \times d$$

$$= 246.12 - (0.46 \times 525 \times 662.5 \times 10^{-3})$$

$$= 86.12 \ KN$$

Provide 2-legged $10 mm \emptyset$ stirrups.

$$A_{sv} = 157.08 \ mm^2$$

Spacing of stirrups: $\quad\quad S_v = \frac{0.87 f_y A_{sv} d}{V_{us}}$

$$= 436.28 \ mm$$

Minimum spacing: $\quad\quad S_{v,min} = \frac{0.87 f_y A_{sv} d}{0.4b}$

$$= 270 \ mm$$

Maximum spacing: $\quad\quad = 0.75 d = 496.875 mm \ or \ 300 mm$

Hence, Provide 2- legged 10mm stirrups at a spacing of 100 mm c/c

c. At mid span:

$$V_e = 226.276 KN$$

Maximum permissible shear stress in concrete: $\tau_{c,max} = 3.5 \dfrac{N}{mm^2}$

Shear stress, $\tau_{ue} = \dfrac{V_e}{bd} = 0.651 \dfrac{N}{mm^2}$

Permissible shear strength of concrete:

$$\tau_c = 0.38 \dfrac{N}{mm^2} < \tau_{c,max} < \tau_{ue} \textbf{(OK)}$$

Required capacity of shear reinforcement:

$$V_{us} = V_e - \tau_c \times b \times d$$

$$= 226.276 - (0.38 \times 525 \times 662.5 \times 10^{-3})$$

$$= 94.107\ KN$$

Provide 2-legged $10mm\emptyset$ stirrups.

$$A_{sv} = 157.08\ mm^2$$

Spacing of stirrups: $\qquad S_v = \dfrac{0.87 f_y A_{sv} d}{V_{us}}$

$$= 400\ mm$$

Minimum spacing: $\qquad S_{v,min} = \dfrac{0.87 f_y A_{sv} d}{0.4b}$

$$= 270\ mm$$

Maximum spacing: $\qquad = 0.75\ d = 496.815\ mm\ or\ 300mm$

Hence, Provide 2- legged 10mm stirrups at a spacing of 100 mm c/c.

11. Design of side Reinforcement

Since, d <750 mm no need.

12. Check for deflection

Clear span $=$ 6519. 3mm

Width of support $=$ 850 mm

1/12 of clear span $=$ 543 mm

Width of support $>$ $\frac{1}{12}$of clear span so L$_x$ is taken as clear span:

L_x $=$ 6519. 3 mm

$\propto$ $=$ 26 (For continuous)

β $=$ 1 (span less than 10m)

For λ,

$$F_s = 0.58 f_y \frac{Area\ of\ steel\ req}{Area\ of\ steel\ provided}$$

$$= 0.58 \times 500 \times \frac{914.42}{981.75}$$

$$= 270.11 N/mm^2$$

Therefore, % Ast = 0.28%

λ = 1.3

γ $=$ 1.08

δ $=$ 1

So, $\propto$ β γ δ λ $=$ 36.5

$$\frac{Lx}{d} = \frac{6519.3}{662.5} = 9.84 < 36.5 \textbf{ (OK)}$$

---

13. Check for development length:

$$L_d = \frac{\emptyset \rho_s}{4\tau_{bd}}$$

$$= \frac{25 \times 0.87 \times 500}{4 \times 1.5 \times 1.6} = 1132.81 \, mm \text{ (For tension)}$$

$$= \frac{25 \times 0.87 \times 500}{4 \times 1.5 \times 1.6 \times 1.25} = 906.25 \, mm \text{ (For compression)}$$

Also,

$$L_d \leq 1.3 \frac{M}{V} + L_o$$

$$M = 0.87 f_y A_{st,prov} \left(d - \frac{f_y A_{st,prov}}{f_{ck} * bd} \right)$$

$$= 512.61 KNm$$

$$V = 225.62 \, KN$$

$$Lo = \frac{850}{2} - 25 = 400 \, mm$$

$$1.3 \frac{M}{V} + Lo = 3353.6 \, mm > L_d \, \textbf{(OK)}$$

8. DESIGN OF SLAB

A slab is a flat, horizontal structural element used to create floors, roofs, and ceilings in buildings. It is typically made of reinforced concrete or a combination of concrete and steel reinforcement. Slabs are an important component of the structural system of a building, providing a platform for all goods and people while also supporting the weight of the building and the people and objects inside it. Slabs can be supported by beams or walls, and can be designed to span large distances without the need for intermediate supports. They can also be designed to resist lateral forces such as wind or earthquakes.

Proper design and construction of slabs is essential to ensuring the safety and stability of a building. Slabs must be designed to meet building codes and standards and constructed with quality materials and workmanship to ensure their durability and longevity.

There are several types of slabs used in building construction:

RCC Slab

Edge Supported Slab   Flat slab

One way Slab   Two way slab
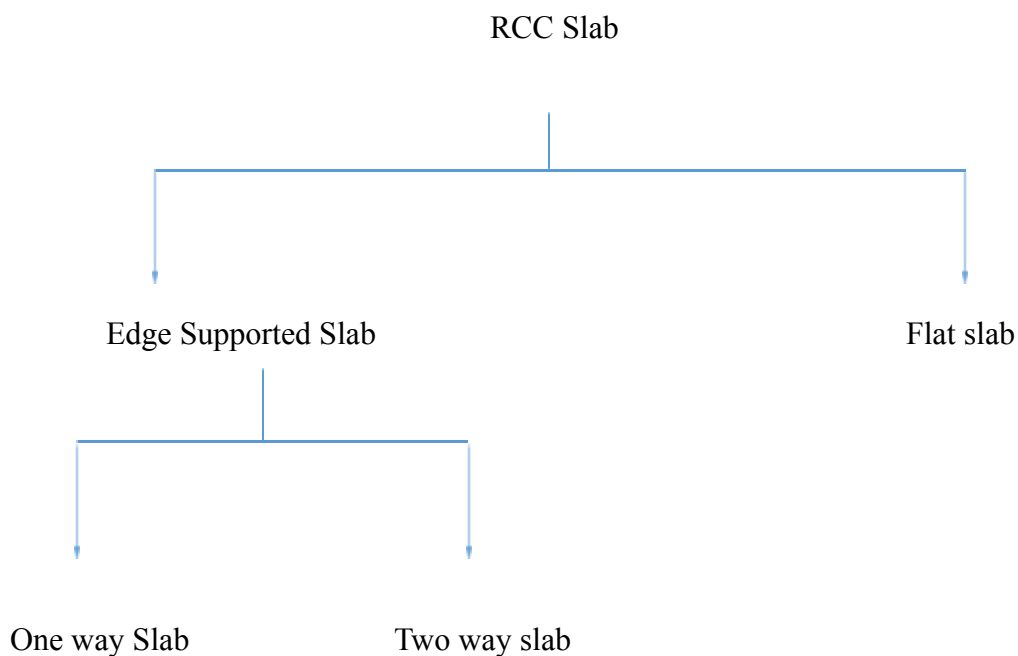
The type of slab used will depend on the specific requirements of the building, including the load it needs to support, the span of the slab, and the architectural design of the building. Edge supported slab can also be classified as simply supported, restrained and cantilever based on support conditions.

Slabs are usually subjected to **uniformly distributed loads, line loads, concentrated loads and partially distributed loads in transverse direction.** To design a slab using principles of beam design, the slab is divided into narrow strips each of which behave as individual beams. This design philosophy is known as **strip based design**. Despite its simplicity, strip based design is merely an approximation and fails to consider the global effects of neighboring members on the strip.

Steps for Slab Design:

1. Determination of dimensions of slabs.
2. Basic Assumptions regarding support conditions, steel grade, concrete grade.
3. Preliminary design for slab thickness.
4. Determination of dead and live loads on the slab.
5. Calculation of critical bending moment and shear force using IS code.
6. Check for the Single Under Reinforced section.
7. Calculation of required rebars for corresponding bending moments for design strip
8. Check for shear, deflection and development length criteria
9. Revision for slab thickness and reinforcement if criteria not satisfied
10. Provision of torsional reinforcement and special detailing (eg. around voids) if necessary

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

In **one-way slabs**, the slab deflects in shorter direction alone in a **cylindrical fashion**, hence main **reinforcement is provided in the shorter direction**. It is because a substantial bending moment develops in 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 a two-way slab, which when loaded will **deflect into a dished surface**.

8.1 Detail Design 1:

Slab no: Storey 9 floor, 3rd Level F37

Unique name: 1960

Step 1: Dimension of Slab:

Dimensions: 4.785 x 5.015m

L_x=4.785m, L_y=5.015m

Depth of Slab D = 130 mm

Diameter of Bars φ = 10mm

Clear Cover = 15 mm

Effective Depth (d) = 110mm

One side discontinuous slab

M30, Fe500

Span to depth ratio=l/d=36.8

Effective depth in short span, d_x=130-10/2-15=110mm   **[Table 16, IS 456, 2000]**

Effective depth in long span, d_y=130-10-10/2-15=100mm

Effective length,

In short span, l_{xeff} =4.785+0.11=4.895mm or 4.785+0.23=5.015m i.e. 4.895m

                                         [Cl 23.2.1,IS 456,2000]

In long span l_{yeff}=5.015+0.1=5.115m or 5.015+0.23=5.245m
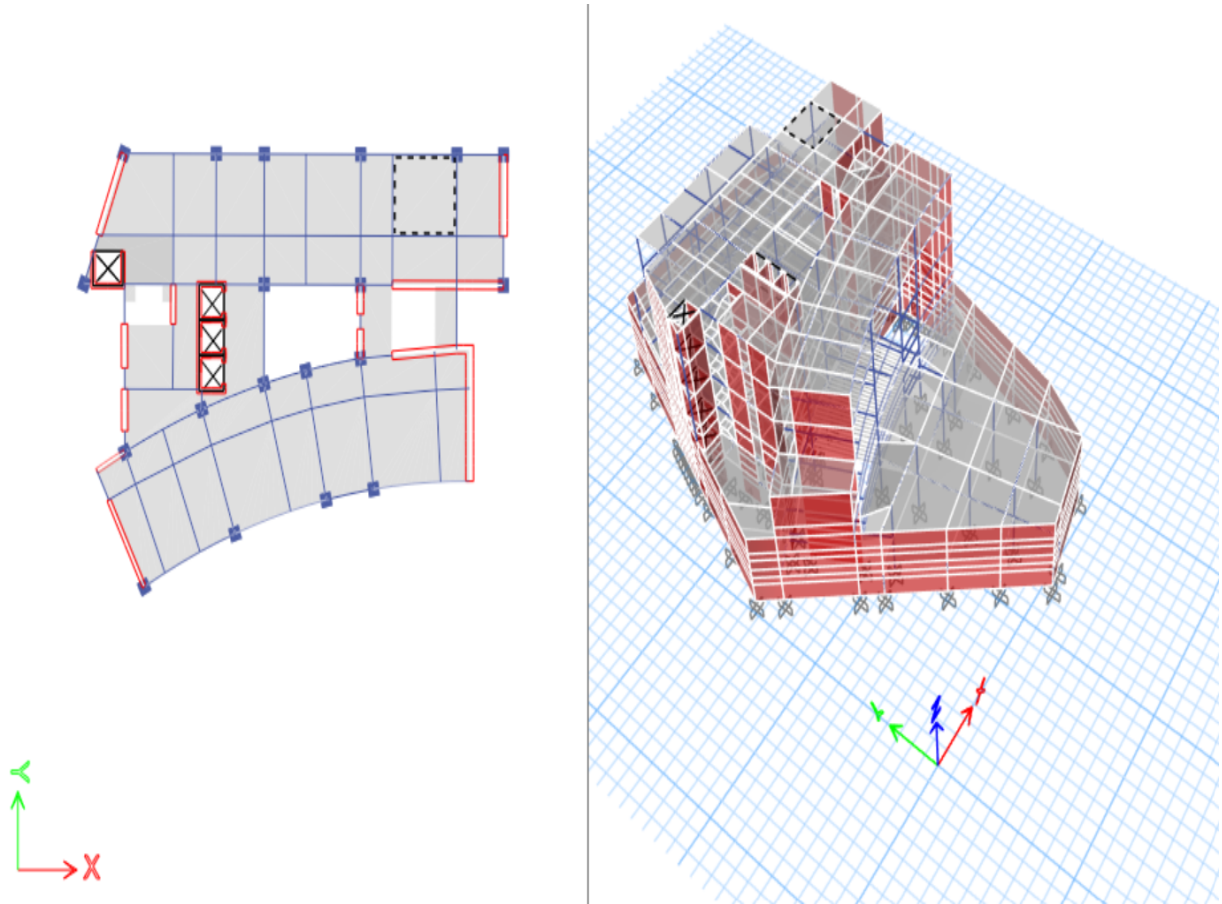
Fig: 8.1 Slab Design

Step 2: Load Calculation

Imposed load= (2.5+1) =3.5KN/m^2

Floor finish =1.15KN/m^2

Self wt. of slab= 0.13 x 25=3.25KN/m^2

Total Factored load=11.85KN/m^2

Step 3: Moment and Shear Calculation

$(L_y/L_x)_{eff}$ = 1.044<2

i.e. So, two way slab

-ve moment co.eff at continuous edge	-ve moment at continuous edge per unit width.	+ve moment co.eff at midspan	+ve moment at midspan
α_x=0.04	M_x=11.36KNm	α_x=0.03	M_x=8.52KNm
α_y=0.037	M_y=10.51KNm	α_y=0.028	M_y=7.95KNm

For shear,

V_y=wL_y/3=20.2KN

$$V_x = wL_y(2 - \frac{Ly}{Lx})/4 = 14.5KN$$

Step 4: Tension Steel Calculation: [Annex G,]

$M_{u,lim} = 0.36 * X_{u,lim}$ /d (1-0.42 *$X_{u,max}$/d)*b*d^2f_{ck}=0.133*$f_{ck}bd^2$

i.e. d=53.35mm<100mm (ok)

To get A_{st},

$$M_u = 0.87f_yA_{st}(d - \frac{A_{st}f_y}{bf_{ck}})$$

For –ve reinforcement at continuous edge edge (short span)

M_x^-=11.36KNm

Ast_x^-=246.62mm^2

Rebar spacing=1000*π*10^2/4*246.62=318.46mm

$A_{st(min)}$=0.12%Bd=156mm^2

Provide 10mmϕ @ 300mm c/c

Ast_x^- provided= πx10^2/(4x300) x 1000=261.8mm^2>A_{stmin} ok

[at top face of continuous edge in short span direction]

For M_y^-=10.51KNm

Ast_y^- =252.21mm^2

Rebar spacing= $(1000*10^2*\pi/4)/252.21$=311.41mm

Provide 10mmϕ at 300mm c/c

A_{sty}^- provided =261.8mm^2

[In long span at top face of continuous edge]

For M_x^+=8.52KNm,

A_{stx}^+=183.13mm^2

Rebar spacing = 428.875mm

A_{stx}^+ provided =261.8mm^2

[short span at bottom face of mid span]

For M_y^+ =7.95KNm,

A_{sty}^+=188.7mm^2

Rebar spacing=416.22mm

Ast provided =261.8mm^2

[longer span at bottom of mid span]

Step 5: Check for Shear:

K=1.3

Along short span, % of steel=0.238

So, τ_c= 0.3604N/mm^2 [Table 19]

Shear Capacity=k $\tau_c$bd=51.54KN>20.2KN (Ok)

Along long span, % of steel=0.238

So, τ_c= 0.3604N/mm^2 [Table 19]

Shear Capacity=k $\tau_c$bd=51.54KN>14.5KN (Ok)

Step 6: Check for Deflection:

Span <10m   [Cl.23.2, IS 456: 2000]

So, α= 26, β=1.

Let % of steel= 0.2

F_y=500 N/mm^2

F_s=0.58*500*183.13/261.8=202.8N/mm^2

From graph for modification factor vs % tension reinforcement graph

γ=1.98

λ=1; (no compression reinforcement provided)

δ=1; (other than flanged beam)

$\gamma*\delta*\lambda*\alpha*\beta$= 51.48>51.15 (ok)

Hence, slab is safe in deflection.

Step 7: Check for Development Length:

Development length, Ld=$\phi*\sigma_s/4\tau_{bd}$=447.92mm   [Cl26.2.1, IS 456,2000]

L_d should not exceed 1.3M_1/V+L_0 =854.21mm

Embedded length available at support assuming support width of 230mm=230-nominal cover=200mm >L_d/3

So, development length is adequate.

8.2 Detail Design 2:

Slab no: Storey 9 floor 3rd ,Label 6

Unique name: 1215

Step 1: Dimension of Slab:

Dimensions: 2.6x5.38m

L_x=2.6m, L_y=5.38m

Depth of Slab D = 130 mm

Diameter of Bars φ = 10mm

Clear Cover = 15 mm

Effective Depth (d) = 110mm

M30, Fe500

Effective length,

l_{xeff} =5.38+0.11=5.49mm or 5.38+0.23=5.61m i.e. 5.49m

[Cl 23.2.1,IS 456,2000]

l_{yeff}=2.6+0.11=2.71m

l_x/l_y=2.025 i.e. <u>One way slab</u>

Step 2: Load Calculation

Imposed load= 3.5KN/m^2

Floor finish =1.15KN/m^2

Self wt. of slab= 0.13x25=3.25KN/m^2

Total Factored load=11.85KN/m^2

Step 3: Moment and Shear Calculation

Factored moment= $(11.85*2.71^2)/8$=10.88 KNm

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

d= 52.22mm<110mm ok

Step 4: Tension Steel Calculation:

Mu=$0.87*f_y A_{st}$(d-0.416x)   [Annex G,]

Ast=235.8mm^2

Rebar Spacing,

S= $1000 \text{x} \frac{\frac{\pi \times 10^2}{4}}{235.8} = 333.1 mm$

Provide 10mm dia @250mm c/c

Ast, provided = 314.16mm^2

For Distribution Bar,

Ast dist=0.12% bD= 156mm^2

Lets provide 8mm dia bars

Spacing = $1000 \text{x} \frac{\frac{\pi \times 8^2}{4}}{156} = 332 mm$

Provide 8mm dia bars at 330mm c/c

Step 5: Check for Shear:

Factored Shear V_u= 11.85*2.71/2= 16.06KN

Nominal Shear Stress τ_v= V_u/ bd = 0.146 N/mm^2 < 1.75N/mm^2 ie τ_{max}/2 ok

K=1.3

% of steel=100 A_s/ bd= 100 * 314.16/2 * 1/ (1000*200)= 0.08%

So, τ_c= 0.29N/mm^2 [Table 19]

Shear Capacity=k* $\tau_c$bd=1.3*0.29=0.377>0.146 (Ok)

Step 6: Check for Deflection:

Span <10m

l/d=5.49/0.11= 49.9   [Cl.23.2, IS 456: 2000]

So, α= 26, β=1.

Let % of steel= 0.2

F_y=500 N/mm^2

F_s=0.58*500*235.8/314.16=217.66N/mm^2

From graph for modification factor vs % tension reinforcement graph

γ=2

λ=1; (no compression reinforcement provided)

δ=1; (other than flanged beam)

γ*δ*λ*α*β= 52> l/d (ok)

Hence, slab is safe in deflection.

Step 7: Check for Development Length:

Development length, Ld=ϕ*0.87f_y/4τ_{bd}=447.92mm [Cl26.2.1, IS 456,2000]

M_1=0.87*f_y*A_{st}*(d-0.416*n)=14.32KNm

V=16.06KN, for 90° bent, 8ϕ = 80mm

L_d should not exceed 1.3M,/V+L_0 =1239.15mm   ok

Embedded length available at support assuming support width of 230mm=230 - nominal cover=200mm >L_d/3

So, development length is adequate.

8.3 RAMP SLAB

A ramp slab is a type of concrete slab that is sloped or inclined to allow vehicular or pedestrian traffic to transition from one level to another. It is commonly used in multi-level car parks, parking garages, and other structures where there is a change in elevation.

Ramp slabs are typically designed to have a minimum slope of 5%, which means that for every 100 feet of horizontal distance, there is a 5-foot change in elevation. This slope is necessary to provide a smooth transition between levels and to ensure that vehicles can safely and easily navigate the ramp.

The design of ramp slabs involves careful consideration of factors such as the weight and type of vehicles that will be using the ramp, the height of the levels being connected, and the available space for the ramp. Proper design and construction of ramp slabs are essential to ensuring the safety and functionality of the structure.

Here are the general design steps for a ramp slab:

Determine the height difference between the levels: The first step in designing a ramp slab is to determine the height difference between the levels being connected. This information is important in calculating the required slope of the ramp and the length of the ramp required to achieve the desired slope.

Calculate the required slope: The slope of the ramp is determined by dividing the height difference by the horizontal distance of the ramp. The minimum recommended slope for a ramp is 5%, but the slope can be increased depending on the specific requirements of the project.

Determine the length of the ramp: Once the slope of the ramp is determined, the length of the ramp can be calculated based on the available space and the required slope. The ramp length should be long enough to provide a smooth transition between levels while maintaining a safe speed for the vehicles using the ramp.

Determine the thickness of the slab: The thickness of the slab is determined based on the weight of the vehicles using the ramp, the type of soil or substrate beneath the slab, and local building codes and regulations.

Plan the reinforcement: The reinforcement of the ramp slab is designed to support the weight of the vehicles and prevent cracking or failure of the slab. The reinforcement is typically in the form of steel bars or wire mesh and is placed in the slab according to the design specifications.

Prepare the construction details: The final step in the design process is to prepare the construction details, including the layout of the reinforcement, the location and size of the expansion joints, and any other specific requirements of the project.

Detail Design :

Slab no: Storey 3.1 ramp floor, Label F127

Unique name: 61

Step 1: Dimension of Slab:

Dimensions:6.425 x 4.16m

L_x=6.425m, L_y=4.16m

Depth of Slab D = 130 mm

Diameter of Bars φ = 10mm

Clear Cover = 15 mm

Effective Depth (d) = 110mm

M30, Fe500

Effective length,

In short span, lx_{eff} =4.16+0.11=4.27mm or 4.16+0.23=4.39m i.e. 4.27m

[Cl 23.2.1,IS 456,2000]

In long span ly_{eff}=6.425+0.1=6.525m
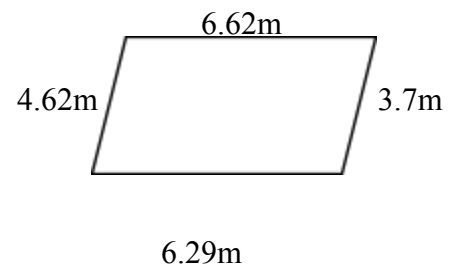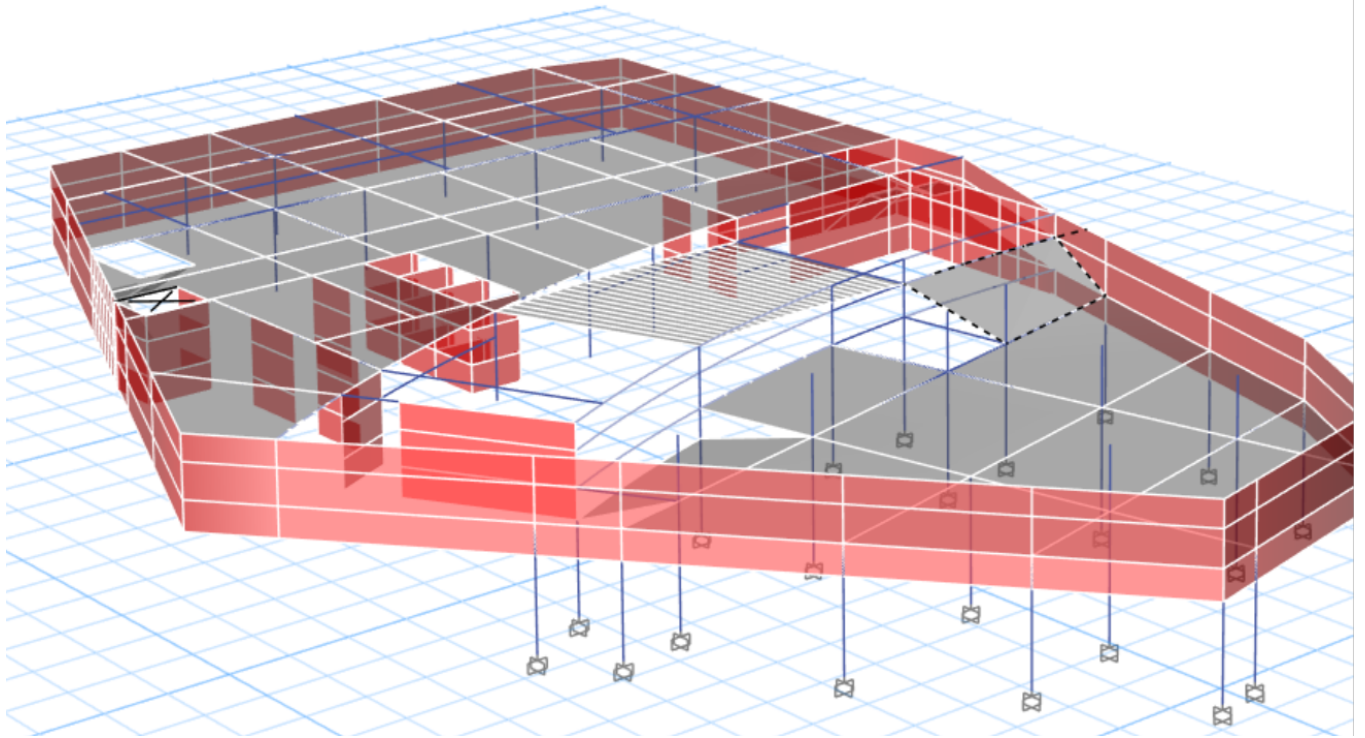
Fig 8.2 Ramp Slab Design

Step 2: Load Calculation

Imposed load= 5KN/m^2

Floor finish =1.15KN/m^2

Self wt. of slab= 0.13 x 25=3.25KN/m^2

Total Factored load=14.1KN/m^2

Step 3: Moment and Shear Calculation

$(L_y/L_x)_{eff}$ = 6.525/4.27=1.528<2

i.e. So, two way slab

-ve moment co.eff at continuous edge	-ve moment at continuous edge per unit width.	+ve moment co.eff at midspan	+ve moment at midspan.
α_x=0.06	M_x=15.43KNm	α_x=0.045	M_x=11.57KNm
α_y=0.	M_y=0	α_y=0.035	M_y=8.99KNm

For shear,

V_y=wL_y/3=30.67KN

$$V_x = wL_y(2 - \frac{Ly}{Lx})/4 = 10.86KN$$

Step 4: Tension Steel Calculation: [Annex G,]

$Mu_{,lim} = 0.36 x Xu_{,lim} /d (1-0.42*X_{u,max}/d)bd^2 f_{ck} = 0.133*f_{ck}bd^2$

i.e. d=62.186mm<100mm (ok)

To get A_{st},

$$M_u = 0.87f_y A_{st}(d - \frac{A_{st}f_y}{bf_{ck}})$$

	A_{st} required	Rebar spacing	Provided Spacing	A_{st} calculated
M_x^-	340	231	225	349.06
M_y^-	0	0	0	0
M_x^+	251.37	312.45	300	261.8
M_y^+	214.32	366.46	300	261.8

Step 5: Check for Shear:

K=1.3

Along short span, % of steel=0.32

So, τ_c= 0.464N/mm^2 [Table 19]

Shear Capacity=k $\tau_c$bd=66.35KN > 30.67KN (Ok)

Along long span, % of steel=0.238

So, τ_c= 0.3604N/mm^2 [Table 19]

Shear Capacity=k $\tau_c$bd=51.54KN > 10.86KN (Ok)

Step 6: Check for Deflection:

        Span <10m   [Cl.23.2, IS 456: 2000]

l/d= 4.27/0/.11= 38.82

So, α= 26, β=1.

Let % of steel= 0.2

F_y=500 N/mm^2

F_s=0.58*500*214.32/261.8=237.41N/mm^2

From graph for modification factor vs % tension reinforcement graph

γ=1.7

λ=1; (no compression reinforcement provided)

δ=1; (other than flanged beam)

γ*δ*λ*α*β= 44.2>l/d(ok)

Hence, slab is safe in deflection.

Step 7: Check for Development Length:

Development length, Ld=ϕσ$_s$/4τ_{bd}=453.125mm   [Cl26.2.1, IS 456,2000]

L_d should not exceed 1.3M$_1$/V+L$_0$ =750.56mm

Embedded length available at support assuming support width of 230mm=230-nominal cover=200mm >L$_d$/3

So, development length is adequate.

---

9. COLUMN DESIGN

Columns in RCC buildings are vertical structural members that transfer the loads from the superstructure (beams, slabs, etc.) to the foundation below. They are usually designed in a rectangular, square or circular shape and are made of reinforced concrete, which is a combination of concrete and steel bars.

Columns play a crucial role in the stability and strength of a building. They provide the necessary support for the weight of the structure and help resist lateral forces such as wind or earthquake loads. The design of columns involves determining the appropriate size, spacing, and reinforcement required to safely carry the loads imposed on them maintaining strength and serviceability of structure.

The size and number of columns in a building depend on several factors such as the height of the building, the type of foundation, the building's intended use, and the architectural design. In general, taller buildings require more columns to support the weight of the structure and resist lateral forces. Properly designed and constructed columns are essential for the structural integrity and safety of an RCC building.

Columns can be classified based on their slenderness ratio, which is the ratio of the effective length of the column to its cross-sectional dimension. Slenderness ratio is an important factor that affects the behavior and strength of a column under load. Some of the common types of columns based on slenderness ratio are:

1. Short column: A short column has a slenderness ratio less than 10, and it fails due to compressive forces. The design of a short column focuses on providing adequate reinforcement to resist the compressive forces.

2. Intermediate column: An intermediate column has a slenderness ratio between 10 and 30, and it fails due to a combination of compressive and flexural forces. The design of an intermediate column focuses on providing adequate reinforcement to resist both the compressive and flexural forces.

3. Long column: A long column has a slenderness ratio greater than 30, and it fails due to buckling. The design of a long column focuses on providing adequate reinforcement to prevent buckling and maintain stability.

Geometrically, some of the common types of columns are:

1. Rectangular column: This is the most common type of column and is used in most RCC buildings. It has a rectangular cross-section and is easy to construct and reinforce.

2. Circular column: This type of column has a circular cross-section and is used in buildings where aesthetic appeal is important. It is also more resistant to buckling than rectangular columns.

3. Square column: This type of column has a square cross-section and is used in buildings where space is limited. It is also easy to construct and reinforce.

4. L-shaped column: This type of column has an L-shaped cross-section and is used in buildings where the columns are located at the intersection of two walls. It provides better lateral support to the walls and is more stable than rectangular columns.

5. T-shaped column: This type of column has a T-shaped cross-section and is used in buildings where the columns support a beam. It provides better support to the beam than rectangular columns.

6. Composite column: This type of column is made of two or more materials, such as concrete and steel. It is used in buildings where high strength and durability are required.

The selection of the type of column depends on various factors such as the architectural design, load requirements, available space, and construction cost.

In a multi-storey seismic prone building, the column can fail under various conditions. Some of the common failure conditions in columns are:

1. Compression failure: This occurs when the column is subjected to high compressive forces and it buckles or crushes. This can happen if the column is too slender or if it is not reinforced properly.

2. Shear failure: This occurs when the column is subjected to high shear forces and it cracks or fails due to insufficient reinforcement or poor quality concrete.

3. Flexural failure: This occurs when the column is subjected to high bending moments and it cracks or fails due to insufficient reinforcement or poor quality concrete.

4. Combination of compression, shear, and flexural failure: In some cases, the column may fail due to a combination of these forces. This can happen if the column is not designed properly to resist the expected loads and forces during an earthquake.

5. Connection failure: This occurs when the connection between the column and the beam or foundation fails due to inadequate design or construction practices. This can result in the collapse of the entire building.

To prevent these failure conditions, the columns in a multi-storey seismic prone building should be designed to withstand the expected loads and forces during an earthquake. The design should consider the appropriate column type, size, reinforcement, and detailing to ensure adequate strength, stability, and ductility. The construction practices should also follow the design specifications to ensure that the columns are constructed properly and that the connections are strong and reliable. Regular inspections and maintenance should also be conducted to detect any potential issues and prevent failure conditions.

The design of a column in an RCC building involves several steps, including:

1. Determination of the load: The first step is to determine the loads that the column will have to carry, including the dead load (weight of the structure), live load (weight of people and furnishings), and any other applicable loads, such as wind or earthquake loads.

2. Selection of the column type: Based on the load requirements and architectural design of the building, the appropriate column type (square, rectangular, circular, etc.) is selected.

3. Calculation of the column dimensions: The next step is to calculate the dimensions of the column, including its height, cross-sectional area, and reinforcement required. This calculation is based on the load requirements and the selected column type.

4. Determination of reinforcement: Once the column dimensions are determined, the next step is to calculate the required reinforcement. This involves determining the spacing and size of the reinforcing bars needed to safely resist the loads imposed on the column.

5. Checking for stability and strength: The design must ensure that the column is stable and has adequate strength to safely carry the loads imposed on it. The stability is checked by analyzing the buckling and sway of the column, while the strength is verified by comparing the maximum stresses in the column to the allowable stresses.

6. Detailing of reinforcement: The final step is to provide detailed drawings and specifications for the column's reinforcement. This includes information on the placement and size of the reinforcing bars, as well as any required hooks or bends.

9.1 Detail Design 1: Square Column

Column ID= C236

Unique Name= 311

Floor= 1st floor

Load Combination: 1 DL + x. LL +(Rsy+0.3Rsx)

Materials:

Concrete grade: M25

Longitudinal Reinforcement: Fe500

Transverse Reinforcement: Fe415

Geometric properties of column

Depth of column D =1000mm

Width of column B =1000mm

Storey height = 3.6m

Unsupported Length =3600-700=2900mm   (IS 456:2000 Cl 25.1.3)
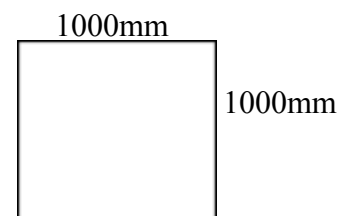
Taking Clear cover (d) =50mm

Effective cover (d') = 50+(32/2) =66mm (Taking rebar size ø = 32mm)

1. Requirements of ductile designing

   Factored Axial load (largest of all) = 11581.43KN

   Axial stress = Factored axial load= $\frac{11581.43 \times 10^3}{1000 \times 1000} = 11.581 N/mm^2$

   $\leq 0.40 f_{ck} = 12 N/mm^2$ (ok)

   (IS 13920:2016 Cl 7.1)

2. Check for minimum member size

   Maximum dimension of rebar in beam d_b = 25mm

   $\therefore$ 20db = 500mm or 300mm

   Minimum dimension of column = 1000mm > 20 × d_b (NBC 105 Cl4.2.1)

   b/d=1>0.4   Hence ok

   All the other requirements of [IS 13920:2016 Cl 7.1] were fulfilled during preliminary design.
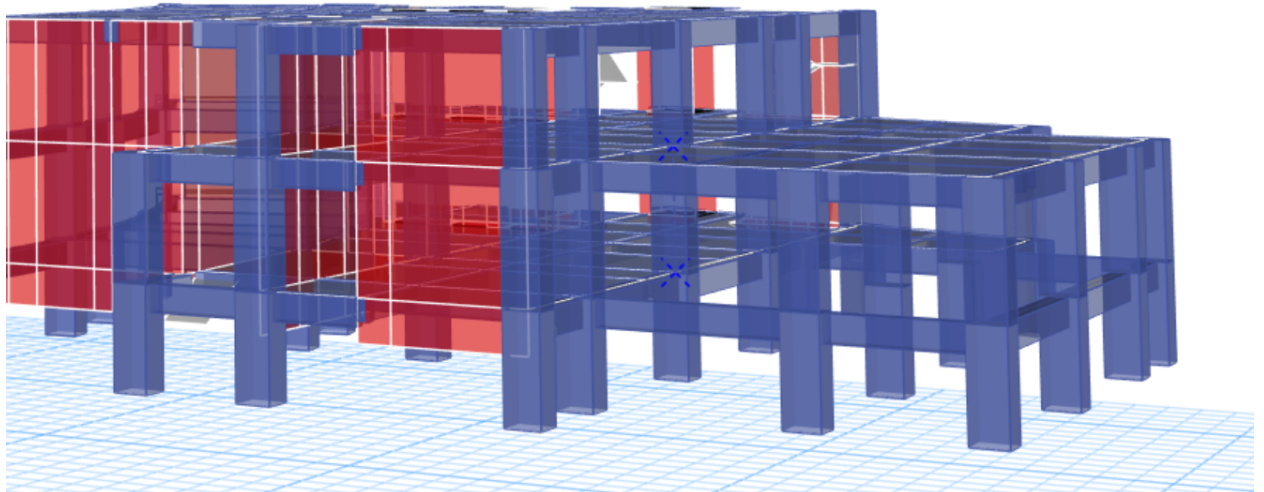
Fig: 9.1 Rectangular Column Section for Design

3. Design Load

P(KN)	M_2(KNm)	M_3(KNm)	V_2(KN)	V_3(KN)
9487.8645	370.343	-20.9541	20.5306	0.3375

4. Check for slenderness   [Annex E, IS 456:2000]

   MOI of section:

   MOI of column= $0.0833 m^4$

   MOI of beam section= $0.015 m^4$

   Length of adjoining beam=7.65m

   Height of column=2.9m

   For column, $I_{eff}/L=0.028$

   For Beam, $I_{eff}/L=0.00196$ [Annex E, Fig 26, IS 456:2000]

   So, $\beta_1=\beta_2= \dfrac{\sum K_c}{\sum K_c + \sum K_b} = \dfrac{2\times0.028\times0.7}{0.7\times2\times0.028+4\times0.00196\times0.35} = 0.934$

   From ETABS, Q (for storey 1) = 0.007227

   Since Q<0.04, our column is non-sway

   $L_{eff}/l=0.9224$

   $L_{eff}=0.9224\times3600=3320.64mm$ for both x & y axes.

5. Classification of column

   $L_{eff}/D=3320.64/1000=3.3206<12$ i.e. short column

6. Minimum Design moments

   Accidental eccentricity

   $e_a= L_{unsupported}/500+D/30= 2900/500+1000/30=39.13mm$ or 20mm [Cl 25.4, IS 456: 2000]

   $e_a=39.13mm$ (<0.05D=50mm)

---

7. Therefore, design moments

   Mu_2=370.343 kN-m

   Mu_3=-20.9541KNm   In both directions, it is biaxially loaded with a short column.

8. Calculation of longitudinal rebar

   Assuming minimum percentage of longitudinal rebar(p)=0.8%

   Area of rebar=0.8/100x1000x1000=8000mm^2

   The reinforcement is to be distributed along four sides equally.

   $$\frac{d^{'}}{D} = \frac{66}{1000} = 0.066 \approx 0.1$$

   p/f_{ck}=0.8/30=0.027

9. Using SP16: 1980 chart 48 for finding uniaxial moment capacity of the section in both directions.

   $$\frac{Pu}{f_{ck}bD} = \frac{9487.8645 \times 10^{\wedge}3}{30 \times 1000 \times 1000} = 0.316$$

   $$\frac{Mu}{f_{ck}bD^2} = 0.75$$

   Mu= 0.075x30x1000^3=2250KNm>Mu_2&Mu_3

10. Check for biaxial bending   [IS 456:2000 Cl 39.6]

    P=0.8%

    Fy=500N/mm^2

    Puz=0.45 f_{ck} Ac+0.75fyAsc

    = 0.45x30x1000x1000(1-0.8/100)+ 0.75x500x1000x1000x0.8/100

    =14904 KN

11. $$\frac{Pu}{Pu_z} = \frac{9487.8645}{14904} = 0.6366$$ [IS   456:2000   Cl   39.6]

    $$\alpha_n = \frac{1+\frac{Pu}{Puz}-0.2}{0.8} = 1.546$$

$$\left(\frac{Mux}{Mux_1}\right)^{\alpha_n} + \left(\frac{Muy}{Muy_1}\right)^{\alpha_n} = \left(\frac{370.343}{2250}\right)^{1.546} + \left(\frac{20.954}{2250}\right)^{1.546} = 0.0623 < 1 \ ok$$

Provided reinforcement:

As, req =0.8% of 1000^2 = 8000mm^2

Provide : 8-32mm & 8 – 22mm bars

Provided area= 9475.043 mm^2 > 8000mm^2 ok

12. Calculation of shear reinforcement

We have, $V_{2\ (major)}$= 20.5306 KN & $V_{3\ (minor)}$= 0.3375 KN

[IS 13920 Cl 7.5]

Design shear force demand on columns is larger value of:

   a. Factored shear demand as per linear structural analysis.

   b. Factored equilibrium shear force demand when plastic hinges are formed at both ends of the beams given by

$$Vu = 1.4 \times \frac{Mu^{As} + Mu^{Bh}}{h_{st}} \quad V \ 1.4 \times \frac{Mu^{Ah} + Mu^{Bs}}{h_{st}}$$

Beam	Capacity +ve (sag) KNm	Capacity –ve (hog) KNm
1	195.1794	-217.3048
2	141.1129	-211.308
3	194.3024	-370.0072
4	165.7529	-370.0072

$$Vu \ major = 1.4 \times \frac{Mu^{4s} + Mu^{3h}}{hst} \quad V \ 1.4 \times \frac{Mu^{4h} + Mu^{3s}}{hst} = 208.35 \ v \ 219.45$$

$$= 219.45 \ KN$$

$$Vu \ minor = 1.4 \times \frac{Mu^{1s} + Mu^{2h}}{hst} \quad V \ 1.4 \times \frac{Mu^{1h} + Mu^{2s}}{hst} = 158.08 \ v \ 139.38$$

$$= 158.08 KN$$

Ductility governs design shear in both direction:

$V_{2\,major}$ = 219.45 KN

$V_{2\,minor}$ = 158.08 KN

Shear Capacity of concrete

50% of longitudinal reinforcement in tension   [IS 456:2000, Cl 40.2]

Ast= 0.5 x 9475.043 =4737.5215 mm²

d= 1000- 50 – 28/2 =936 mm

$100\frac{Ast}{bd} = 100 \times \frac{4737.5215}{1000 \times 936} = 0.506$

τ_c = 0.5 N/mm²   [IS 456:2000, Table19]

Compression factor $\delta = 1 + 3\frac{Pu}{Ag \times fck} = 1 + \frac{3 \times 9487.8645 \times 10^3}{1000 \times 1000 \times 30} = 1.95 > 1.5$

[IS 456 : 2000 Cl 40.2.2]

δ=1.5

Actual shear capacity τ_c =1.5 x 0.5 = 0.75 N/mm ²

Shear capacity of concrete τ_v = 750 KN

13. Stress Check   [Cl 40.1, IS 456:2000]

Nominal Shear stress, $\tau_{v,\,major}$ = V_2/ bd = $\frac{219.45 \times 10^3}{1000 x\,936} = 0.235 N/mm^2$

Nominal Shear stress, $\tau_{v,\,minor}$ = V_3/ bd = $\frac{158.08 \times 10^3}{1000 x\,936} = 0.169 N/mm^2$

$\tau_{c\,max}$ =3.5 N/mm²

Shear stress both minor and major <3.5 KN/mm²   ok

14. Reinforcement calculation for central region

Nominal Shear stress < τ_c , min shear reinforcement is to be provided. So follow

[NBC 105 Annex A Cl 4.2.3]

a. Diameter of transverse reinforcement = 8mm

b. Spacing of tiers 300mm

So, provide 8- 300mm c/c in the central region.

15. Reinforcement Calculation for confining region

   Confining zone is taken max. of

   - Larger dimension of column = 1000mm
   - 1/6 of clear span = 483.33mm
   - 450mm   [NBC 105: 2020 Annex A: Cl 4.3 a]

     Lo = 1000mm

   Spacing is min of   [NBC 105: 2020 Annex A: Cl 4.3 b]

   - ¼ x1000 = 250mm
   - 6x 28 = 168mm
   - 100mm

     Provide spacing of 100mm   Splicing of vertical bars

          [IS 456:2000, Cl 26.2.1]

     Max of 50% bars should be spliced at a section

iii. A_{sh}= max of {0.18 Sv h f_{ck} / fy(A_g/ A_k-1), 0.05Sv h f_{ck}/fy

        = {0.915 Sv or 1.084Sv

     So, Sv = 0.25*π* 8^2/ 1.084 = 105 mm

16. Development Length

   $Ld = \frac{0.87\,fy\phi}{4\,\tau b\,d} = \frac{500 \times 0.87 \times 32}{4 \times 1.6 \times 1.4} = 1553.57mm$

   Provide lap length of 1600mm

17. Hooks on Transverse Reinforcement [NBC 105: 2020, Annex A Fig 4-5, amendment]

   Hook length = 8 ϕ or 75mm

              = 8x 8 or 75mm

              = 64 or 75 mm

    So, provide 75 mm

9.2 Detail Design 2: Design of Circular Column

Column ID= C74

Unique Name= 362

Floor= 3-Parking floor

Load Combination: 1 DL + x LL+ (Rsy+ 0.3 Rsx)

Materials:

Concrete grade: M25

Longitudinal Reinforcement: Fe500   600mm

Transverse Reinforcement: Fe415

Geometric properties of column

Diameter of column D =600mm

Storey height = 3980mm

Unsupported Length =3980-700=3280mm   (IS 456:2000 Cl 25.1.3)
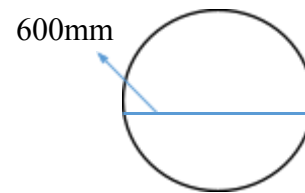
Taking Clear cover (d) =50mm

Effective cover (d') = 50+(32/2) =66mm (Taking rebar size ø = 32mm)
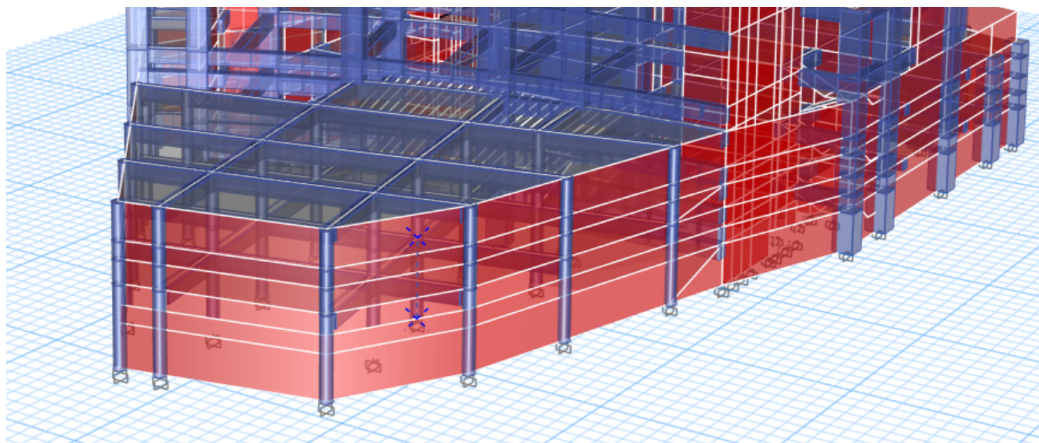
Fig 9.2: Circular Column Section for Design

1. Design Load

P(KN)	M$_2$(KNm)	M$_3$(KNm)	V$_2$(KN)	V$_3$(KN)
624.74	486.91	-14.17	365.05	378.77

2. Check for slenderness   [Annex E, IS 456:2000]

   MOI of section:

   MOI of column= $\frac{\pi D^4}{64}$ =0.0064m^4

   MOI of beam section= 0.015m^4

   Length of adjoining beam=6.5m

   Height of column=3.28m

   For column, I$_{eff}$/L=0.002

   For Beam, I$_{eff}$/L=0.0023

   So, $\beta_1 = \beta_2 = \dfrac{\Sigma K_c}{\Sigma K_c + \Sigma K_b} = \dfrac{2 \times 0.002 \times 0.7}{0.7 \times 2 \times 0.002 + 4 \times 0.0023 \times 0.35} = 0.465$

   From ETABS, Q for storey 3-parking = 0.0013

   Since Q<0.04, our column is non-sway

   L$_{eff}$/l=0.596   [Annex E, Fig 26, IS 456:2000]

   L$_{eff}$=0.596x3980=2372 mm for both x & y axes.

3. Classification of column

   L$_{eff}$/D=2372/600=3.95<12 i.e. short column

4. Minimum Design moments

   Accidental eccentricity

   e$_a$= L$_{unsupported}$/500+D/30= 3280/500+600/30=26.56mm or 20mm [Cl 25.4, IS 456:2000]

   e$_a$=26.56mm

   0.05D= 0.05x600=30mm> 26.56mm

   Therefore, design moments

Mu$_2$=486.91KNm

Mu$_3$=-14.17KNm

In both directions, it is a biaxially loaded short column.

5. Calculation of longitudinal rebar

Assuming minimum percentage of longitudinal rebar(p)=2%

Area of rebar=$\frac{2\times\pi}{4\times100}\times 600^2$=5654.87mm²

The reinforcement is to be distributed along four sides equally.

$\frac{d^{'}}{D} = \frac{66}{600} = 0.11\approx0.1$

p/f$_{ck}$=2/30=0.0667

6. Using SP16: 1980 chart 60 for finding uniaxial moment capacity of the section in both directions.

$\frac{Pu}{f_{ck}bD} = \frac{624.74\times10^{\wedge}3}{30\times600\times600} = 0.06$

$\frac{Mu}{f_{ck}bD^2} = 0.08$

Mu= 0.08x30x600³=518.4KNm>Mu$_2$&Mu$_3$

7. Check for biaxial bending   [IS 456:2000 Cl 39.6]

P=0.8%

Fy=500N/mm^2

Puz=0.45xf$_{ck}$xA$_c$+0.75f$_y$A$_{sc}$

= 0.45x30x0.25x π x 600² x (1-2/100)+ 0.75x500x0.25 x π x 600² x 2/100

=5861.27KN

8. $\frac{Pu}{Pu_z} = \frac{624.74}{5861.27} = 0.107 < 0.2$

$\alpha_n = 1,$

$$\left(\frac{Mux}{Mux_1}\right)^{\alpha_n} + \left(\frac{Muy}{Muy_1}\right)^{\alpha_n} = \left(\frac{486.91}{518.4}\right)^1 + \left(\frac{14.17}{518.4}\right)^1 = 0.966 < 1 \ ok$$

Provided reinforcement:

As, req =2% of 0.25 x π 600^2 = 5654.87mm²

Provide : 8 – 32mm bars

Provided area= 6433.98 mm² > 5654.87mm² ok

9. Calculation of shear reinforcement

We have, $V_{2\ (major)}$= 365.05 KN & $V_{3\ (minor)}$= 378.77 KN

[IS 13920 Cl 7.5]

Design shear force demand on columns is larger value of:

c. Factored shear demand as per linear structural analysis.

d. Factored equilibrium shear force demand when plastic hinges are formed at both ends of the beams given by

$$Vu = 1.4 \times \frac{Mu^{As} + Mu^{Bh}}{hst} \ V \ 1.4 \times \frac{Mu^{Ah} + Mu^{Bs}}{hst}$$

Beam	Capacity +ve (sag) KNm	Capacity –ve (hog) KNm
1	25.17	58.36
2	62	74
3	57.46	88.47
4	82.57	110.81

$$Vu\ major = 1.4 \times \frac{Mu^{4s} + Mu^{3h}}{hst} \ V \ 1.4 \times \frac{Mu^{4h} + Mu^{3s}}{hst} = 60.16 \ v \ 59.19$$

$$= 60.16 KN$$

$$Vu\ minor = 1.4 \times \frac{Mu^{1s} + Mu^{2h}}{hst} \ V \ 1.4 \times \frac{Mu^{1h} + Mu^{2s}}{hst} = 34.88 \ v \ 40.58$$

$$= 40.58 KN$$

Vu major & Vu minor from structure analysis governs design shear in both direction:

$V_{2\ major}$ = 365.05 KN

$V_{2\ minor}$ = 378.77 KN

Shear Capacity of concrete

50% of longitudinal reinforcement in tension   [IS 456 Cl 40.2]

Ast= 0.5 x 6433.98 =3217 mm^2

d= 600- 50 – 32/2 =534 mm

$$100\frac{Ast}{bd} = 100 \times \frac{3217}{\frac{\pi}{4}\times534^2} = 1.44$$

τ_c = 0.0.748 N/mm^2 [IS 456:2000, Table19]

Compression factor $\delta = 1 + 3\frac{Pu}{Ag \times fck} = 1 + \frac{3\times624.74\times10^3}{\frac{\pi}{4}\times600^2\times30} = 1.22 < 1.5$

[IS 456 : 2000 Cl 40.2.2]

δ=1.22

Actual shear capacity τ_c =1.22 x 0.748 = 0.913 N/mm^2

Shear capacity of concrete τ_v = 258.22 KN< Vu, minor

Req. Shear capacity of ties= 378.77-258.22=120.55KN

$Vu_s = \frac{0.87\ Asv\ d}{Sv}$

So, $\frac{Asv}{Sv} = \frac{120.55}{0.87 \times 415\times534} = 625.3\ mm$

Design of Lateral ties:

$\Phi_t = \Phi_l/4$= 32/4= 8mm or 6mm

Adopt bars of dia 8mm

Min spacing for lateral ties < 600/2= 300 mm or 16 Φ_t= 512mm or 300mm

Asv = π x 8^2 /4 * 4= 201.07 mm

Sv= 321.54mm

Let us provide 4 closed loop ties of spacing 250 mm c/c

10. Reinforcement Calculation for confining region

Confining zone is taken max. of [NBC 105: 2020 Annex A: Cl 4.3 a]

- Larger dimension of column = 600mm
- 1/6 of clear span = 546.67mm
- 450mm

Lo = 600mm

Spacing is min of [NBC 105: 2020 Annex A: Cl 4.3 b]

- ¼ x600 = 150mm
- 6x 32 = 192mm
- 100mm

Provide spacing of 100mm

A_{sh}= max of {0.18 Sv h f $_{ck}$ / fy(A_g/ A_k-1), 0.05Sv h f_{ck}/fy

$\quad$ = {1.43Sv or 0.9Sv

So, Sv = 0.25*π* 8^2/ 1.43 = 115 mm

11. Splicing of vertical bars [IS 456:2000, Cl 26.2.1]

Max of 50% bars should be spliced at a section

$Ld = \frac{0.87\,fy\phi}{4\,\tau b\,d} = \frac{500 \times 0.87 \times 32}{4 \times 1.6 \times 1.4} = 1553.57mm$

Provide lap length of 1600mm

12. Hooks on Transverse Reinforcement [NBC 105: 2020, Annex A Fig 4-5, amendment]

Hook length = 8 φ or 75mm

$\quad\quad\quad\quad$ = 8x 8 or 75mm

$\quad\quad\quad\quad$ = 64 or 75 mm

So, provide 75 mm

10. DESIGN OF STAIRCASE

A staircase in an RCC building is a critical component that provides vertical access between different floors of the building. The design and construction of an RCC staircase require careful consideration of several factors, including the location, size, and loading conditions.

The structural components of an RCC staircase typically include the following:

1. Stringers: These are the structural members that support the treads and risers and transmit the loads to the RCC beams or columns.

2. Treads: These are the horizontal surfaces that provide a stepping surface for the user.

3. Risers: These are the vertical surfaces that support the treads and provide the necessary height between two consecutive treads.

4. Landing: This is the level platform at the top or bottom of the staircase that provides a resting place for the user before proceeding to the next level.

It is essential to follow the relevant codes and standards while designing and constructing an RCC staircase to ensure its safety and durability. Proper reinforcement detailing, such as providing adequate steel reinforcement in the stringers, treads, and risers, is crucial for the staircase's strength and stability. The use of slip-resistant finishes on the treads and landings can also help prevent accidents due to slipping.

Design Steps for Staircase:

The design of a staircase in an RCC building involves several steps, which are as follows:

1. Determine the location and size of the staircase: The location and size of the staircase should be determined based on the building's architectural design and the number of floors in the building.

2. Calculate the rise and run of the steps: The rise and run of the steps are calculated based on the floor-to-floor height of the building and the desired tread depth and riser height.

3. Determine the number of steps: The number of steps in the staircase is determined by dividing the total height of the staircase by the riser height.

4. Calculate the dimensions of the landing: The dimensions of the landing are calculated based on the number of steps in the staircase and the building's architectural design.

5. Design the structural members of the staircase: The structural members of the staircase, including the stringers, treads, and risers, should be designed based on the loading conditions and the building codes and standards.

6. Provide reinforcement details: The reinforcement details, including the spacing and size of the reinforcement bars, should be provided based on the design requirements and the building codes and standards.

7. Prepare the construction drawings: The construction drawings should include the detailed dimensions, reinforcement details, and other necessary information to ensure proper construction of the staircase.

10.1 Detail Design 1 (Open- well Staircase):

2.33m

1.1m   1.1m

1.62m   1.86m

0.9m
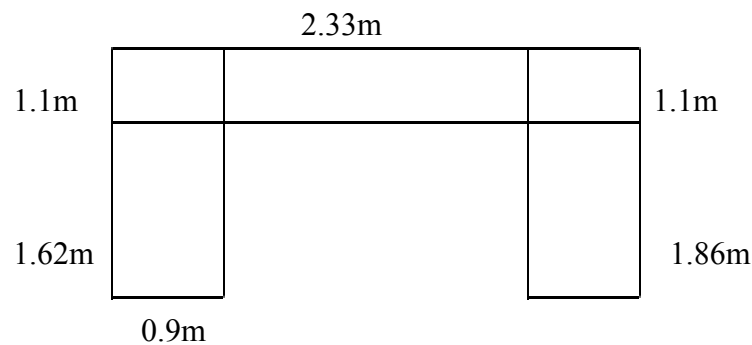
1. Known Data:

   Storey height= 3.6m

   Height of flight= 1.371m

   Tread (T)= 330mm

   Riser(R)= 130mm

   No. of riser= 1371/130= 11

   No. of tread= 10

   Length of flight= 1.26m

Width of flight =1.1m

Dwaist=150mm

c.c=15mm

ϕ=10mm

d=150-10/2-15=130mm
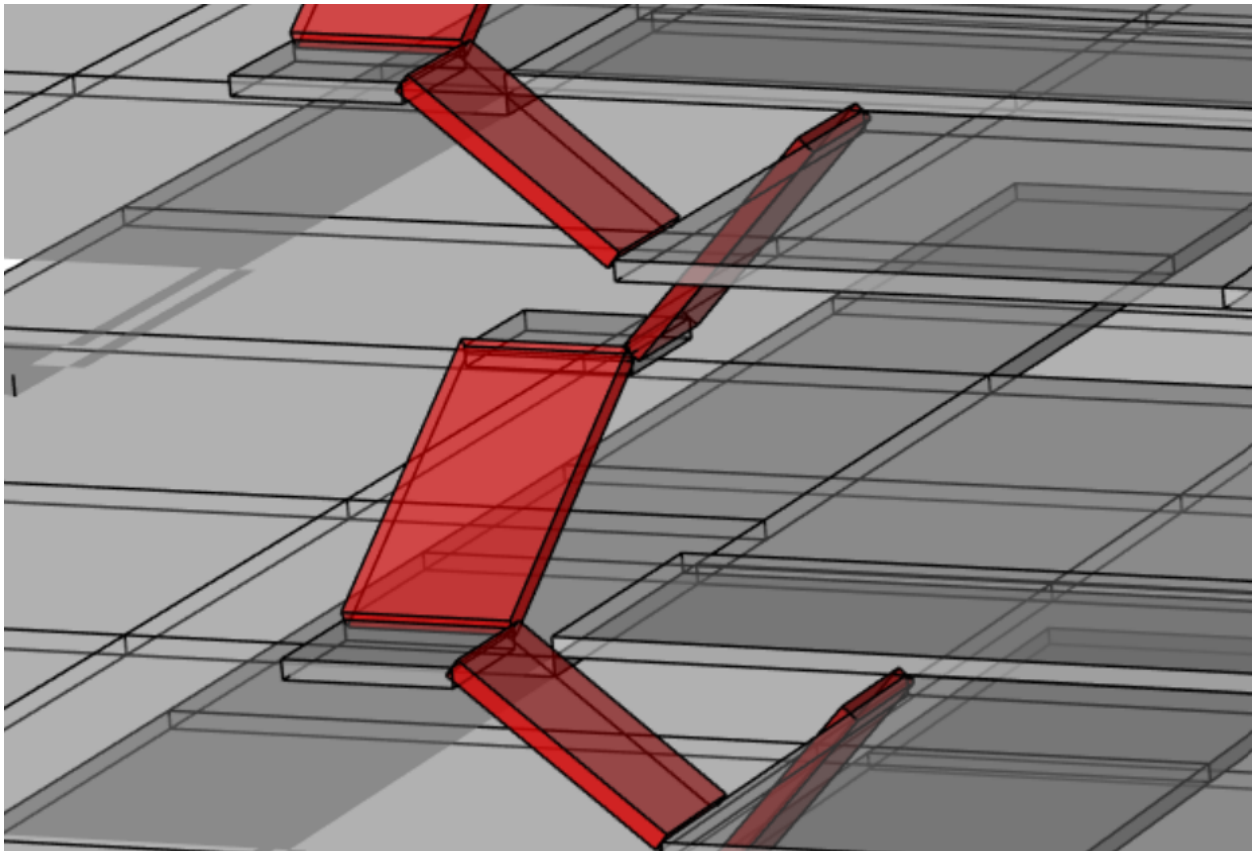
Fig : 10.1 Open-well Staircase Section for Detail Design

2. Load Calculations:

2.1 For Landing:

Dead Load from slab = 0.13*25 = 3.25KN/m^2

Floor finish =1.15 KN/m^2

Imposed Load = 4 KN/m^2

Total Design Load = 8.4x1.5 = 12.6KN/m

Half Load = 6.3 KN/m

2.2 For Flight:

i. Waist Slab = $25*0.15*1/0.33*\sqrt{0.13^2 + 0.23^2}$ = 4.03KN/m

ii. Wt. of steps = 250.5x0.13x0.33x1/0.33 =1.625KN/m

iii. Imposed Load = 4KN/m

iv. Floor Finish =1.15KN/m

Total load =10.805KN/m

Design Load =16.21KN/m

3. Analysis

16.21KN/m

6.3KN/m

A   1.26m   1.1m   B
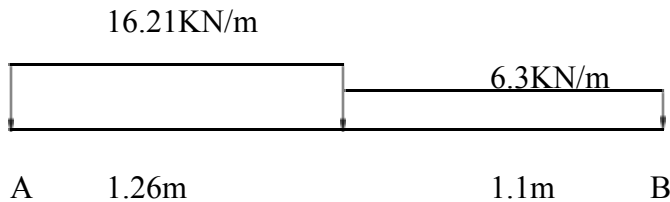
R_A * 2.36 - 16.21 * 1.26 * (1.26/2+1.1)-6.3 * 1.1 * 1.1/2=0

R_A=16.587KN

R_B=16.21x1.26+6.3x1.1-16.587=10.768KN

4. Max Moment Calculation (M_{max}),

V_x=0

16.587-16.21x=0

X=1.023m from A

M_{max}=16.587x1.023-(16.21x1.023x1.023/2)=8.486KNm

5. For Fe 500,

M_{max}=0.133*$f_{ck}bd^2$

8.486*10^6=0.133*30*1000* d_{req}^2

So, d_{req}=46.12mm<130mm ok

6. From Annex G, (IS 456:2000)

$M_{max} = 0.87 f_y . A_{st} . d(1 - \frac{f_y A_{st}}{f_{ck} bd})$

$A_{st} = 153.06 mm^2$

$A_{st,min} = 0.12 bd = 0.12 \times 1000 \times 150 = 180 mm^2$

Adopt $\phi = 10mm$

$Spacing = \frac{1000}{\frac{180}{\frac{\pi \times 10^2}{4}}} = 436.33mm; \ 3d = 390mm \ or \ 300mm$

Adopt Spacing = 300mm

Provide 10mm ϕ bars @ 300mm c/c

i.e. $Area_{provided} = \pi x \frac{10^2 x 1000}{4x300} = 261.8 mm^2$

Check for shear,

$\tau_v = V_u / bd \quad = \frac{16.587 x 10^3}{1000 \times 130} = 0.127 N/m^2$ \qquad [IS 456: 2000, Cl 40.1]

, k = 1.3 \qquad [Cl Table 40.2.1.1, IS 456: 2000]

$100 A_s / bd = \frac{100 \times 261.8}{1000 \times 130} = 0.2$

So, $\tau_c = 0.33 N/mm^2$

$K \ x \ \tau_c = 0.429 N/mm^2 > \tau_{max}$ (ok) \qquad [Table 19, IS 456: 2000]

7. Detailing

Provide 10mm ϕ bars @ 300mm c/c

For distribution bars

Area = 0.12 % of bD = 180mm^2

$Spacing \ required = \frac{900}{180 \times \frac{4}{\pi \times 10^2}} = 392.7 \approx 300mm$

10. 2 Detail Design 2 (Open-well Staircase)
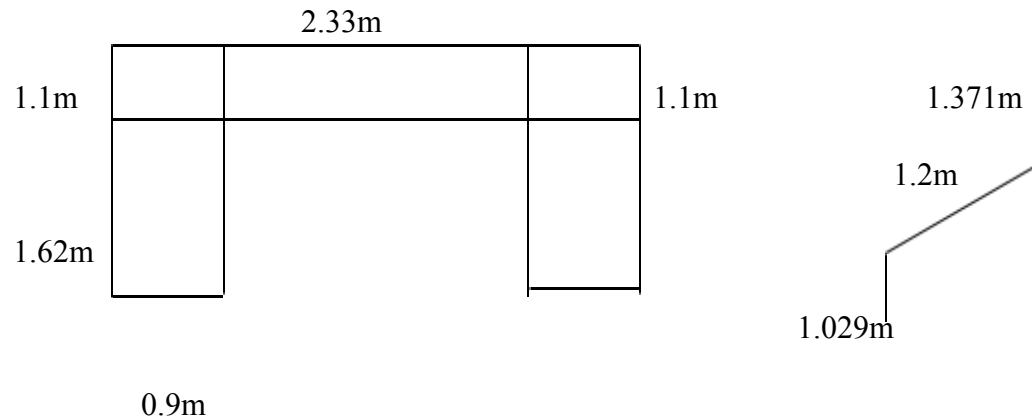
Known Data:

Storey height= 3.6m

Height of flight= 1.029m

Tread (T)= 330mm

Riser(R)= 130mm

No. of riser= 1029/130= 8

No. of tread= 7

Length of flight= 1.62m

Width of flight =0.9m

Dwaist=150mm

c.c=15mm

ϕ=10mm

d=150-10/2-15=130mm

1. Load Calculations:
    a. For Landing:

Dead Load from slab=0.13x25=3.25KN/m^2

Floor finish =1.15 KN/m^2

Imposed Load=4 KN/m^2

Total Design Load=8.4x1.5=12.6KN/m

Half Load= 6.3 KN/m

   b. For Flight:

v.   Waist Slab= $25\text{x}0.15\text{x}1/0.33\text{x}\sqrt{0.13^2 + 0.23^2}$ =4.03KN/m

vi.   Wt. of steps=25x0.5x0.13x0.33x1/0.33=1.625KN/m

vii.   Imposed Load= 4KN/m

viii.   Floor Finish=1.15KN/m

        Total load=10.805KN/m

        Design Load=16.21KN/m

2. Analysis

   16.21KN/m

6.3KN/m

A   1.26m   1.1m   B
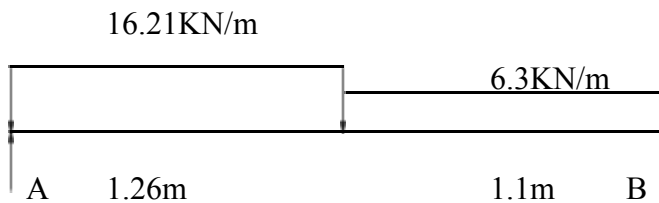
16.495   10.698

R_A x 2.35-16.21x1.25x(1.25/2+1.1)-6.3x1.1x1.1/2=0

R_A=16.495KN

R_B=16.21x1.25+6.3x1.1-16.495=10.698KN

V_{max}= 16.495KN

3. Max Moment Calculation (Mmax),

V_x=0

16.495-16.21x=0

X=1.018m from A

M_{max}=16.495x1.018-(16.21x1.018x1.018/2)=8.393KNm

_____

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4. For Fe 500,

$M_{max}=0.133*f_{ck}bd^2$

$8.393 \times 10^6=0.133*30*1000*d_{req}^2$

So, $d_{req}=45.87mm<130mm$ ok

5. From Annex G, (IS 456:2000)

$M_{max}=0.87f_y.A_{st}.d(1-\frac{f_y Ast}{f_{ck}bd})$

$8.393*10^6=0.87*500*A_{st} \times 130(1-\frac{500 \times A_{st}}{30 \times 1000 \times 130})$

$Ast=151.354mm^2$

$A_{st,min}=0.12bd=0.12 \times 1000 \times 150=180mm^2$

Adopt $\phi=10mm$

$Spacing=\dfrac{1000}{\frac{180}{\frac{\pi \times 10^{\wedge}2}{4}}}=436.33mm$; 3d=390mm or 300mm

Adopt Spacing =300mm

Provide 10mm ϕ bars @ 300mm c/c

i.e. $Area_{provided}=\pi x \frac{10^{\wedge}2 x 1000}{4x300}=261.8mm^2$

6. Check for shear,

$\tau_v= V_u/ bd \quad =\frac{16.495 \times 10^{\wedge}3}{1000 \times 130}= 0.127N/m^2$ [IS 456: 2000, Cl 40.1]

,k=1.3   [Cl Table 40.2.1.1, IS 456: 2000]

$100 A_s/ bd= \frac{100 \times 261.8}{1000 \times 130}= 0.2$

So, $\tau_c= 0.33N/mm^2$

$K* \tau_c=0.429n/mm^2 > \tau_{max}$ (ok)   [Table 19, IS 456: 2000]

7. Detailing

Provide 10mm ϕ bars @ 300mm c/c

For distribution bars

        Area =0.12 % of bD=180mm²

$$\text{Spacing required}=\frac{900}{180\times4/(\pi\times10^2)}=392.7\approx300mm$$

Detail Design 3:
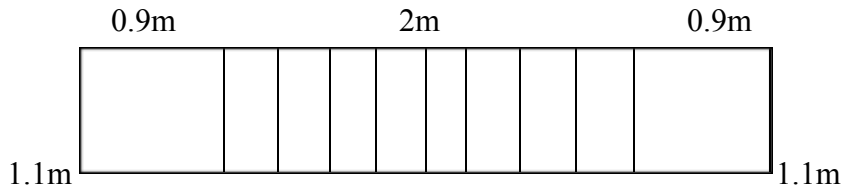
Known Data:

Storey height= 3.6m

Height of flight= 1.2m

Tread (T)= 330mm

Riser(R)= 130mm

No. of riser= 9

No. of tread= 8

Length of flight= 2m

Width of flight =1.1m

Dwaist=150mm

c.c=15mm

ϕ=10mm

d=150-10/2-15=130mm

1. Load Calculations:

    a. For Landing:

    Dead Load from slab=0.13x25=3.25KN/m²

    Floor finish =1.15 KN/m²

Imposed Load=4 KN/m^2

Total Design Load=8.4x1.5=12.6KN/m

    b. For Flight:

ix.   Waist Slab= 25x0.15x1/0.33x$\sqrt{0.13^2 + 0.23^2}$ =4.03KN/m

x.   Wt. of steps=25x0.5x0.13x0.33x1/0.33=1.625KN/m

xi.   Imposed Load= 4KN/m

xii.   Floor Finish=1.15KN/m

        Design Load=16.21KN/m

2. Analysis

    UDL on landing slab= 12.6/2=6.3KN/m

    UDL on flight=16.208KN/m
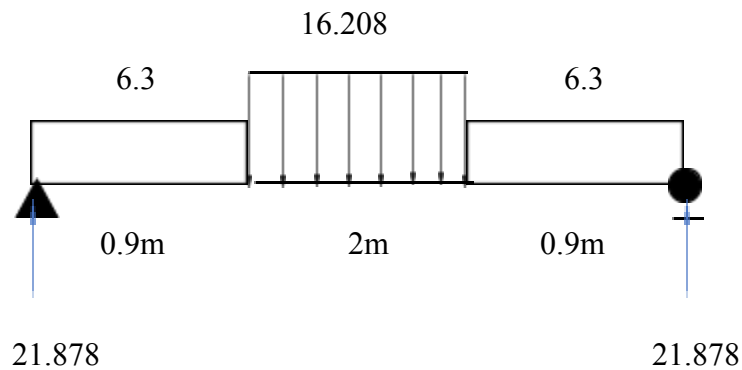
V_{max}= 21.878KN

3. Max Moment Calculation (M_{max}),

V_x=0

X=1.9m from A

M_{max}= 21.878*(0.9+2/2)-6.3*0.9x(0.9/2+2/2)-(16.208x2/2x1/2x2/2)

=33.3467-8.104=25.243KNm

4. Design of flight   [From Annex G, (IS 456:2000)]

    $M_{u,lim}$=0.36 $(\frac{Xu}{d})\times\{1 - 0.416\times\frac{Xu}{d}\}\times(b\times d^2\times f_{ck})$

M_u=25.243KNm

X_u/d=0.46 for Fe 500

f_{ck}=30 Mpa

b=1000mm

d=79.268mm<130mm ok

i.e. it can be designed as singly reinforced under reinforced section (SRURS)

$A_{st} = \dfrac{Mu}{0.87fy(d-0.416xu)}$

$X_u = \dfrac{0.87f_y A_{st}}{0.36f_{ck}bd}$

A_{st}=446.58mm^2

5. Check for shear,

$\tau_v = V_u/\,bd \quad = \dfrac{16.495\times10^3}{1000\times130}= 0.127$N/m^2 [IS 456: 2000, Cl 40.1]

,k=1.3   [Cl Table 40.2.1.1, IS 456: 2000]

100 $A_s/\,bd$= $\dfrac{100\times446.58}{1000\times130} = 0.343\%$

So, τ_c= 0.418N/mm^2

Kx τ_c =0.543N/mm^2 > τ_{max} (ok)   [Table 19, IS 456: 2000]

6. Detailing

A_{st}=446.58mm^2

Adopt ϕ=10mm

Spacing= $\dfrac{1100}{\frac{446.58}{\frac{\pi\times10^2}{4}}}$ =193.456mm; 3d=390mm or 300mm

Adopt Spacing =190mm

Provide 10mm ϕ bars @ 190mm c/c

i.e. Area$_{provided}$= $\pi x \dfrac{10^2 x 1100}{4 x 190}$ =454.7mm^2

Provide 10mm ϕ bars @ 190mm c/c

For distribution bars

Area =0.12 % of bD=180mm^2

Spacing required= $\dfrac{1100}{180\times4/(\pi\times10^2)}$ = 480mm≈$400mm$

Hence provide 10mm dia bars with 300mm c/c.

10.3 Detail Design 3 (Dog-Legged Staircase)

1. Known Data

    - Floor height = 3.6m

    - Riser Height (R) = 160mm

    - Number of riser = 3.6/ 0.160 =22

    - Number of tread = 2*10 = 20

    - Length of flights in each floor (Going Length) = 3.3m

    - Tread Width (T) = 330mm

    - Width of flight = 1.69m

    - Width of landing slab = 2.47m

       Assumed Data:

    - Depth of waist slab(D) = 150mm

    - Clear cover = 15mm

    - Diameter of bar(Φ) = 10mm

    - d = D – cc – Φ/2 = 130mm
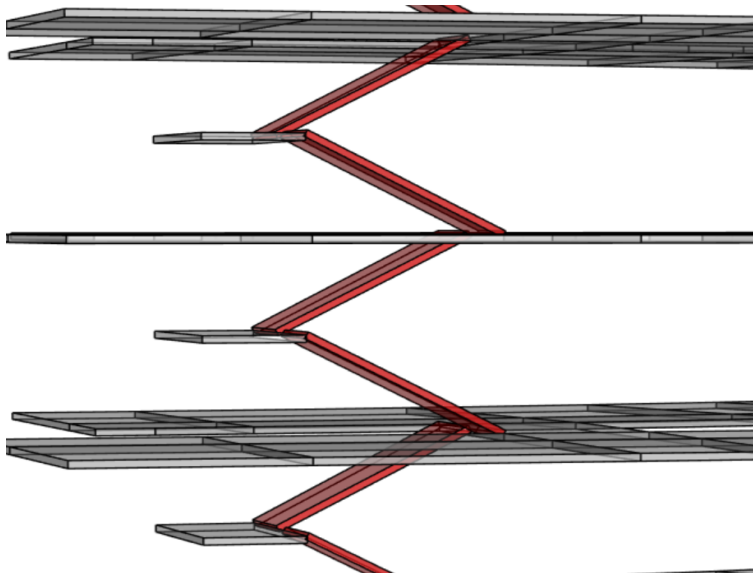
Fig 10.2 Dog Legged Staircase Section for Detail Design

2. Load Calculation

   For Landing:

   Dead load from slab = 0.130m*25KN/m³

   $\qquad$ = 3.25 KN/m²

   Floor Finish = 1.15KN/m²

   Imposed Load = 4 KN/m²

   Total Design load = (3.25 + 1.15 + 4)*1.5*1.69 KN/m

   $\qquad$ = 21.30 KN/m

   For Flight:

   Weight of waist slab on horizontal plane = (25* 0.15 * (3.3²+1.8²)^(1/2)) / 3.3

   $\qquad$ = 4.272KN/m²

   Weight of steps = 25 * 0.5 * 0.16 = 2 KN/m²

   Floor Finish = 1.15 KN/m²

   Imposed Load = 4 KN/m²

   Total load on flight = (4.272+2+1.15+4)*1.5*1.69

   $\qquad$ = 28.954 KN/m

3. Analysis

   UDL on landing Slab = 21.30 KN/m

   UDL on flight= 28.95 KN/m

   28.95 KN/m

   21.30 KN/m

   A   3.3m   2.47m   C

   R_AY= 79.48KN   R_CY=68.67KN
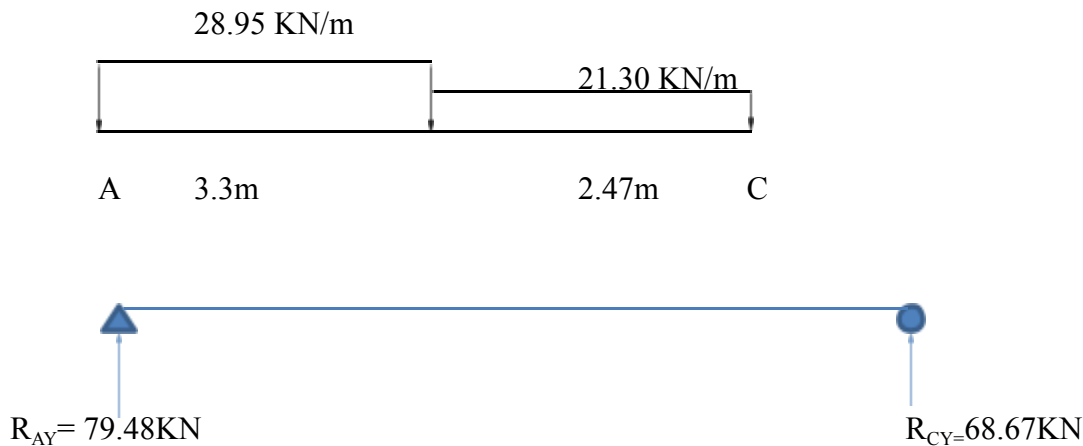
   Maximum Bending Moment at 2.75m from A = 79.48 * 2.75 – 28.95 * 2.75²/2

   $\qquad$ = 109.103 KNm

4. Design of Flight

<u>Check for depth of flight:</u>

For singly reinforced under reinforced section (SRURS): $d > d_{bal}$

$M_{u,lim} = 0.36(Xu/d)\{1\text{-}0.416(Xu/d)\}*(b*d^2*fck)$

For,

$M_{u,lim}$ = 109.103 KNm

Xu/d = 0.46

f_{ck} = 30 N/mm^2

b = 1690mm

We get,

d_{bal} = 127 mm< 130mm (OK)

Hence, the flight can be designed as a singly reinforced under-reinforced section.

<u>Finding A_{st} for SRURS:</u>

A_{st} = Mu/[0.87*fy*(d-0.416*xu)]

Where, $Xu = (0.87*fy* A_{st})/(0.36*f_{ck}*b*d)$

We get,

A_{st} = 1931.5mm^2

Which is the required area of steel to be provided for flight longitudinally.

A_{st} minimum = 0.12% of bD = 405.6 mm^2 (OK)

<u>Check for Shear:</u>

Minimum % of steel to be provided (Pt) = (100*1931.5)/(1690*130) = 0.88%

For Pt = 0.88 and M30 concrete:

τ_c = 0.626(IS 456 table 19; by interpolation)

Maximum shear force that the flight could resist = k* τ_c*1690*130

$$= 178.79\ KN > 79.48\ KN$$

(OK)(support reaction)

(where, k = 1.3 for D = 150mm)

5. Detailing

   Detailing for longitudinal rebar:

   Required area of steel = 1931.5mm^2

   Provide 12mm dia bar

   Cross sectional area = 113.097mm^2

   Required Spacing = 1690/(1931.5/113.097) = 98.957mm

   So provide 12mm dia bars along the width of flight at spacing of 90 mm c/c

   (Minimum spacing required = 3*d = 3*130 = 390mm or 300mm)

   A_{st} provided = 1690*113.097/90 =2123.7mm^2

   Detailing of distribution bar:

   Area of distribution bar required = 0.12% of b*D = 304 mm^2

   Spacing required = 1690/(304/78.54) = 436 mm

   Hence, provide 10 mm dia bars with spacing of 300mm c/c

11. DESIGN OF SHEAR WALL

A shear wall is a structural element designed to **resist lateral forces** acting on a building, such as those caused by wind or earthquakes. These walls are typically made of reinforced concrete, masonry, or wood and are placed in strategic locations throughout a building to help distribute these forces and prevent structural failure.

The basic principle behind a shear wall is to provide a **rigid and stable structure that can transfer lateral forces** to the foundation of the building. This is achieved by using a combination of materials and reinforcement that can withstand the loads imposed on the building.

Shear walls are commonly used in high-rise buildings, but they can also be found in smaller structures like residential homes. The **location and orientation of these walls are critical** to their effectiveness, as they **need to be positioned perpendicular to the direction of the lateral forces** location so that the structures have adequate stiffness to resist the lateral load and **control drift** of the overall building structure. It **acts as the vertical deep cantilever beam** to resist the in-plane shear and bending moments caused by the lateral loads.

In our building, the shear wall is of **slender type** i.e. H/L > 2 (as per **IS 13920 Cl. 10.1.4**), so the design procedure for such wall is followed as per IS 13920. Although the shear is used to resist the in-plane shear and in-plane bending moment, we have checked it for out-of-plane shear and moment also since Shear wall modeled as shell element shows behavior in out of plane.

For the design of shear wall, the wall with maximum shear and moment is chosen from the ETABS and the reinforcement for others are provided as per the designed shear wall.

11.1 Detailed Design

Pier ID: PW 50

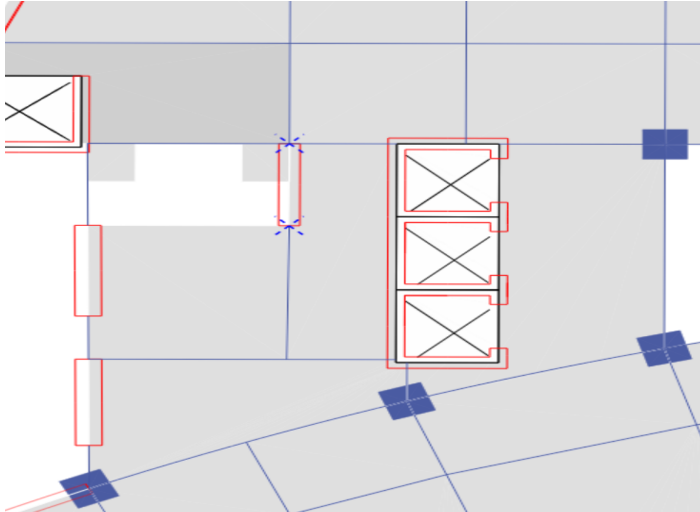Wall label: W50

Unique Name: 880

Fig 11.1: Shear Wall Section for Detail Design

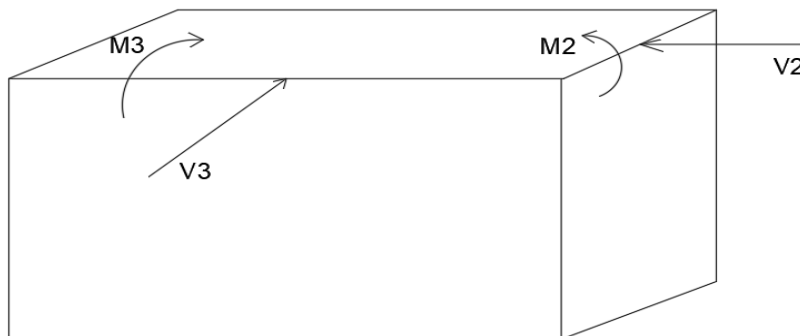Step 1: General dimensions of shear wall (Pier PW50)

• Length of shear wall (L) = 2.339m

• Height of shear wall (H) = 48.6m

• Thickness of shear wall (t) = 400mm

(H/L) = 20.778 >2

So the wall is of slender type.   [**IS 13920 cl. 10.1.4]**

Step 2: Design forces and moments (from ETABS)

Notations:

V_2 =Maximum In-plane shear force= 361.585KN

V_3 = Maximum Out-of-plane shear force=311.29KN

M_2 = Maximum Out-of-plane moments=415.2KNm

M_3 = Maximum In-plane moment=1028.4KNm

Pu= Maximum Axial Force=4575.57KN

Step 3: Vertical Reinforcement calculation

From   [**Annex A (IS 13920 : 2016)]**

Es= $2x10^5$MPa

Fy=500MPa

F_{ck}=30MPa for concrete grade M30

$$\beta = \left\{ 0.002 + \left(\frac{0.87fy}{Es} \right) \right\} \div 0.0035 = 1.193$$

$$\frac{X^*}{Lw} = \frac{0.0035}{0.0035 + (0.002 + \frac{0.87fy}{Es})} = 0.456$$

Assume Steel reinforcement ρ=1%

Φ=0.87fyxρ/fck=0.145

λ=Pu / (f_{ck} x t_w x L_w)=0.163

$$\frac{Xu}{Lw} = \frac{\Phi + \lambda}{2\Phi + 0.36} = 0.4738 > 0.456$$

So it is case <b>:

$$\alpha_1 = [0.36 + \phi(1 - \frac{\beta}{2} - \frac{1}{2\beta}) = 0.358$$

$$\alpha_2 = [0.15 + \frac{\phi}{2} (1 - \beta + \frac{\beta^2}{3} - \frac{1}{3\beta}) = 0.15$$

$\alpha_4 = (\frac{\phi}{\beta} - \lambda)$=-0.0415

$\alpha_5 = (\frac{\phi}{2\beta})$=0.061

$$\alpha_1\left(\frac{Xu}{Lw}\right)^2 + \alpha_4\left(\frac{Xu}{Lw}\right) - \alpha_5 = 0$$

On solving, $\frac{Xu}{Lw} = 0.474$

$\alpha_3 = \frac{\phi}{6\beta} x[\frac{1}{\frac{Xu}{Lw}} - 3]$=-0.018

$$\frac{Mu}{f_{ck} t_w l_w^2} = \alpha_1\left(\frac{Xu}{Lw}\right) - \alpha_2 x\left(\frac{Xu}{Lw}\right)^2 - \alpha_3 - \frac{\lambda}{2}$$

i.e. Mu=4759.08KNm

Here, the Mu is moment capacity=4759.08KNm

Maximum in-plane moment in shear wall=1028.4KNm<4759.08KNm

i.e. reinforcement is sufficient.

Steel area (Ast)=9356mm^2 in 2 curtains.

For 1 curtain Ast=4678mm^2

Provide 20mm φ, Spacing= 2339/{4678X4x7/(22x20^2)}=153mm

i.e. Provide 20mm φ, @150mm in two curtains**.**   **[IS 13920 : 2016 Cl. 10.1.9]**

Step 4: Check for Out of plane moment

Considering effective depth along thickness = 400-20-10-20/2 = 360mm

Mu$_{lim}$=0.87 x f$_y$ x A$_{st}$ x d x{1-(A$_{st}$ x f$_y$)/(bd x f$_{ck}$)}

=692.76kNm

Maximum out of plane moment is 415.2kNm < 692.76 kNm OK

Step 5: Design for In-plane shear:

Assuming effective length be 0.8Lw   [**IS 13920:2016 Cl. 10.2.1**]

Nominal Shear Stress $(\tau_v) = V_u/(t_w \text{ x } d_w) = 0.483 \text{N/mm}^2$

Design Shear Strength of concrete $(\tau_c) = 0.66 \text{N/mm}^2$ **[From Table 19 IS 456:2000]**

$\tau_c > \tau_v$

Therefore, we require only minimum horizontal rebar to resist the shear force.

Provide 8mm φ, spacing< A_{sv} x0.87 x fy/(0.4b) =273mm

Adopt spacing 200mm

i.e. Provide 8mm φ rebars @spacing 200mm.

Step 5: Check for out-of-plane shear

Considering only of concrete section

Design strength in out of plane

$= \tau_c * t_w * d_w$

=0.66*0.4*2.339=617.496KN>361.85KN(OK)

Step 6: Check for boundary element:

.2fck=6 KN/m²

$$\frac{P}{A} \pm \frac{M}{Z} = \frac{361.585}{2.339*.4} \pm \frac{1028.4}{2.339^2 * \dfrac{0.4}{6}} = (\text{-2,433,3.2}) \text{ Mpa} < 6 \text{ Mpa}$$

This suggests there is no need for a boundary element in a designed shear wall.

Conclusion:

Hence, provide 20mm φbar @150mm in vertical and 8mmφ @200mm in horizontal

direction.

---

12. DESIGN OF BASEMENT WALL

The basement wall serves the critical function of retaining lateral earth pressure and preventing moisture seepage into the building. In our building, two and a half floors (basement 2, basement 1, and semi-basement) are located underground.

Based on the final construction, the basement wall behaves as a propped cantilever as it is restrained at floor level and backfill being provided for the final construction of the wall. However, backfilling is done progressively as the basement wall is constructed in real practice. This induces cantilever action of the basement wall. Thus, considering the worst scenario and remaining on the safer side, the basement wall is designed as a cantilever slab. Lateral load due to soil and surcharge load is considered for the design of basement wall.
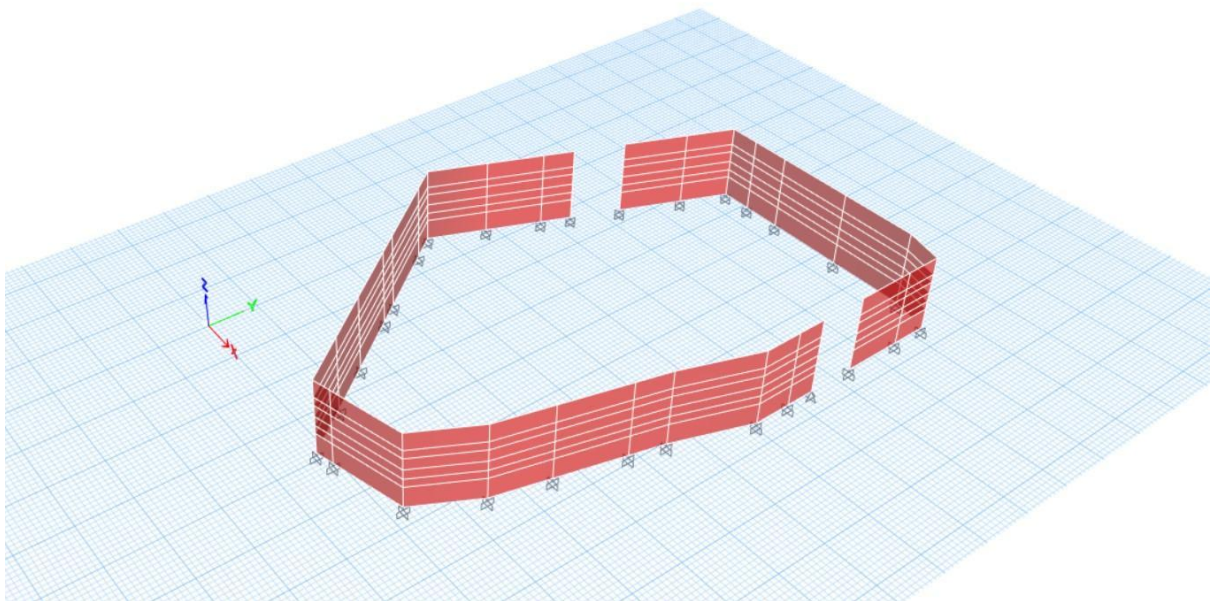
Fig 12.1 : Basement wall model in ETABS

Calculations
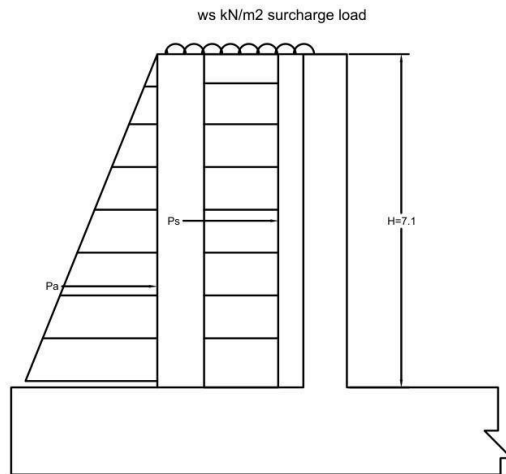
Fig 12.2 : Schematic representation of load on Basement wall

1) Design Constraints:

Clear Height of Basement Wall, H=7.8-.7=7.1

Specific Weight of soil, γ= 18 kN/m³

Angle of Internal Friction, θ= 30⁰

Surcharge load = 10 kN/m² (as per British Standard 8002, Minimum surcharge to be considered for vehicular load and building nearby)

Safe Bearing Capacity of Soil, q = 165 kN/m²

2) Approximate Design of section:

Let effective depth of wall, d =H/18=7100/18=394

Overall depth, D= 394+25+10/2 =424 mm

Provide, D= 450

Therefore, d=D-25-10/2= 420 mm

Slenderness ratio, l/d=7100/420= 16.9 [IS456:2000,Cl. 32.3.4]

3)Moment Calculation:

$K_a = \frac{1-sin\theta}{1+sin\theta}$ =0.333

Lateral load due to soil pressure, P_a=.5*K_a * γ* H^2

=0.5*.333*18*7.1²=151.07 kN/m

Lateral Load due to surcharge Load, P_s= K_a*W*H=.333*10*7.1²=23.643 kN/m

Characteristic Bending Moment at the base of Wall

Since, Weight of wall gives insignificant moment, so this can be neglected in the design

$M_c = P_a*H/3 + P_s*H/2 = 441.5$ kNm

Design Moment, $M = 1.5*M_c = 1.5*441.5 = 662.2$ kNm

4)Check for depth

Considering unit length of wall

$M_{ulim} = .133*f_{ck}*bd^2 = .133*30*1000*420^2 = 703.84$ kNm

Depth of wall form moment consideration:

$$d = \sqrt{\frac{M}{.133 f_{ck} b}} = 407.4$$

5)Calculation of Main steel reinforcement:

$$A_{st} = .5*\frac{f_{ck}}{f_y} bd(1 - \sqrt{1 - \frac{4.6M}{f_{ck}bd^2}}$$

[IS 456:2000,Cl. 32.5.a]

This gives: $A_{st} = 4391.68$ mm^2

Min. $A_{st} = 0.0012*bD = 540$ mm^2 $<A_{st}$

Max. Dia of Bar = D/10 = 450/10 = 45 mm   [IS456:2000, Cl. 26.5.2.2]

Since   D>200   mm,   Reinforcements   should   be   provided   into   two   [IS456:2000,Cl.32.5.1]

Grids, one near each face of the wall

Providing 20 mm dia bar, spacing of bars,

$$S_v = \frac{1000*314.15}{\frac{4391.68}{2}} = 143 \text{ mm}$$

Provide 20 mm dia @ 125 mm c/c at back face of wall

$$\text{Actual } A_{st} = \frac{1000*314.15}{\frac{125}{2}} = 5026.4 \text{ mm}$$

$$P_t = \frac{4833.07*100}{1000*450} = 1.11\% \qquad \text{[IS456:2000, Cl. 32.5.b]}$$

Max. Spacing = 3d= 3*420=1260 or 450 mm

Provide vertical reinforcement 20 mm dia @ 125 c/c at the both face of wall

6)Check for shear [IS 456-2000 Cl. 31.6.2.1 IS 456-2000 Table 19]

Shear force ,V= P_a+P_s=174.713 kN/m

Design shear force, V_u =1.5*V =174.713=262.07 kN/m

Nominal shear stress, $\tau_v = \dfrac{V_u}{bd}$ =0.62 N/mm²

Permissible shear stress, τ_c =0.66 N/mm²   Hence safe

Check for deflection: [IS 456-2000 Cl. 23.2.a]

L_{eff}=7.1 m

Allowable Deflection= l_{eff}/250=30.08 mm

ActualDeflection= $\dfrac{W_s l_{eff}^4}{8EI} + \dfrac{W_a I_{eff}^4}{30EI}$

$$= \frac{7520^4}{5000*\sqrt{30}*\dfrac{1000*450^3}{12}}\left(\frac{.333*10}{8}+\frac{.333*18*7.1}{30}\right) = 28.21 \quad mm$$

Which is less than allowable deflection. Hence, Safe.

7)Calculation of Horizontal reinforcement steel bars [IS456:2000, Cl. 32.5.c]

Area of Hor. Steel reinforcement= .002*bH=5964 mm^2

As the temperature changes occurs at front face of basement, 2/3 of

Horizontal reinforcement is provided at the front face and 1/3 of horizontal reinforcement is provided in the inner face.

Front face Horizontal reinforcement steel,

=2/3*5964=3976 mm^2

Providing 12 mm dia bars

No. of Bars required, N = 3976/113.1 $\approx$ 36 nos

$$Spacing = \frac{H - Clear\ Cover\ at\ both\ sides - dia}{N - 1}$$

=(7100-25*2-12)/36 = 195.5 mm

Providing 12 mm bars @ 180 mm c/c

Inner face Horizontal Reinforcement steel,

= 1/3 x 5964 = 1988 mm^2

Since reinforcement is half of front face

Providing 12 mm bars @ 360 mm c/c

8)Curtailment of Reinforcement [IS456:2000, Cl. 32.5.d]

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

$$l_d = \frac{\sigma_s \phi}{1.6 * 4 * \tau_{bd}} = \frac{.87 * 500 * 20}{1.6 * 4 * 1.5} = 906 \text{ mm}$$

The curtailment of bars can be done in two layers 1/3 and 2/3 heights of the stem above the base.

Let us curtail bars at 1/3 distance i.e. 2366.67 mm from base

Lateral Load due to soil pressure, P_a = .5 x 0.333 x 18x (2/3 *7.1)^2 = 67.14 KN/m

P_a = 67.14 KN/m

Lateral Load due to surcharge load, Ps = K_awH

= 0.333 x 10 x (2/3 *7.1) = 15.76 KN/m

Characteristic Bending Moment at the base of wall,

M_c = P_a x h/3 + P_s * h/2

M_c = 67.14* 7.1/3 + 15.76 * 7.1/2

M_c = 214.846 KN-m

Design Moment, M=1.5M_c=322.269 KN-m

Since this moment is less than half of the moment at base of stem, spacing of vertical reinforcement are doubled from 2366.67 mm from the base of the wall

Providing 20 bars @ 250 mm c/c above 2366.77 mm from base.

13. FOUNDATION DESIGN

A foundation refers to the sub-structure of a building or structure that is designed to support and transmit the weight of the superstructure (the above-ground portion) to the underlying soil or rock in a safe and stable manner. The foundation is typically the first component of a building or structure to be constructed and plays a crucial role in ensuring the stability, durability, and safety of the entire structure. The design of the foundation is based on the soil conditions, loads to be supported, and the intended use of the building or structure.

The two choices for modelling the footing are the Rigid Footing Model and the Flexible Footing Model. The Rigid Footing Model is applicable when the deformation of the footing is linear, meaning that it behaves like a rigid body. On the other hand, the Flexible Footing Model is used when the deformation is not linear. Although mat foundations are generally thick and rigid, incorporating their flexibility into the model would make it more accurate.

Determining loads is an essential aspect of the design process, and it is relatively straightforward to define loads from the superstructure. This is done by assuming fixed supports in the analysis of the superstructure, and the reaction on those supports is then applied to the footing as a load. The soil underneath the mat balances those loads with a reaction pressure called the contact pressure, which varies according to the type of soil and the flexibility of the footing. Obtaining accurate parameters for modelling the response of the soil to mat deformation is not a trivial task.

With the help of the CSi SAFE 2020 program, we developed a FEM model to represent the mat as a thick plate with the properties of M30 concrete and account for the flexibility of the footing. This software considers the flexibility of the footing, so the mat is neither completely rigid nor completely flexible. The soil support on the mat is modelled as Winkler medium, with spring stiffness values being the subgrade modulus of the soil (k). This generates a non-uniform pressure distribution proportional to the deflection of the footing, and the response of the mat (bending moment and shear forces within the mat) is computed as per usual formulation for structural mechanics in FEM.

For stability checks (the limit on the bearing pressure and settlement of soil), a uniform pressure distribution was assumed, which can be considered consistent with a perfectly rigid footing and soil as an independent spring support. However, the assumption of complete rigidity in stability checks, while considering flexibility in the FEM model for reinforcement design, can be considered a limitation of our work. A weak justification for this approach is that the stability checks are performed for serviceability loads and not limit state loads, so the behaviour of the foundation is not significantly different from that of a rigid body.

Subgrade Modulus of Soil

The subgrade modulus is defined as the ratio of the pressure applied to the soil to the resulting vertical deformation of the soil. The subgrade modulus is calculated from

$$\text{Modulus of subgrade, k} = \frac{Safe\ Bearing\ capacity * FOS}{Permissible\ settlement} = \frac{250*3}{.05} = 1500\ KN/m^3$$

The structure is located in Lainchaur, Kathmandu and the bearing capacity of soil at site is known. A research article entitled "Investigation of Soil at Different Locations of the Kathmandu Valley of Nepal" published in the American Journal of Science, Engineering and Technology suggests that the highest and lowest values of bearing capacity in Kathmandu were found to be 163 KN/m^2 and 58.6 KN/m^2 respectively. Low bearing pressure of soil may demand pile foundation for a high rise building of 52.2 m. Binding the scope of the project, we considered the bearing pressure of soil to be 250 KN/m^2 assuming that strength is increased by soil stabilisation.

Foundation Type Selection

For high rise buildings, raft foundation is more suitable than isolated footings covering more than 50 % of the plot. A simple calculation is done for choosing the footings:

DL+LL=206924.81 KN (from ETABS)

Plinth area= 1506 mm^2

Permissible bearing pressure= 200*1.25= 250 KN/m^2 (Bearing pressure increased by 25% for medium soil subjected to earthquake) [Clause 6.3.5.2, Table 1, IS 1893(Part I: 2016)]

$$\frac{1.5(DD+LL)}{\text{Permissible bearing pressure}} = \frac{1.5 * 206924.81}{250} = 1241 > .5 * \text{Plinth Area i.e } 753$$

So, the raft foundation is suitable for our building.

Special case for foundation

Raft foundation in our building plan is irregular. It is rigorous to manually calculate the bearing pressure under each column. It is difficult to design depth, longitudinal rebar is required in a raft foundation. Similarly, checking for punching failure, shear failure, and other issues is challenging. As a result, we model the raft foundation in ETABS and evaluate various depths that fulfil safer conditions in punching shear and flexural failure conditions.

Depth of Mat Foundation: 2220 mm

Subgrade Modulus of soil: 1500 KN/m³

Design Process:

1. Selection of Footing Type

2. Calculation of Load, and Defining Load Combination

3. Preliminary Sizing (depth of foundation)

4. Modelling

5. Stability Checks (Permissible Bearing and Settlement)

6. Check for punching shear and modification of footing depth if required

7. Design for Moments and Shears reinforcement obtained from analysis

Load Combinations:

Serviceable condition: (DL+LL) – This load combination is used to check for bearing pressure and permissible deflection

Limit state condition: 1.5(DD+LL) –This load combination is used to determine the reinforcement detailing for the raft foundation

Check for Settlement:

The permissible deflection for the raft foundation is 60 mm (as per Table 1 of IS 1904–1986). However, we take it as 50 mm as soil stabilisation is done to improve soil characteristics. The soil is modelled as Area Spring Stiffness with modulus of subgrade as 1500 KN/m^3.

The permissible deflection under service load obtained in SAFE is 13.8 mm, much below the permissible limit. Large area of mat foundation may be a reason for this low settlement.
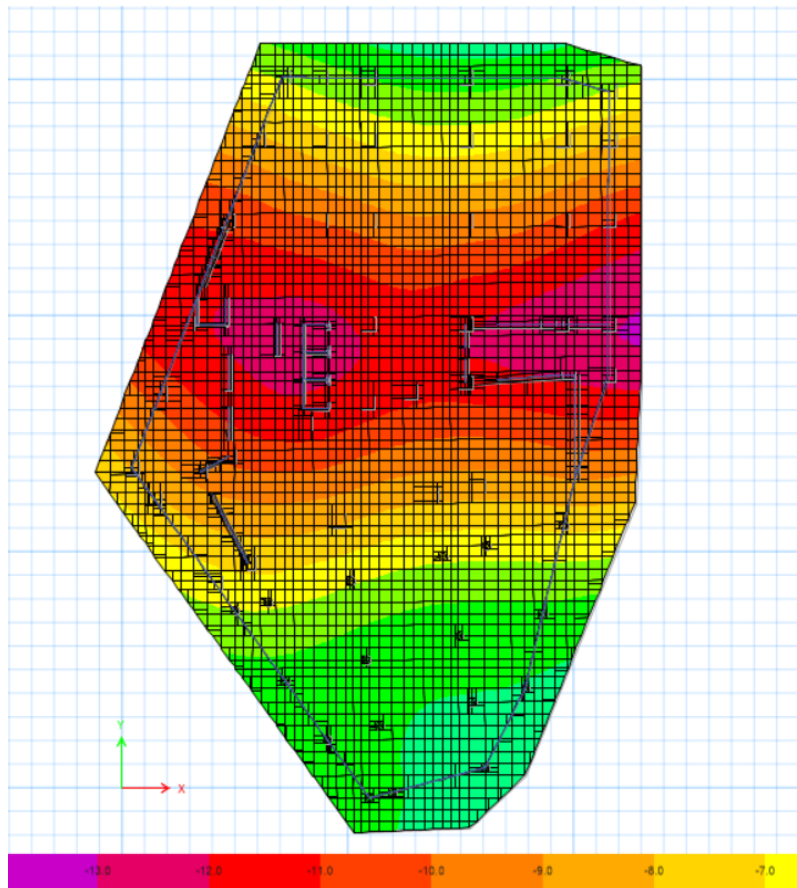

Fig 13.1: Settlement of Raft foundation modelled in SAFE

Check for allowable Bearing Pressure:

Permissible safe bearing pressure of soil $= 250$ KN/m^2

From SAFE analysis, Maximum bearing pressure obtained as 245 KN/m^2 for serviceable conditions. From this we can conclude stabilization of soil is not enough and it also suggests incorporate pile foundation as sub structure for our building.

Permissible safe bearing pressure of soil during an earthquake (250*1.25=312.5 KN/m^2). Bearing pressure obtained for load combination (DL+0.8LL+0.8EQ) is 240.779 KN/m^2 which is less than the permissible value during earhquake (312.5 KN/m^2).
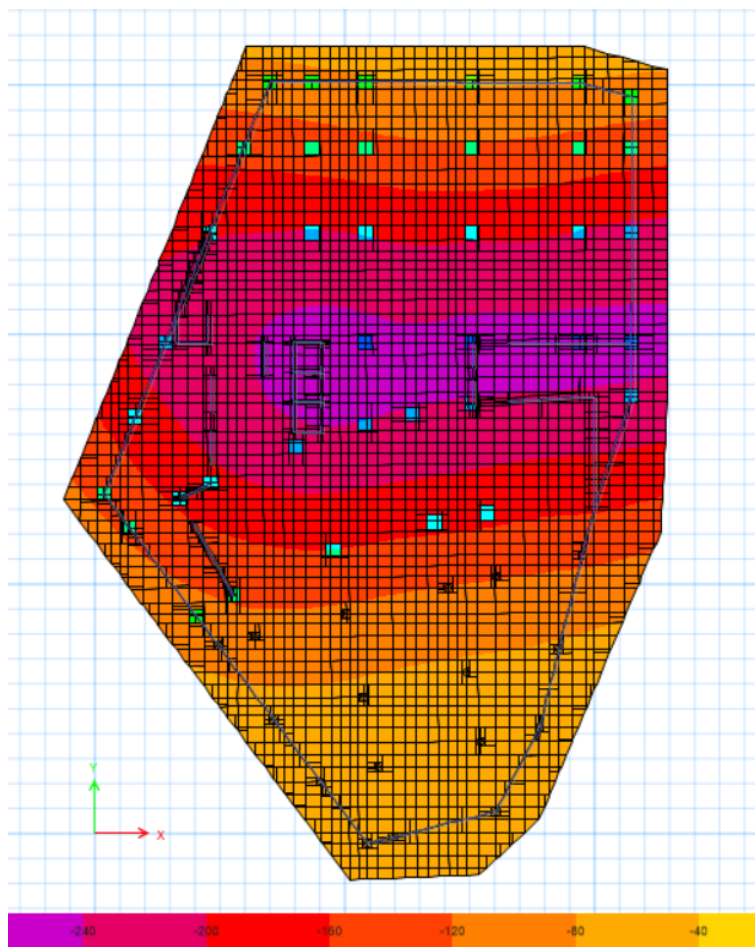
Fig 13.2 : Contact Pressure modelled in SAFE for serviceable condition

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Check for punching Shear:

Punching shear demand/capacity ratio found from SAFE analysis is found to be less than 1 in all possible locations of punching shear failure. In raft foundation, punching shear failure is typically not expected to occur along the shear wall location because the shear walls provide additional support to the slab and help distribute the load. The shear walls are designed to resist lateral loads and transfer them to the foundation, rather than to directly support vertical loads. This is represented by N/C (not calculated) below the basement wall and shear wall.
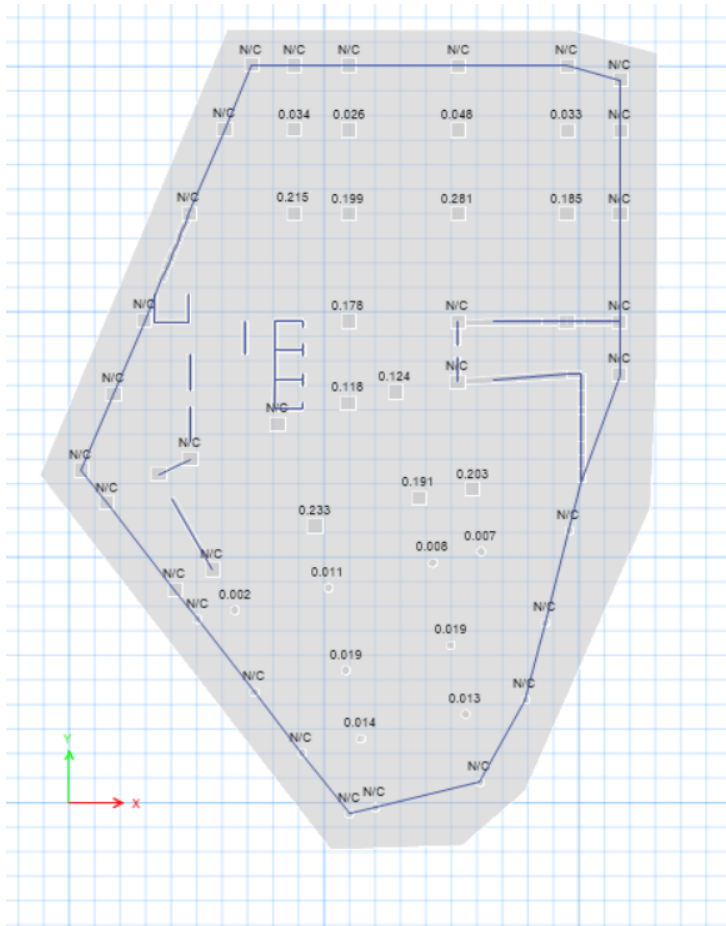
Fig 13.3 : Punching shear demand/capacity ratio from SAFE

Longitudinal Reinforcement

It will be difficult to find bearing pressure for irregular raft foundation. So, we extracted bearing pressure at ultimate state condition form SAFE Analysis which is used for reinforcement required for the section.
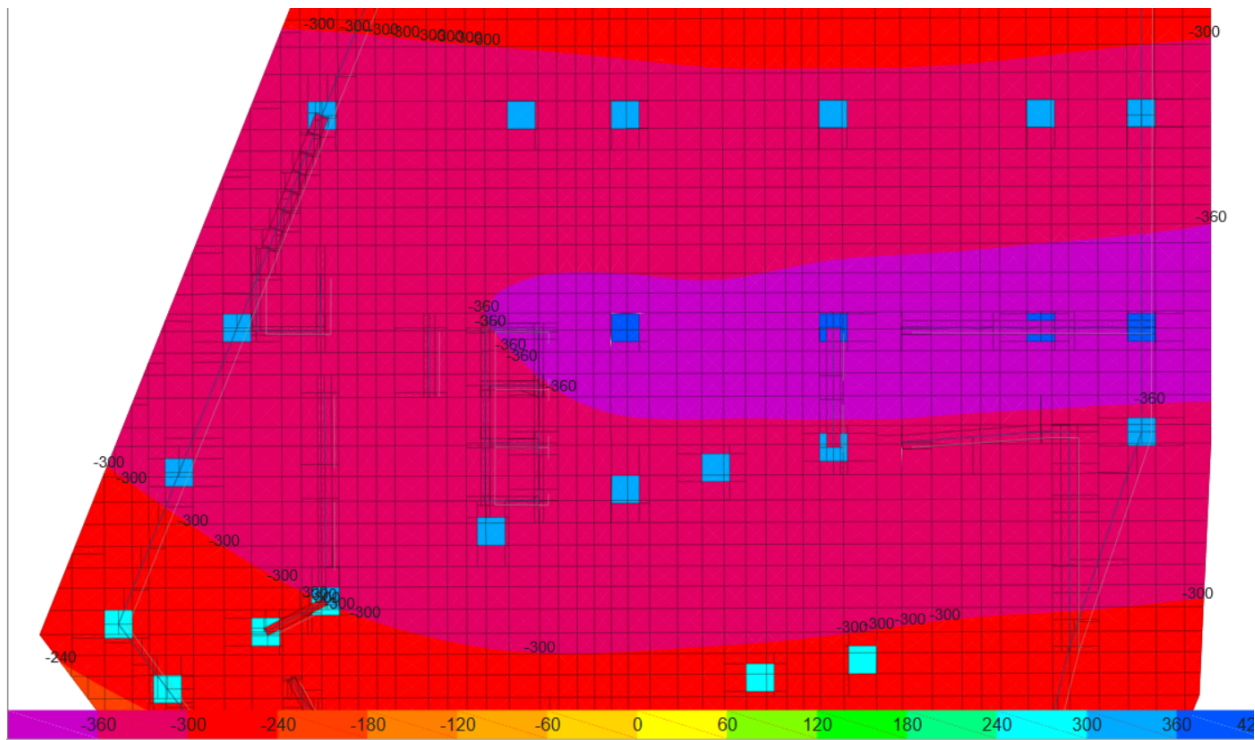
Fig: 13.4 Design of Critical strip

Manual Design:

In X direction

1. Contact pressure and length obtained form SAFE

Upward soil pressure, q = 368 KN/m^2

Maximum span length, l= 7.5 m

2. Moment Calculation   (IS 456-2000, Table 12)

Max. support moment : $\frac{q*l^2}{10} = \frac{368*7.5^2}{10} = 2070\ KNm\ per\ m$

Max. span moment: $\frac{q*l^2}{12} = \frac{368*7.5^2}{12} = 1725\ KNm\ per\ m$

3. Depth from moment consideration (SP 16, Table C)

Depth of footing, d= $\sqrt{\frac{M}{0.133*f_{ck}*b}} = \sqrt{\frac{2070*10^6}{0.133*30*1000}}$ =720.27 mm

4. Setting depth of foundation

Since footing is critical in shear, depth is increased by 3 times. So,

 d = 2160 mm

Providing 20 mm Φ bars and 50 mm clear cover,

D =1450+50+20/2= 2220 mm

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5. Check for two-way shear (IS 456-2000)

Nominal Shear Stress: $\tau_v = \dfrac{V}{b_0 * d}$ (Cl. 31.6.2.1)

b_0 is the perimeter

$\tau_v \leq k\tau_c$ (IS 456-2000) ;

$\quad k_s = \left(0.5 + \beta_c\right) \leq 1;$

$\quad \beta_c = L/B$

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

Here, (Cl 3.6.3.1)

$\quad \beta_c = 1$

$\quad k_s = 1$

$\quad f_{ck} = 30$

$\quad \tau_c = 1.369 \ N/mm^2$

a) Edge Column

Since projection is 2.5 m from the centre of the column, each column has an area up to at least 'd' distance from the edge of the column. So, max. load = critical load

b) Interior Column

$b_0 = 4(d + 1000) \ mm = 12640 \ mm$

Maximum load, P= 11686 KN (Max. axial force from ETABS for interior column)

$\tau_v = \dfrac{11686 * 10^3}{9200 * 1450} = 0.876 \ N/mm < \tau_c \quad (\tau_c = 1.369 \ N/mm^2 \ )$ Hence, safe.

6. Area of Steel (IS 456-2000, Cl 26.5.2.1)

Minimum area of steel : $A_{st,min}$ = 0.12% of bD = 2664 mm^2

a) Area of steel at support (Bottom Bars) [IS 456-2000 Annex G-1.1 b]

$$A_{st} = 0.5 * \frac{f_{ck}}{f_y} * b * d * (1 - \sqrt{1 - \frac{4.6*M}{f_{ck}*b*d^2}})$$

M= 2070 KNm/m

b= 1000 mm

d= 2160 mm

f_{ck} = 30 N/mm^2

f_y = 500 N/mm^2

On solving, we get,

A_{st} =2243 mm^2 < $A_{st,min}$

For 25 mm Φ bars,

A_b = 490.87 mm^2

Spacing of bars:

$$S_v = \frac{A_b}{A_{st}} * 1000 = 184.26 \, mm$$

Provide 25 mm Φ bars at 150 mm c/c

$A_{st, provided}$ = 2664 mm^{2+}

$$P_{st} = \frac{A_{st}}{b*d} * 100\% = 0.123\%$$

b) Area of steel at mid span (Top Bars) [IS 456-2000 Annex G-1.1 b]

M = 1725 KNm

On solving, we get,

A_{st} = 1864.6 mm^2 < $A_{st,min}$

For 25 mm Φ bars at 150 mm c/c

$A_{\text{st, provided}} = 2664 \text{ mm}^2$

$P_{\text{st}} = \dfrac{A_s}{b*d} * 100\% = 0.123\%$

7. Check for development length (IS 456:2000 Cl 26.2.1.1)

Bond Stress for M30 concrete (τ_{bd}) = 1.5 N/mm²

For deformed bar (τ_{bd}) = 1.6*1.5 = 2.4 N/mm²

Development length (L_d) = $\dfrac{\Phi*\sigma_s}{4*\tau_{bd}}$ = $\dfrac{25*0.87*500}{4*2.4}$ = $1134 \text{ } mm$

L_o = effective depth (d) or 12 Φ whichever is greater = 1450 mm

$L_d \leq 1.3 * \dfrac{M_l}{v} + L_o$ = $\dfrac{1.3*2070*10^6}{11686*10^3}$ + 1450 = $1680.27 > L_d$ Hence, ok

8. Load Transfer from Column to Footing (IS 456: 2000 Cl 24.4)

Nominal bearing stress in column = $\dfrac{P_u}{A_c}$ = $\dfrac{11686*10^3}{1000*1000}$ = 11.7 N/mm²

Allowable Bearing Stress = 0.45 * f_{ck} = 0.45*30 = 13.5 N/mm² > nominal value

Hence, ok

Footing doesn't require dowel bars. However, the column bars have been extended inside the footing.

9. Design of side reinforcement (IS 456:2000 Cl 34.5.2)

For footing depth greater than 1m, nominal side reinforcement of 360 mm² per m of length is required which should supersede the minimum tensile reinforcement.

$A_{\text{st, min}} = 2664 \text{ mm}^2$

Spacing = $\dfrac{490.87}{2664} * 1000 = 184.26 \text{ } mm$

Provide 25 mm Φ bars @175 mm c/c (4 bars on side)

10. Chain bars (SP 34)

As per Sp-34, suggested spacing of chair bars is 30 times its diameter with at least 12 mm as its diameter. Providing 16 mm Φ chair bars @480 mm c/c.

From manual design

Along X direction, provide 25 mm Φ bars @150 mm c/c at top and 150 mm c/c at bottom

Side reinforcement : 25 mm Φ bars @175 mm c/c

Chair bars : 16 mm Φ bars @480 mm c/cement along X direction. This is supported by following reason:

The Maximum bearing pressure along the Y strip is 368 KN/m^2 which is located for a strip length of 7.5 m.

The Maximum length of the strip is 7.52 m which is not at the location of highest bearing pressure.

Comments:

- Convention rigid foundation design is the design methodology used for manual design. According to IS 2950:(Part I) 1981, the variation of neighbouring column loads and column spacing shouldn't exceed 20% for the division of rafts into perpendicular strips and their analysis as independent beams. This condition does not hold true in our case. Only the longest distance between two columns for each strip in the X direction is used for design.

- The assumption of perfect rigidity in manuals while considering flexibility in the FEM model for reinforcement design can be considered as a limitation of our work.

- The full impact of a basement wall and shear wall's presence is not taken into account in the manual design. However, SAFE takes into account the wall's stiffness and its impact, which results in a variation in design reinforcement.

- Since the plan for foundation is irregular, strips were manually made in SAFE. Irregular in foundation shape makes our manual calculation tedious.

- A 52.2 m high rise building may need a pile foundation due to low soil bearing pressure. According to the project's scope, we estimated the soil's bearing pressure to be 200 KN/m^2, assuming that stabilising the soil would enhance bearing.

14. DUCTILITY AND DUCTILE DETAILING

A ductile material is a material that can be deformed under tensile stress without losing its toughness or strength. When applied to reinforced concrete members and structures, the term "ductility" implies the ability to sustain significant in-elastic deformations prior to collapse. It is the ratio of absolute maximum deformation, curvature or rotation to the corresponding yield deformation. Under reinforced section shows ductile deformation whereas over reinforced section shows brittle deformation, so over reinforced structures should be avoided.

14.1 Significance of Ductility

Ductility is a crucial property in structural design, particularly in the design of structures that will be subjected to tensile forces or deformation. The following are some of the reasons why ductility is significant in structure design:

1. Flexibility: Ductile materials can bend and deform under load without breaking, allowing them to absorb energy and dissipate it over a longer period. This means that they are better able to withstand sudden shock loads or vibrations, such as those from earthquakes or wind gusts.

2. Safety: Ductility ensures that a structure can undergo a degree of deformation without failure. This is particularly important for structures that will be subjected to dynamic loads, as they will not suddenly collapse in the case of sudden loading.

3. Maintenance: Ductility is significant in reducing maintenance costs. It means that structures are less likely to crack or fail under loading, thereby requiring fewer repairs.

4. Cost-Effective: Ductile materials can help reduce the overall cost of a structure. By allowing for some deformation and load-bearing capacity, engineers can design structures that are lighter and use fewer materials, thus reducing the overall cost of construction.

14.2 Variables Affecting Ductility

1. Tension steel ratio
2. Compression steel ratio
3. Shape of Cross-section

4. Lateral Reinforcement

14.3 Design for ductility

Sufficient ductility can be ensured by paying attention to detailing of reinforcement, bar cut offs, splicing and joint details. Following are simple design details:

1. Material selection: The selection of the appropriate material is critical to ensuring ductile behavior. Materials such as mild steel and aluminum are ductile and are commonly used in structural applications.

2. Cross-sectional shape: Structures with circular or oval cross-sections are inherently more ductile than those with square or rectangular cross-sections. Therefore, designing structures with round or oval cross-sections can help enhance ductility. Offsets from beam to column should be avoided.

3. Reinforcement: The addition of reinforcement, such as steel bars or mesh, can improve the ductility of concrete structures. Tensile reinforcement should be restricted, while compressive reinforcement with stirrups should be more

4. Connections: The design of connections between structural members plays an essential role in ensuring ductility. Connections must be designed to allow for some degree of movement and deformation without failure. Connections should be made monolithic.

5. Redundancy: The use of redundant members in a structure can help to ensure that the structure remains stable even if some members fail.

6. Load distribution: The distribution of loads in a structure should be uniform to ensure that all members are subjected to similar stresses. This can help to prevent localized failure and promote ductility. Inelasticity should be confined to beams only and columns should remain elastic.

7. Splices and anchorage should be sufficient to prevent bond failures.

Detailing for Ductility (Based on IS 13920: 2016)

- At least two bars should be provided continuously, both at the top and bottom.
- The positive moment resistance at the face of a joint should not be less than one-half of the negative moment resistance provided at that face of the joint.
- Neither the negative nor the positive resistance at any section along the member should be less than one-fourth of maximum moment of resistance provided at the face of either joint.

Curtailment of Tension Reinforcement in Flexural Members:

The term "curtailment of tension reinforcement" refers to reducing the amount of tension reinforcement in a flexural member (such as a beam or a slab) near the ends of the member. The following are some reasons why curtailment of tension reinforcement may be necessary in the design of flexural members:

1. Preventing premature failure: When a flexural member is subjected to bending, the tensile stresses are highest at the bottom of the member. If the amount of reinforcement is constant along the length of the member, there may be a large concentration of reinforcement at the ends, which can lead to premature failure due to excessive cracking or localized crushing of concrete.

2. Controlling deflection: In some cases, reducing the amount of reinforcement near the ends of a member can help to control deflection. By reducing the stiffness of the member at the ends, the curvature of the member can be more uniform along its length, resulting in reduced deflection.

3. Cost savings: Reducing the amount of reinforcement at the ends of a member can help to reduce the cost of construction, as less reinforcement is required.

The design of the curtailment of tension reinforcement involves determining the appropriate length of the reduced reinforcement zone, as well as the amount of reinforcement to be used in this zone. These parameters are typically determined based on the size and loading of the member, as well as other design considerations such as deflection and cracking.

It is important to note that the curtailment of tension reinforcement must be done carefully to ensure that the member remains safe and structurally sound. The design should be based on sound engineering principles, and appropriate safety factors should be applied to account for uncertainty and variability in the loads and materials.

14.4 FIRE RESISTANCE OF CONCRETE ELEMENTS

The principles employed in the calculations of the fire resistance of structural elements are based on the international research data on the insulating properties of the concrete, strength of concrete and steel reinforcement/ prestressing tendons at high temperatures ($\approx 700°C$) and considerations of such effects as spalling, disposition of reinforcement and the nature of load distribution.

Clause 26.4.3 of IS 456-2000 gives nominal cover to meet a specified period of fire resistance.

The factors that influence the fire resistance of concrete elements are as follows:

a. Size and shape of structural elements
b. Loads distribution
c. Disposition and properties of the reinforcing bars and prestressing tendons
d. Type of aggregate and concrete
e. End conditions
f. Cover to reinforcement

15. DISCUSSION

The project provided us with the opportunity to gain knowledge and practical application of our theoretical knowledge in the particular sector of concrete building design in civil engineering. During the course of the project, we encountered various problems, thereby requiring us to make choices among different alternatives, in cases where we lacked the experience to do so. It required us to get a deeper understanding of the subject from different sources, such as interviews with qualified personnel or searching for appropriate references. We had to deviate from the approaches and methods we learned in our bachelor's coursework to adapt to the needs of the project and solve certain issues. Hence, we have completed the project with the utmost confidence in our own choices taken throughout the project, albeit with some limitations. In academic work, it is considered good practice to mention any unresolved questions, and discuss the limitations of one's own work. We accept and embrace the room for improvement on our part, and discuss some of these aspects in the hope that it will provide further direction for the project groups that may refer to our report in the future to improve upon what has already been explored instead of starting from scratch.

A. **ETABS and its Limitations:**

ETABS is a widely used structural analysis software. The main benefit of the software is that it makes analysis easy and fast. However, the main disadvantage would be the quality of the work produced would be heavily dependent on the users themselves, i.e. our project group in this scenario. Lack of experience of using the software and lack of proper knowledge about the steps to be taken in case of specific problems caused the use of the software to be a much harder task than it should have been.

Limitations of ETABS are:

- Nepal has developed its own earthquake design code for structures, the NBC code. However, such a code has not yet been implemented in ETABS. This caused us to partially rely on the IS code that has been implemented in ETABS, for performance criteria checks of the building.
- Ductile detailing is not completely implemented by ETABS.

- The user interface of ETABS, even though it is a graphical user interface, is still outdated by today's standards. The user interface is unintuitive and learning how to use it requires rather a large amount of time.
- ETABS is not completely capable of modeling curved structures. Although features have been provided for modeling such a structure, during the detailing of curved beams, it was incapable of providing a true representation of the curved beams, replacing such beams by straight beams.

B. SAFE and its limitations

SAFE is a software based on the finite element analysis for the analysis and design of slabs and foundations. Its use has allowed for the analysis of the foundation with ease. Furthermore, its incorporation with ETABS has allowed the users to transfer entire structures with their respective loading from one software to another. However, such a software doesn't come without its limitations:

- SAFE provides us two methods of analyzing a structure, a strip-based method and a finite element method. For the analysis of our structures, we are required to analyze the structure based on strips. For a regular structure, SAFE is capable of developing such strips automatically with the required width, however for irregular structures with members not placed orthogonally, SAFE requires the users to develop the strips manually. The strips so developed require a considerable amount of time for development itself, while only providing the analysis for the area enclosed within the strip.

C. Check Criteria for Input Data

As input of data of the structural quantities and loading pattern is done into ETABS, user error vastly accounts for any discrepancies in the performance of the structure. Therefore, the value should be checked in relation to the total summation of un-factored base reactions, which must be within the permissible limit with the weight of the building calculated from preliminary design. A careful study of frequencies and mode shape provides helpful information on the correctness of the input data.

D. Effect of Shear wall on Inter Story Drift:

We compare the effect of shear wall on drift evaluation. We observe the contribution of the shear walls provided in damping the interstory deflections of the entire structure as mentioned in the IS 13920:2016, according to which the moment resisting frame should contribute to at least 25% of the entire moment.

E. Static analysis vs. Dynamic analysis:

The major advantage of using the forces obtained from a dynamic analysis as the basis for a structural design is that the vertical distribution of forces may be significantly different from the forces obtained from equivalent static load analysis. Consequently, the design becomes more earthquake resistant. The current code allows the results obtained from a dynamic analysis to be normalized so that maximum dynamic base shear is equal to the base shear obtained from a simple two-dimensional static load analysis.

Height of Building	= 43.2
Building	= Irregular
Seismic Zone	= V

So, Dynamic analysis was to be performed.

Static Base Shear, VB_{sx}	=21245 KN
Static Base Shear, VB_{sy}	=21245 KN
Dynamic Base Shear, VB_{dx}	= 11308KN
Dynamic Base Shear, VB_{dy}	= 10741 KN
Scale factor in X-direction	=1.864
Scale factor in Y-direction	=1.96

F. Shear Wall Modelling:

Shear walls were modeled as shell elements and columns are line elements. For shear walls connected to columns, strain compatibility requires the FEM nodes in column and connected edge of shear wall to be rigidly connected. So, the wall is modeled from the centerline of the columns. This ensures that deformations are compatible, and forces are transferred properly. But the location of the wall would be slightly offset from its actual position. This offset in position implies that the stiffness center of the shear wall is also slightly offset. This may affect the position of the center of stiffness, forces in shear wall and other members. Another choice could be to model the shear wall from the center of the column at one end to the actual position at another end. This however means that the length of the wall is longer in model than what would actually be provided. This implies that the model considers greater stiffness than actually present, which in turns underestimates drifts, and shear forces in other lateral load resisting elements i.e. columns.

Actual Positions

Modeled Positions

Column   Shear Wall

(Area Element)   (Line Element)
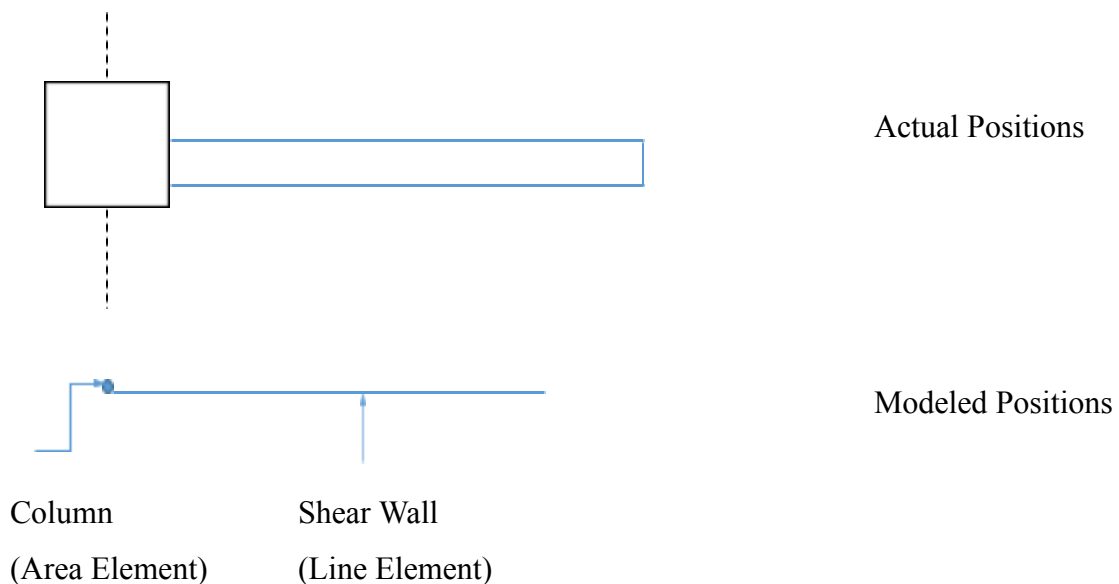
Figure 15.1: Shear Wall Model vs Real Position

Modeling was done as shown in the figure above. The effect of shift in the stiffness center of the wall was not rigorously checked. And we assumed it was not much relevant as the total stiffness is not affected but rather only its location.

G. Use of Shear Walls

Shear walls have been modeled as shell elements. The use of shear walls were primarily focused on the lift cores. However, such use caused a large amount of eccentricity in the structure. Therefore, in order to account for such eccentricity, shear walls have been placed symmetrically in the entire building. However, our inexperience in placement of shear walls required us to use a hit and trial method for the purpose, while taking into consideration the basic rules for the placement of shear walls. Symmetricity has been maintained as much as possible, while the shear walls are placed in the outer walls where possible. The shear walls used are of varying thickness as per their requirements. Such use of shear walls has allowed us to develop a building with a low time period even though it is irregular. Hence, the design of the structure is conservative.

H. Use of Curved Structural Elements

The presence of curved beams in the structure provided a unique problem in the design of the entire structure. The tools provided in the software allowed for the modeling of such a curved structure i.e. curved beams and curved slabs. However, such curved structures provided their own problems. The presence of such structures caused concentration of forces at such areas. The straight structures in the same areas were seen to be more capable of withstanding higher loads for the same cross sections. The curved sections also provided for the irregularity of the structure, to accommodate for which greater structural rigidity was required to be provided. The incorporation of the curved nature in the ramp slabs required breaking the entire span of the ramp slab into shorter spans for proper representation of the ramp slab in the software.

I. Bearing Capacity of the Soil

The bearing capacity of the soil at the site has not been provided. Hence, in order to determine its bearing capacity, appropriate assumptions were made based on the average bearing capacity of the valley and the specific locations, based on the values obtained from the literature review. The assumptions made have been clearly identified and presented in such cases.

J. Use of Mat Foundation

For a large structure, the use of pile foundations are preferred as the mat foundations are incapable of resisting the deflection caused by such a large structure. However, the pile foundations are out of our scope, which has somewhat limited our ability to develop an economical foundation. The use of an extremely deep mat foundation, along with large projections were required in order to develop a safe structural foundation. Furthermore, the average bearing capacity of the soil was incapable of withstanding such a large structure, which required us to use a larger bearing capacity. Such a large bearing capacity however, is obtainable through methods such as soil stabilization despite having its own economic drawbacks.

K. Effect of Using Torsional Multiplier

A beam element is incapable of transferring torsion. Therefore, during its modeling, analysis and design, a torsional modifier is required to allow the beam element to represent its theoretical property. When a small value of torsional modifier is used in a beam model, the torsional stiffness of the beam is reduced during analysis which results in any compatibility torsional moment to be distributed to other beams and members as bending moment, torsional moment or shear force. This results in lower torsion value in design and thus, transverse reinforcement demand is less. Torsional moment is resisted by the beam's concrete to some extent until concrete cracks. This value is called the cracking moment of resistance for torsion. After that, the shear reinforcement provided can help in resisting torsional moments. If in analysis and design torsion is removed, shear reinforcement provided won't resist any more torsion than it was designed for. Thus, the provision of lesser shear stirrups in turns supports the modeling assumption of smaller torsional capacity. Care must be taken to not reduce the torsional stiffness of the beam which is required to sustain equilibrium torsion. For example,

Reducing or not reducing torsion affects the following parameters for design:

1. Torsion Moment on the beam is reduced. Hence, shear stirrup required to resist that torsion is reduced. In some cases, the beam section size would be insufficient to resist the torsion and then an increase in beam section would be necessary. Thus, reducing torsional

moment has benefits in terms of economy of shear reinforcement and smaller section sizes.

2. For the torsional moment to be redistributed, the beam must twist by at least some angle; this means cracks would be developed. However, the minimum shear reinforcement requirements

Design codes can be assumed to be sufficient to take care of such cracks.

3. Bending moment in other beams can increase, which necessitates greater longitudinal reinforcement. Also, the bending moment in slab can increase which similarly necessitates higher longitudinal reinforcement.

16. CONCLUSION

During the course of the project various problems were encountered and dealt under the guidance of our supervisor Assoc. Prof. Dr. Kshitij Charana Shrestha.

Historical data and seismological studies have indicated that the entire region of Nepal lies in an active seismic zone and is prone to earthquakes. Due to the frequent earthquakes that strikes Nepal every year, earthquake resistant design seems to be a most important aspect in infrastructure development. Hence, we completed this project abiding by the earthquake resistant design codes (NBC 105:2020 and IS 1893 (Part-I):2016) and ductile detailing of concrete (IS 13920:2016), which makes the buildings more ductile and earthquake resistant. The use of such building codes provided us with an opportunity for a comparison of the respective building codes and their appropriateness in their application in respective scenarios. This building project provided us with a general idea regarding analysis with an earthquake and response spectrum, earthquake resistant design, and ductile detailing that needs to be done in order to ensure safety for both structures and human life during an earthquake. It helped us understand the mechanism of the transfer of lateral earthquake load into vertical members and finally to the foundation.

The design included both two-way and one-way slabs. The incorporation of large multiple lift cores and the change in orientation of the structure, caused the structure to be irregular, which in itself came with its own difficulties. The irregularities required the structure to be designed through the design and placement of shear walls in an asymmetrical way, which contrasted with our understanding of it obtained through books. We learned to design circular and square columns as well as the interrelationship of the beams and the columns. The total size of footing being more than 50% of the plinth area required the usage of Raft Foundation, which was done so incorporating the need for the basement area to be used as parking acting as a single block. The basement wall was designed as a propped cantilever wall with fixity at the mat foundation and lateral restraints at top. With the collective knowledge of our team, we idealized, analyzed and designed the building. We hope this design meets the expectations of the respected supervisor, rest of the teachers and friends, who helped us directly or indirectly during the project work. In spite of constant supervision and incessant hard work of every team member, errors might have transpired. We have tried to minimize error and are open to suggestions if any.
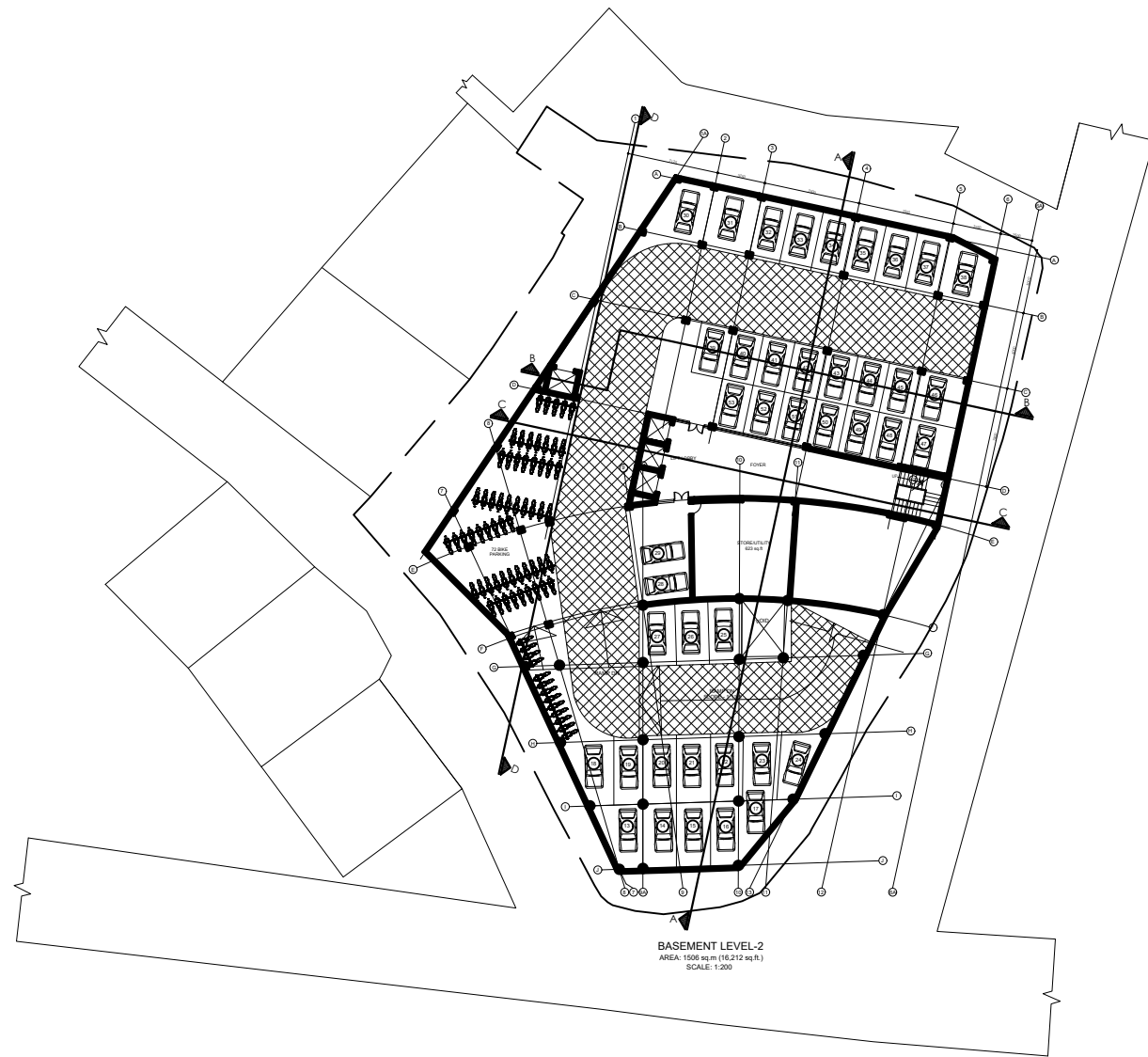
17. LIMITATIONS

1. The bearing capacity of soil at the site is not available, so the bearing capacity of soil is taken by referring to the papers available for the site.

2. Soil-structure interaction has not been taken into consideration. The structure has been supposed to be fully fixed at the foundation level, which may not represent the real soil-structure interaction.

3. The contribution of the infill walls has not been considered in the design of the building, where they have been considered as a non structural element. As a structural element, they affect the stiffness of the building as a structure itself, primarily causing irregularities in the structure such as weak storey and soft storey.

4. The building is designed using the elastic properties of the structural members. In order to account for the non-linearity, the response reduction factor has been included. However, the non-linearity at the material level has not been considered in our design.

5. The building is not designed to meet the economic aspect either, which is beyond our scope.

6. A constraint of our study is the assumption of perfect rigidity in stability checks while considering flexibility in the FEM model for reinforcement design.

7. A 52.2 m high rise building may need a pile foundation due to low soil bearing pressure. Binding to the scope of the project, we designed a raft foundation, assuming that stabilizing the soil through mechanical, chemical, and soil compaction stabilizing techniques would enhance bearing .

18. REFERENCES

1) IS 456: 2000, Indian Standard Code of Practice for Plain and Reinforced Cement Concrete, 4th revision, Bureau of Indian Standards, New Delhi.

2) IS 875(Part 1):1987, Indian Standard Code of Practice for Design Loads (Other than earthquake) for Buildings and Structures, Part 1- Dead Loads, Indian Standards Institution.

3) IS 875(Part 2):1987, Indian Standard Code of Practice for Design Loads (Other than earthquake) for Buildings and Structures, Part 2- Imposed Loads, Indian Standards Institution.

4) IS 13920:2016, Indian Standard Code of Practice for Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces, Bureau of Indian Standards, New Delhi, p. 14.

5) IS 1893(Part 1):2002, Indian Standard Criteria for Earthquake Resistant Design of Structure, Part 1- General Provisions and Buildings, Indian Standards Institution.

6) SP 16:1980, Design Aids for Reinforced Concrete to IS 456:1978, Bureau of Indian Standards, New Delhi, p. 232.

7) SP 34:1987, Handbook on Concrete Reinforcement and Detailing, Bureau of Indian Standards, New Delhi, p.192.

8) IS 1904:1988, Indian Standard Code of Practice for Design and Construction of Foundations in soils : General Requirements

9) IS 2905(Part I)-1981, Indian Standard for Design and Construction of Raft foundations: Part I Design

10) Pillai, S.U. and Menon, D. (2009), Reinforced Concrete Design, 3rd edition, Tata McGraw Hill Publishing Company Ltd, New Delhi.

11) Subramanian N. (2013), Design of Reinforced concrete structures, 1st edition, Oxford University Press, New Delhi.

12) Clough, R.W. and Penzien, P., Dynamics of Structure-Second Edition, CBS Publisher and Distributors Pvt. Ltd., New Delhi-2015.

13)Jain, A.K., Reinforced Concrete Structures Limit State Design- Sixth Edition, Nemchand Brothers and Sons' Publisher.

14)Bhavikatti, S.S.,(2016), Design of Reinforced Concrete Elementary Structures-Second Edition, New Age International Publishers,New Delhi.

BASEMENT LEVEL-2
AREA: 1506 sq.m (16,212 sq.ft.)
SCALE: 1:200

BASEMENT LEVEL -1
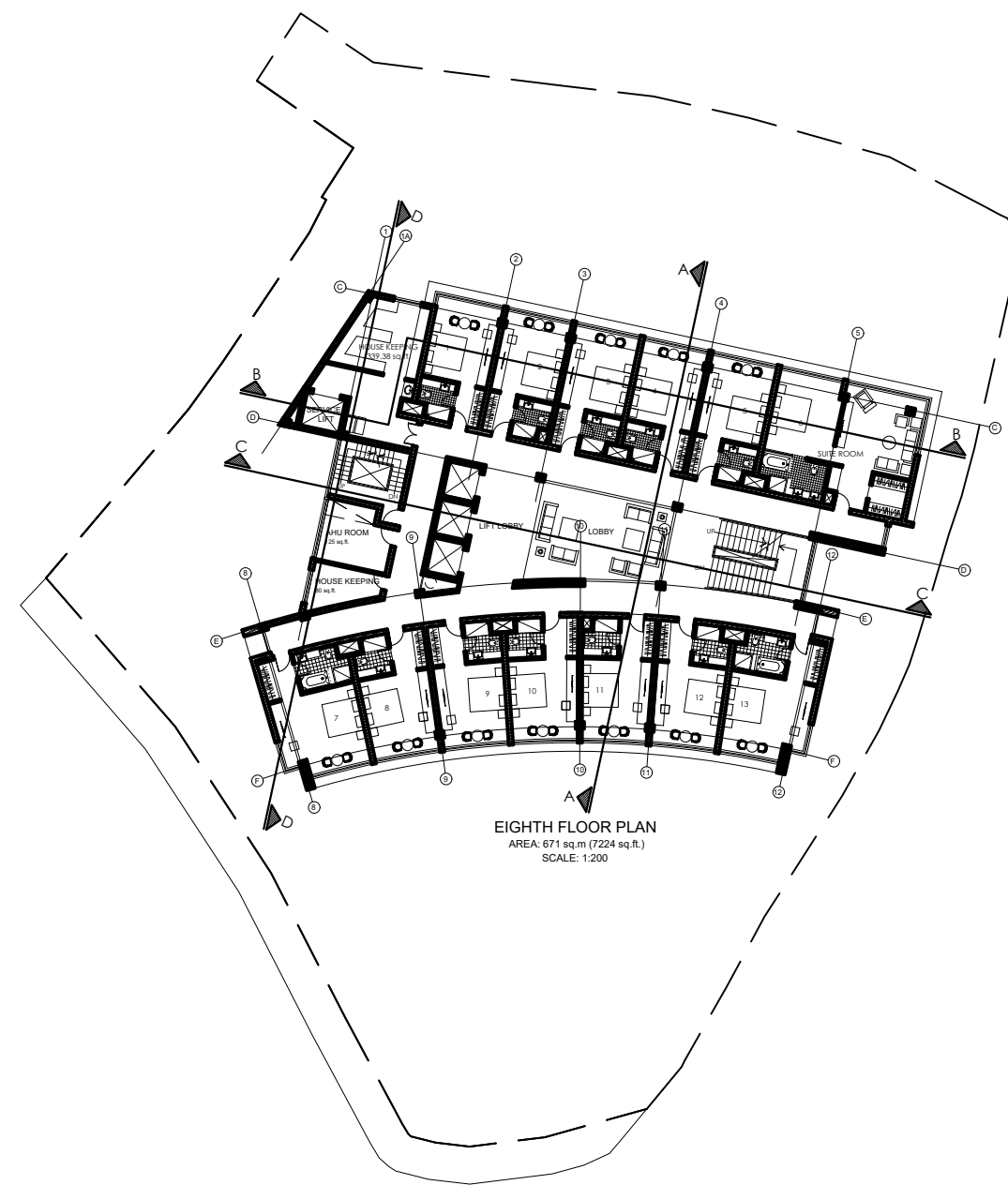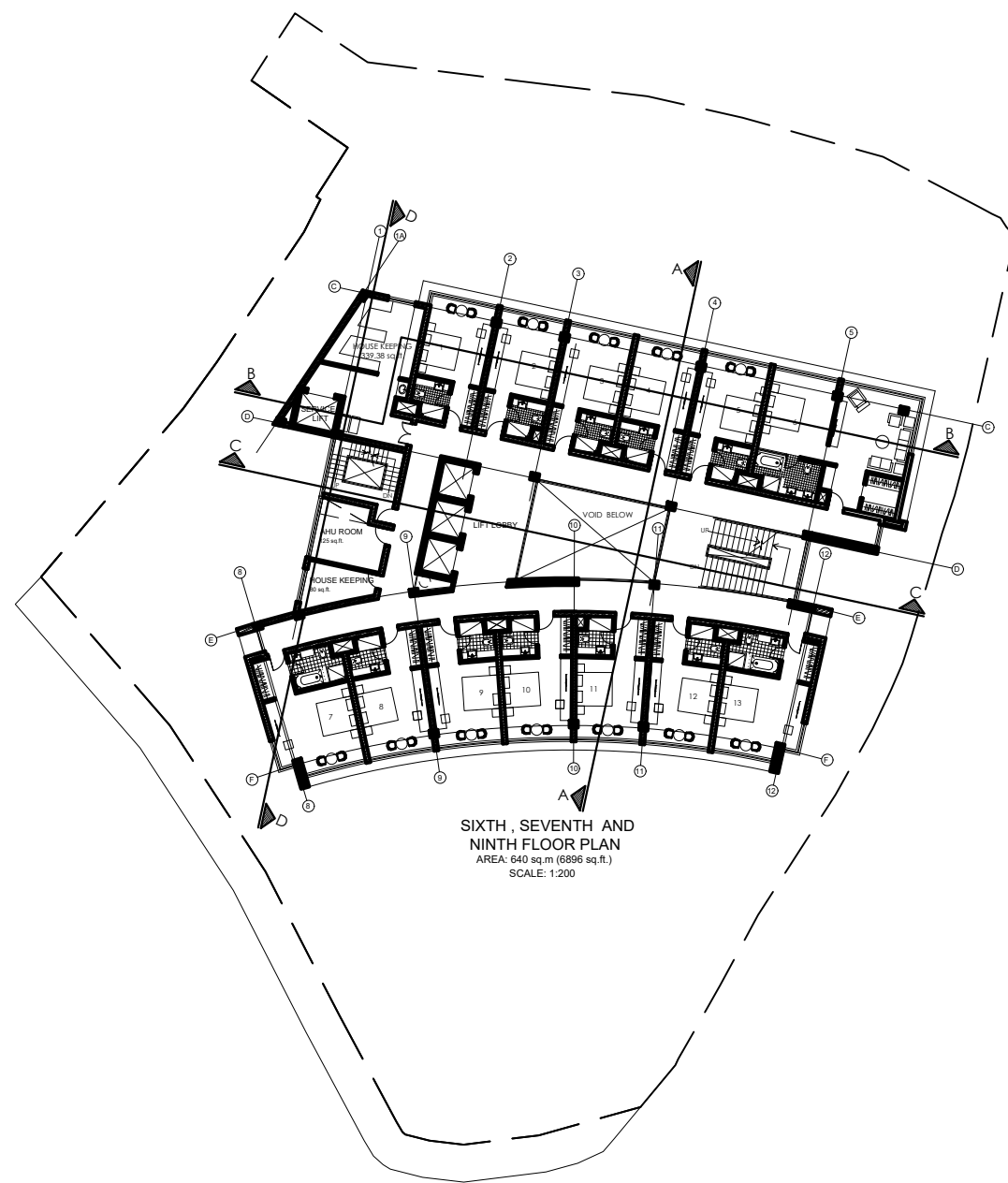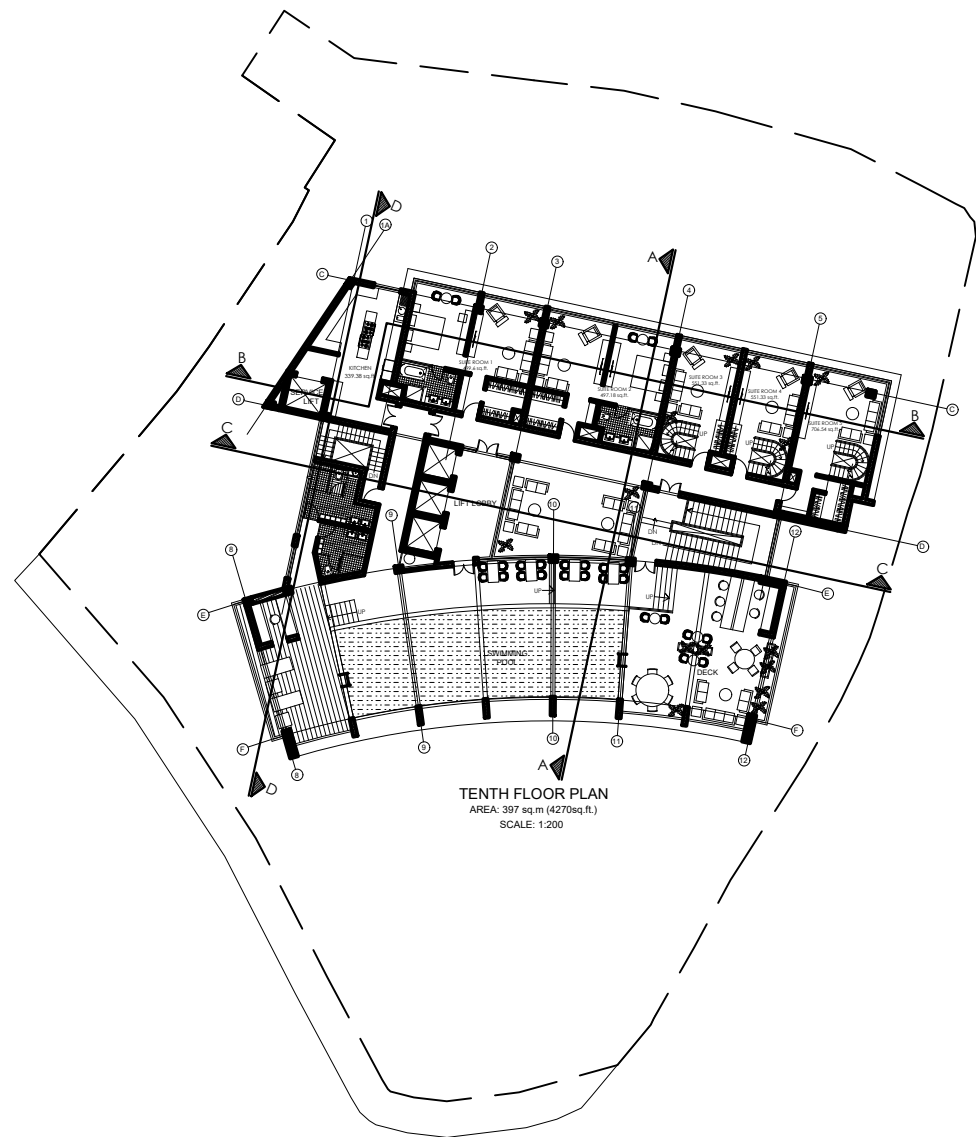AREA: 1611 sq.m (17,338 sq.ft.)
SCALE: 1:200

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **"Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"**	Sheet Title: **Floor Plans**	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		
			Abhinav Shrestha	(075BCE009)		Sheet No:
			Akhilesh Prasad Gupta	(075BCE011)		**1**
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

SEMI-BASEMENT FLOOR PLAN
AREA: 636 sq.m (6844 sq.ft.)
SCALE: 1:200

GROUND FLOOR PLAN
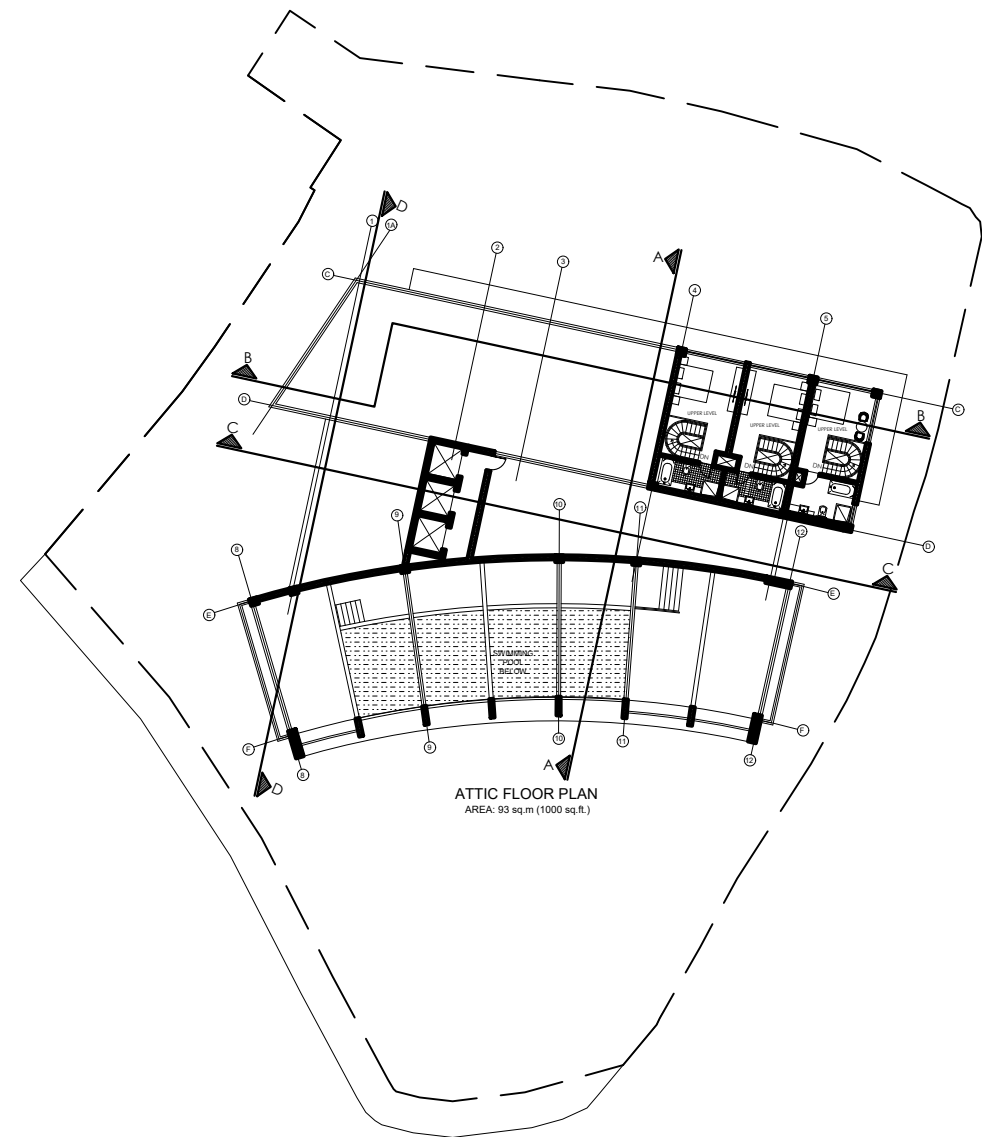AREA: 557 sq.m (6001 sq.ft.)
SCALE: 1:200

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Floor Plans	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		
			Abhinav Shrestha	(075BCE009)		Sheet No:
			Akhilesh Prasad Gupta	(075BCE011)		2
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

FIRST FLOOR PLAN
AREA: 543 sq.m (6842 sq.ft.)
SCALE: 1:200

LVL.±0000

MEZZANINE FLOOR PLAN
AREA: 645 sq.m (6943 sq.ft.)
SCALE: 1:200

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Floor Plans	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		Sheet No:
			Anil Chapagain	(075BCE018)		3
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

SECOND FLOOR PLAN
AREA: 671 sq.m (7224 sq.ft.)
SCALE: 1:200

THIRD FLOOR PLAN
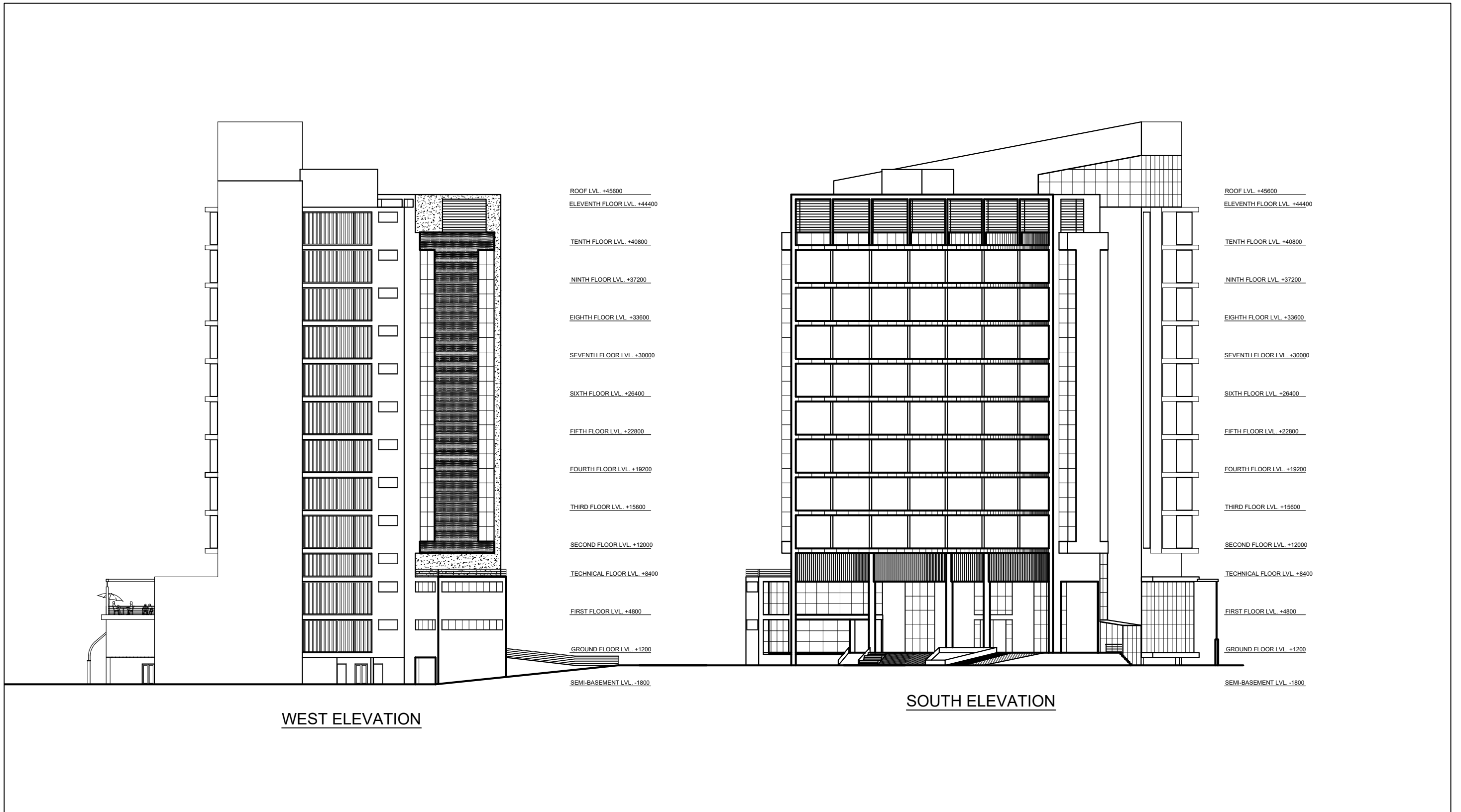AREA: 640 sq.m (6896 sq.ft.)
SCALE: 1:200

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **"Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"**	Sheet Title: **Floor Plans**	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		
			Abhinav Shrestha	(075BCE009)		Sheet No:
			Akhilesh Prasad Gupta	(075BCE011)		**4**
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

FOURTH FLOOR PLAN
AREA: 640 sq.m (6896sq.ft.)
SCALE: 1:200

FIFTH FLOOR PLAN
AREA: 546 sq.m (6955 sq.ft.)
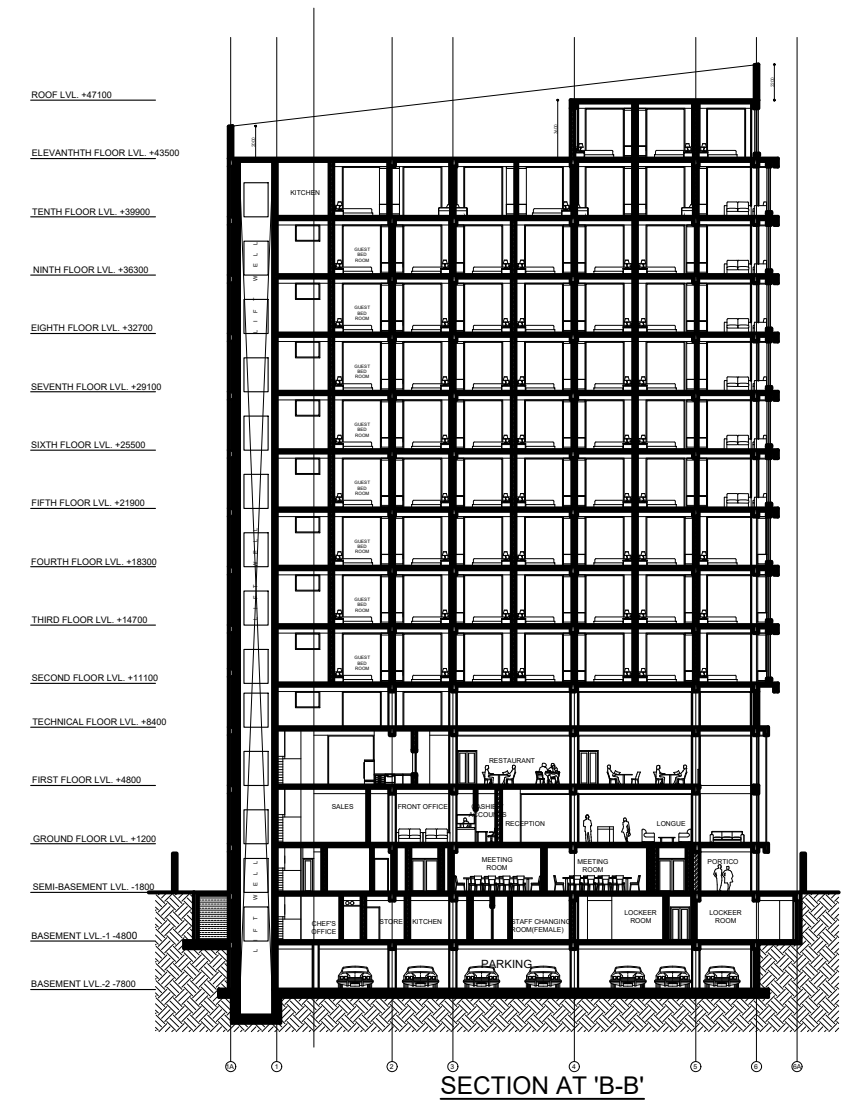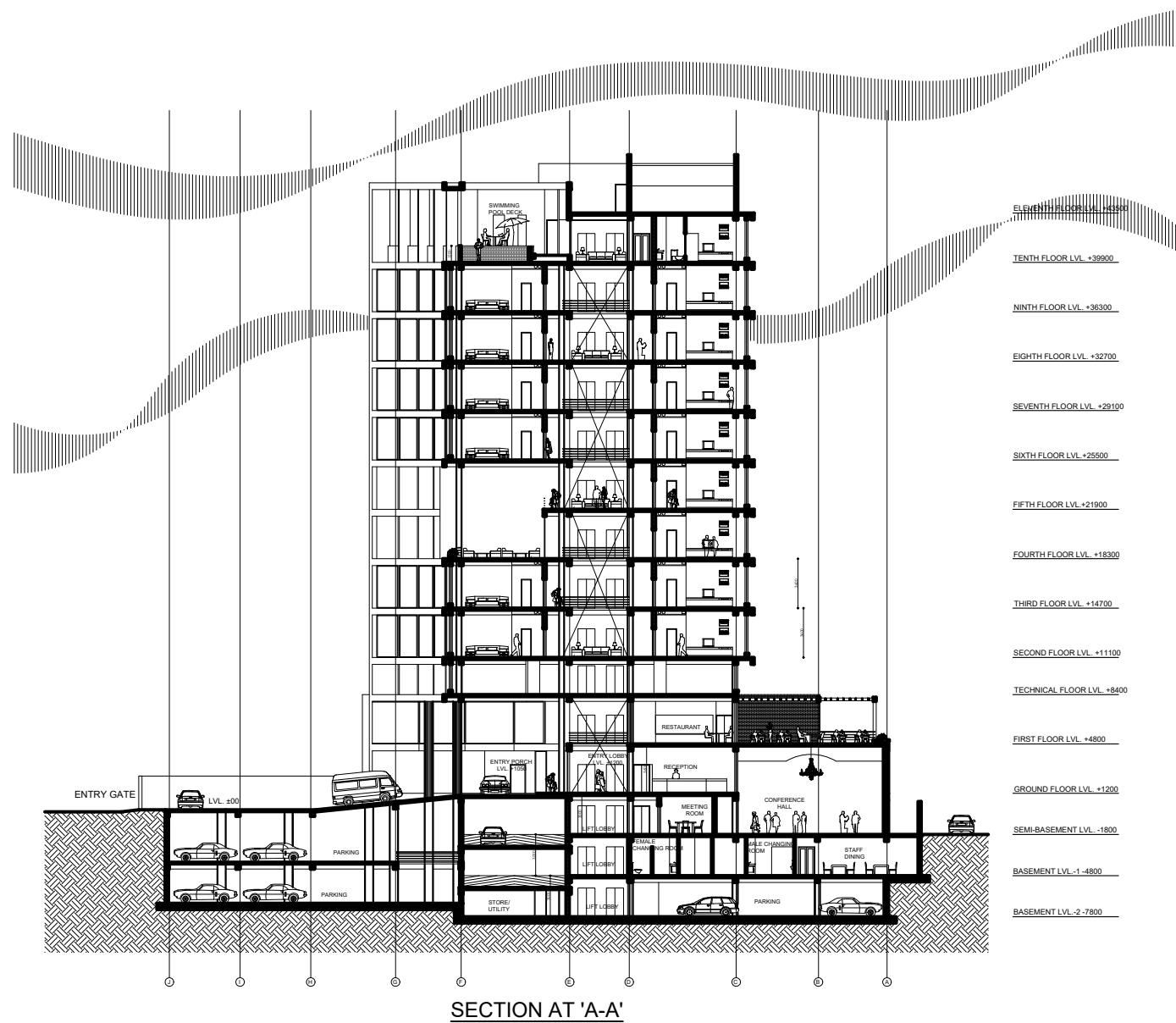SCALE: 1:200

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **"Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"**	Sheet Title: **Floor Plans**	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		
			Abhinav Shrestha	(075BCE009)		Sheet No:
			Akhilesh Prasad Gupta	(075BCE011)		**5**
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

SIXTH , SEVENTH AND
NINTH FLOOR PLAN
AREA: 640 sq.m (6896 sq.ft.)
SCALE: 1:200

EIGHTH FLOOR PLAN
AREA: 671 sq.m (7224 sq.ft.)
SCALE: 1:200

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **"Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"**	Sheet Title: **Floor Plans**	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		
			Abhinav Shrestha	(075BCE009)		Sheet No:
			Akhilesh Prasad Gupta	(075BCE011)		**6**
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

TENTH FLOOR PLAN
AREA: 397 sq.m (4270sq.ft.)
SCALE: 1:200

ATTIC FLOOR PLAN
AREA: 93 sq.m (1000 sq.ft.)

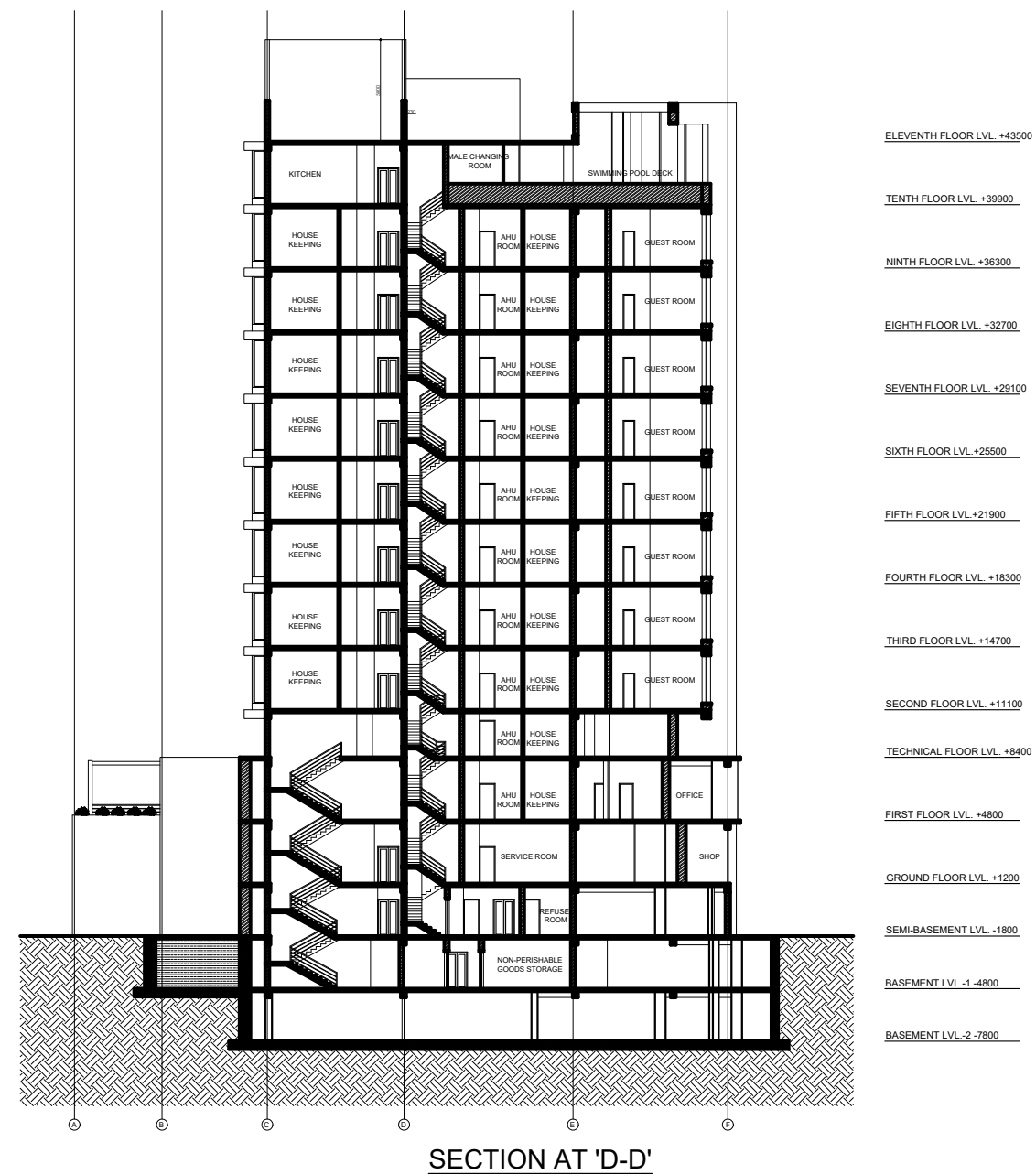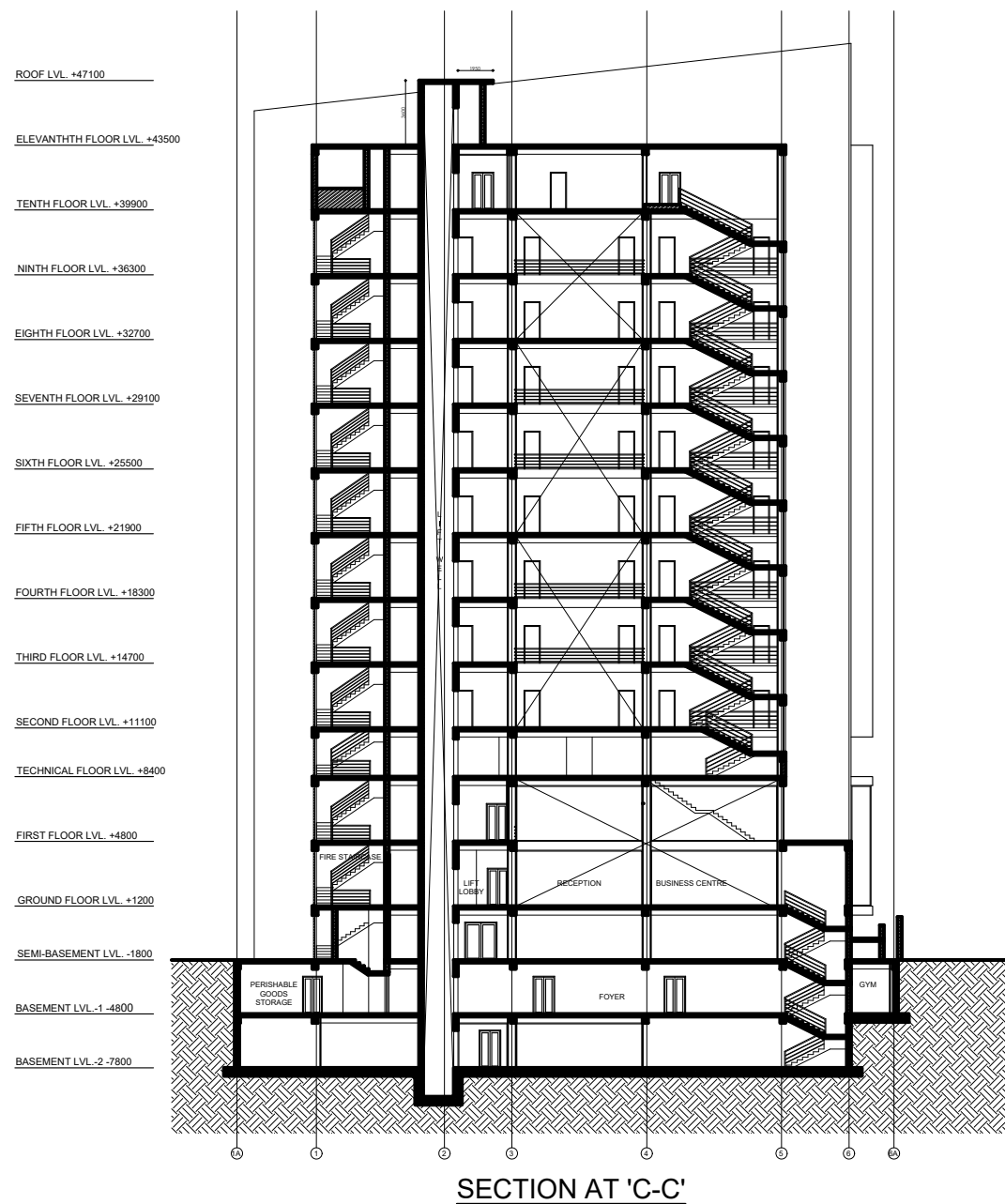| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **"Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"**	Sheet Title: **Floor Plans**	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		
			Abhinav Shrestha	(075BCE009)		Sheet No:
			Akhilesh Prasad Gupta	(075BCE011)		7
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

BACK ELEVATION

EAST ELEVATION

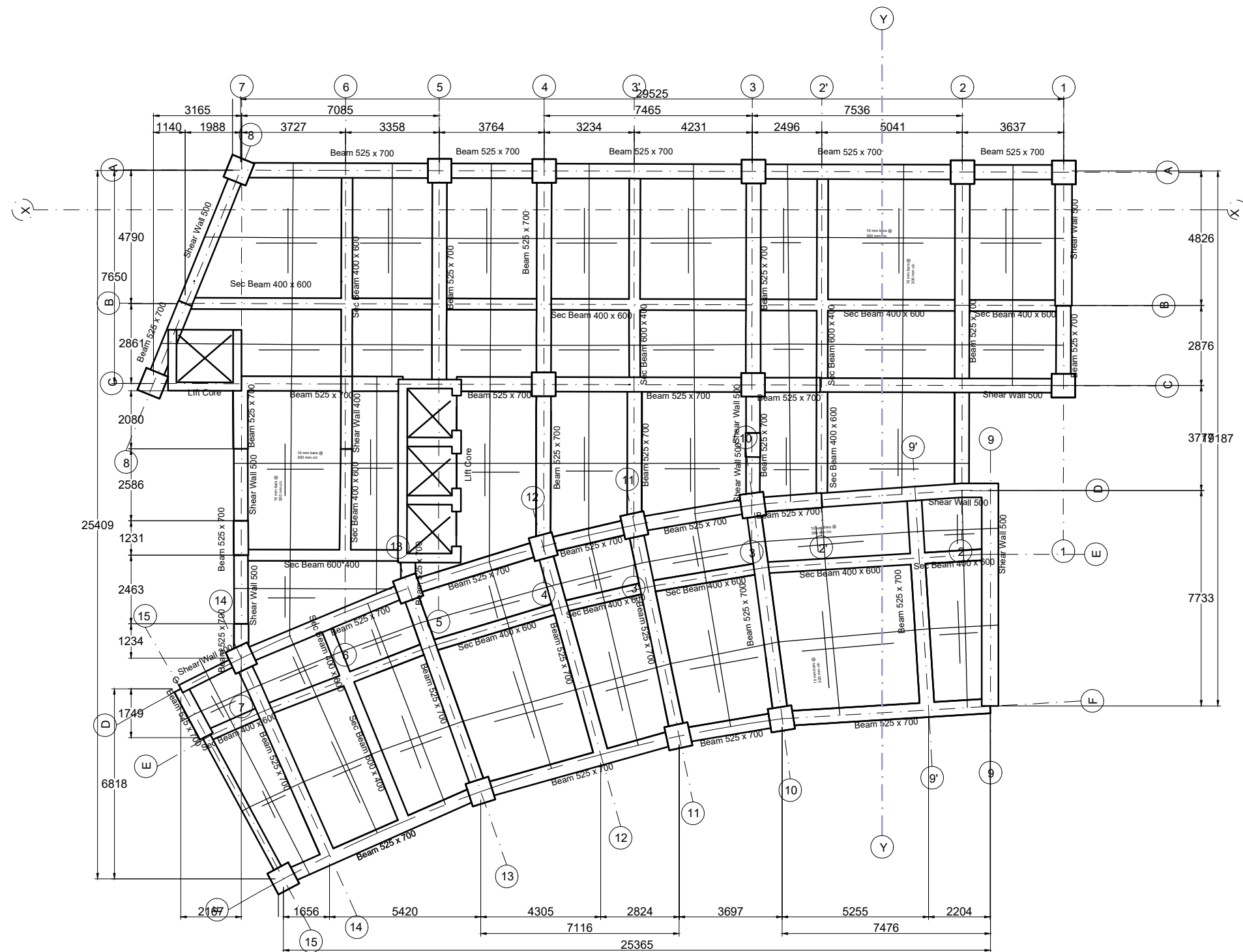| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Elevations	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota Abhinav Shrestha Akhilesh Prasad Gupta Anil Chapagain Shreeya Khanal Sophiya Shrestha	(075BCE004) (075BCE009) (075BCE011) (075BCE018) (075BCE158) (075BCE165)		Sheet No: 8

ROOF LVL. +45600
ELEVENTH FLOOR LVL. +44400
TENTH FLOOR LVL. +40800
NINTH FLOOR LVL. +37200
EIGHTH FLOOR LVL. +33600
SEVENTH FLOOR LVL. +30000
SIXTH FLOOR LVL. +26400
FIFTH FLOOR LVL. +22800
FOURTH FLOOR LVL. +19200
THIRD FLOOR LVL. +15600
SECOND FLOOR LVL. +12000
TECHNICAL FLOOR LVL. +8400
FIRST FLOOR LVL. +4800
GROUND FLOOR LVL. +1200
SEMI-BASEMENT LVL. -1800

WEST ELEVATION

ROOF LVL. +45600
ELEVENTH FLOOR LVL. +44400
TENTH FLOOR LVL. +40800
NINTH FLOOR LVL. +37200
EIGHTH FLOOR LVL. +33600
SEVENTH FLOOR LVL. +30000
SIXTH FLOOR LVL. +26400
FIFTH FLOOR LVL. +22800
FOURTH FLOOR LVL. +19200
THIRD FLOOR LVL. +15600
SECOND FLOOR LVL. +12000
TECHNICAL FLOOR LVL. +8400
FIRST FLOOR LVL. +4800
GROUND FLOOR LVL. +1200
SEMI-BASEMENT LVL. -1800

SOUTH ELEVATION

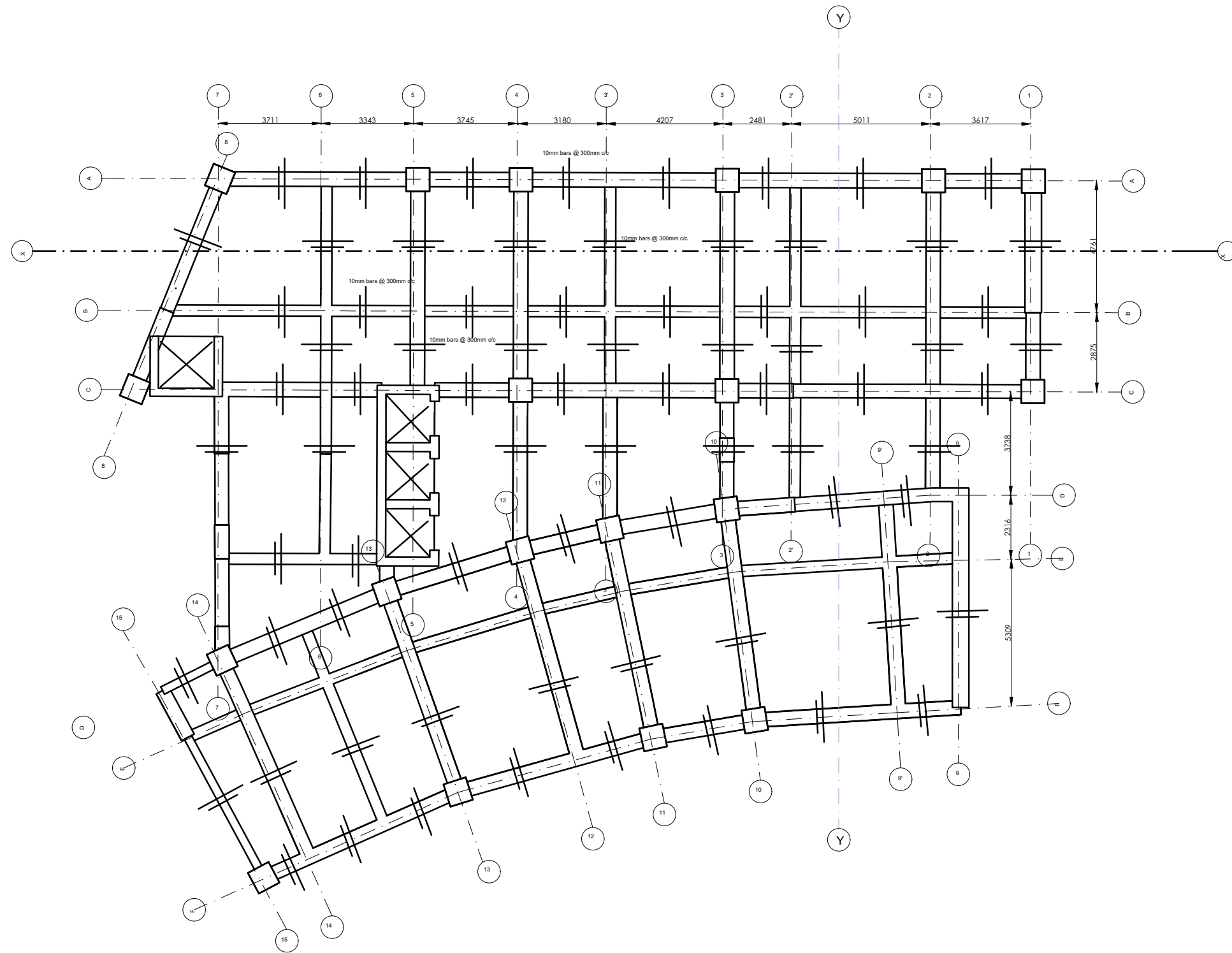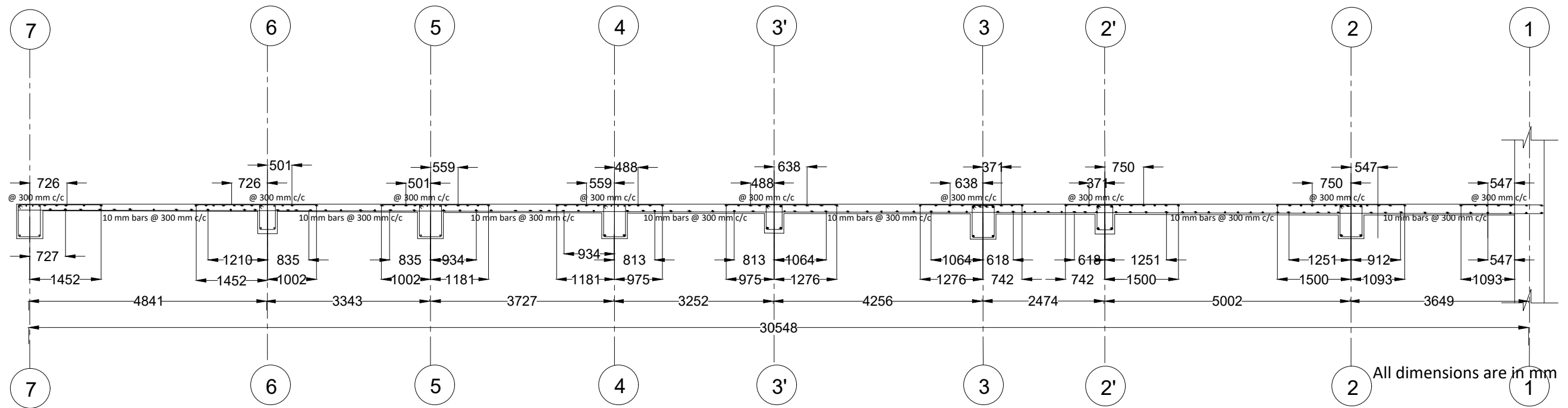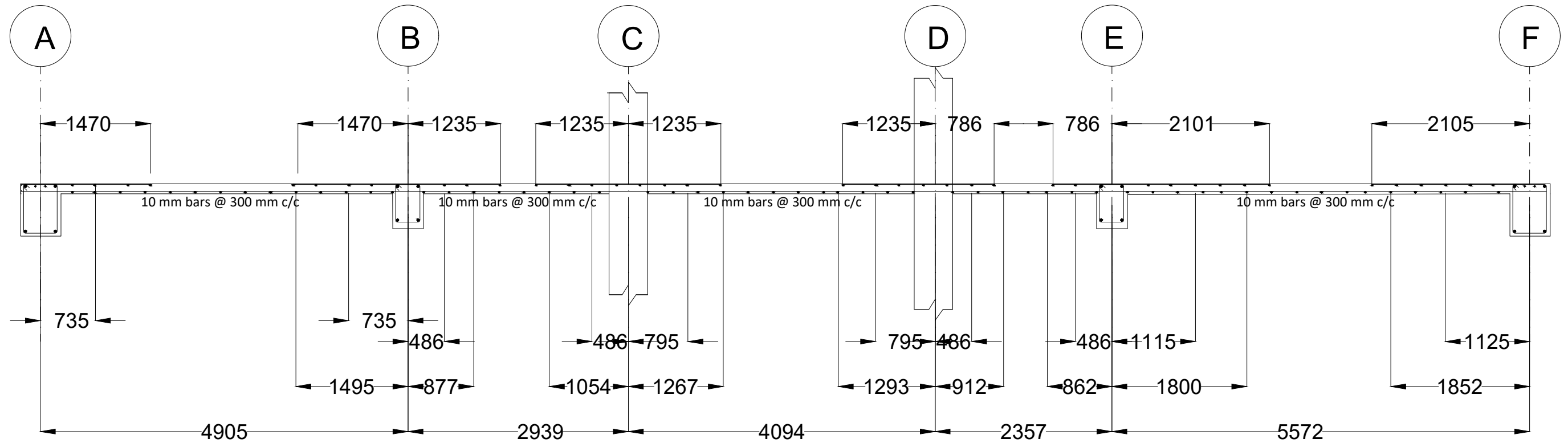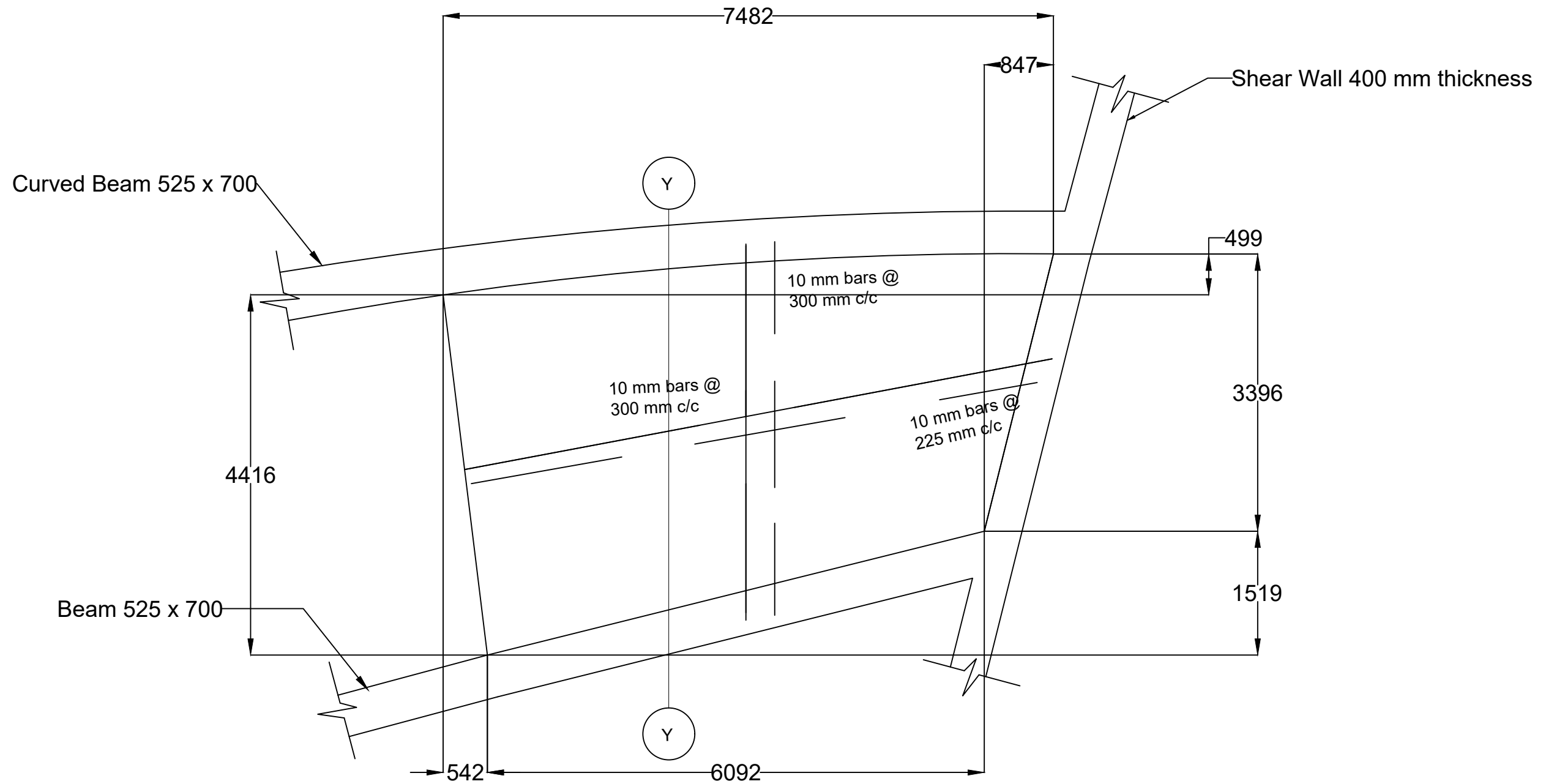| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Elevations	Group Members: Aashish Sapkota (075BCE004) Abhinav Shrestha (075BCE009) Akhilesh Prasad Gupta (075BCE011) Anil Chapagain (075BCE018) Shreeya Khanal (075BCE158) Sophiya Shrestha (075BCE165)	Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
					Sheet No: 9

SECTION AT 'A-A'

SECTION AT 'B-B'

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Sections	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		Sheet No: 10
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

SECTION AT 'C-C'

SECTION AT 'D-D'

Left elevation levels:
ROOF LVL. +47100
ELEVANTHTH FLOOR LVL. +43500
TENTH FLOOR LVL. +39900
NINTH FLOOR LVL. +36300
EIGHTH FLOOR LVL. +32700
SEVENTH FLOOR LVL. +29100
SIXTH FLOOR LVL. +25500
FIFTH FLOOR LVL. +21900
FOURTH FLOOR LVL. +18300
THIRD FLOOR LVL. +14700
SECOND FLOOR LVL. +11100
TECHNICAL FLOOR LVL. +8400
FIRST FLOOR LVL. +4800
GROUND FLOOR LVL. +1200
SEMI-BASEMENT LVL. -1800
BASEMENT LVL.-1 -4800
BASEMENT LVL.-2 -7800

Right elevation levels:
ELEVENTH FLOOR LVL. +43500
TENTH FLOOR LVL. +39900
NINTH FLOOR LVL. +36300
EIGHTH FLOOR LVL. +32700
SEVENTH FLOOR LVL. +29100
SIXTH FLOOR LVL. +25500
FIFTH FLOOR LVL.+21900
FOURTH FLOOR LVL. +18300
THIRD FLOOR LVL. +14700
SECOND FLOOR LVL. +11100
TECHNICAL FLOOR LVL. +8400
FIRST FLOOR LVL. +4800
GROUND FLOOR LVL. +1200
SEMI-BASEMENT LVL. -1800
BASEMENT LVL.-1 -4800
BASEMENT LVL.-2 -7800

Room labels (Section C-C): FIRE STAIRCASE, LIFT LOBBY, RECEPTION, BUSINESS CENTRE, PERISHABLE GOODS STORAGE, FOYER, GYM

Room labels (Section D-D): KITCHEN, MALE CHANGING ROOM, SWIMMING POOL DECK, HOUSE KEEPING, AHU ROOM, GUEST ROOM, SERVICE ROOM, OFFICE, SHOP, REFUSE ROOM, NON-PERISHABLE GOODS STORAGE

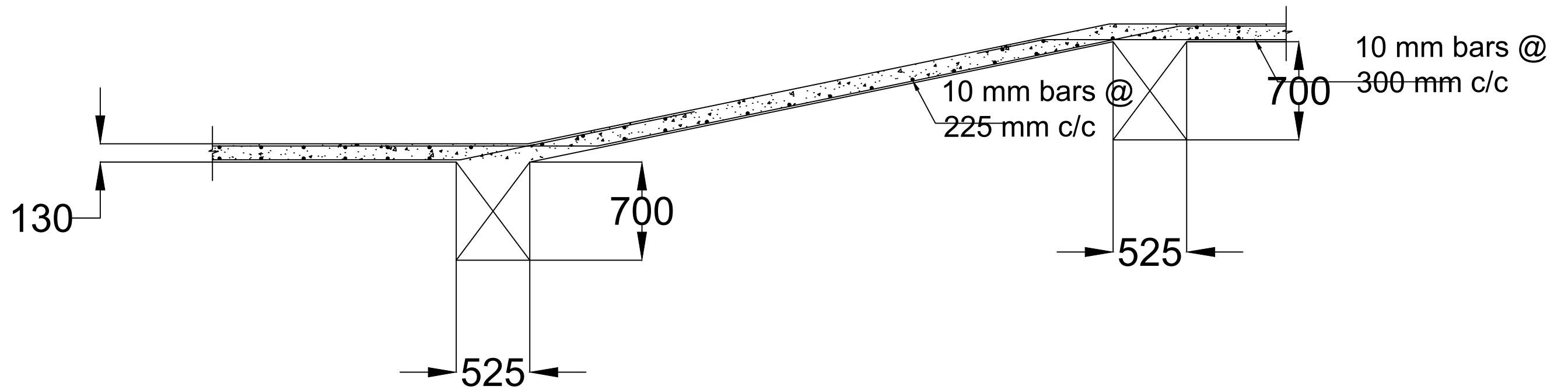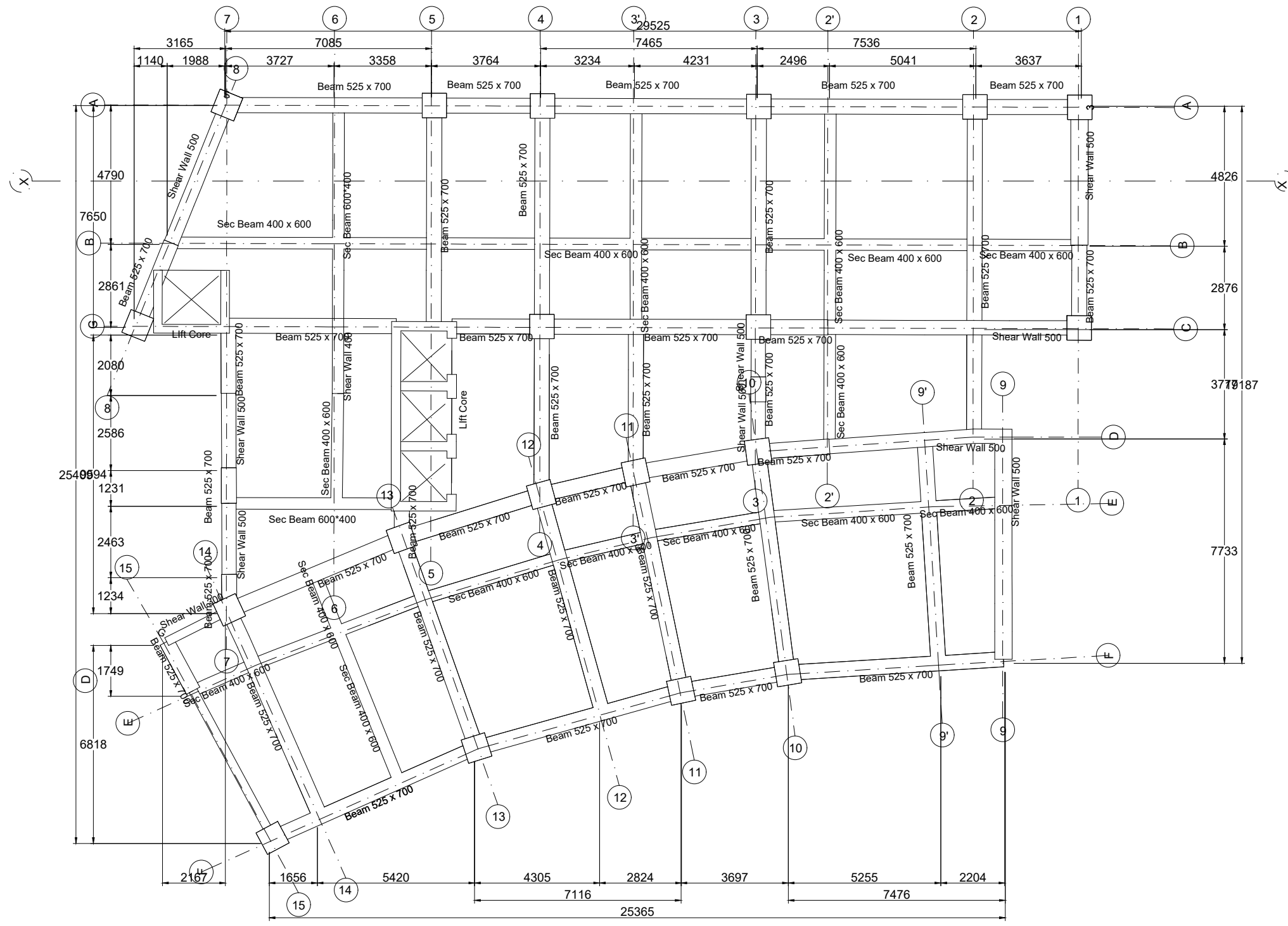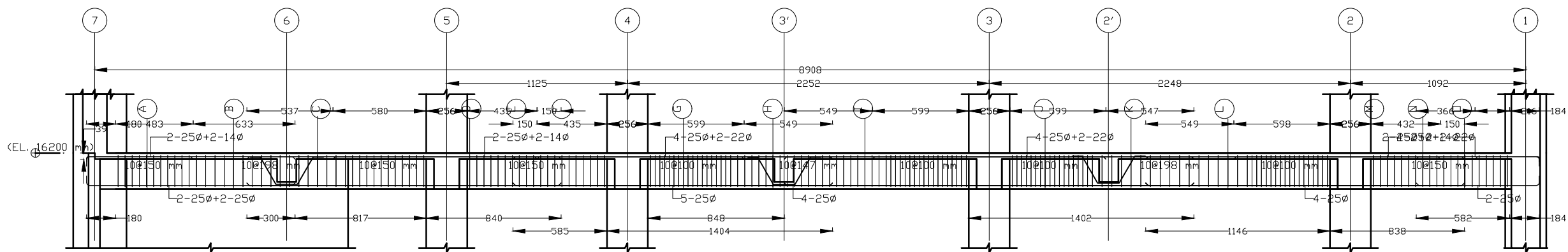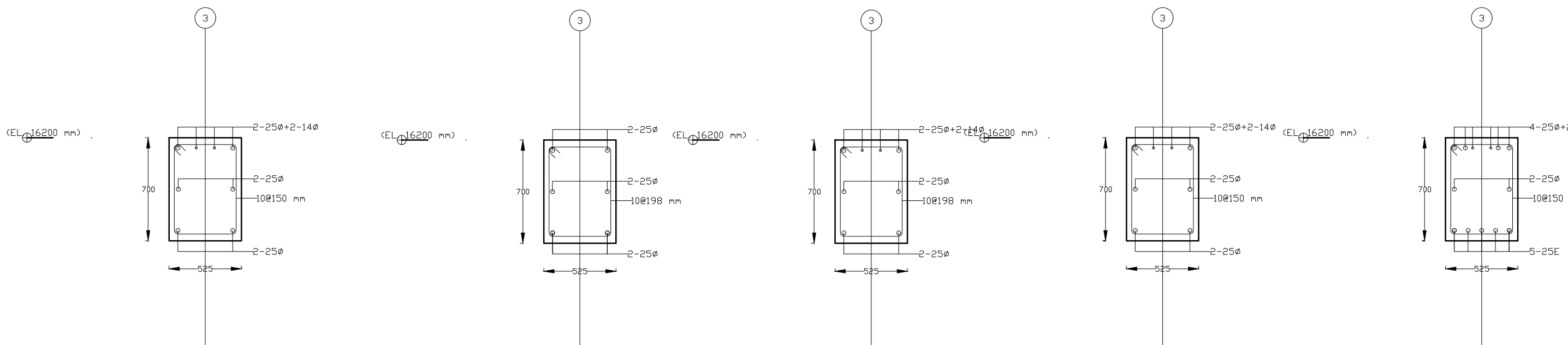| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Sections	Group Members:	Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota (075BCE004) Abhinav Shrestha (075BCE009) Akhilesh Prasad Gupta (075BCE011) Anil Chapagain (075BCE018) Shreeya Khanal (075BCE158) Sophiya Shrestha (075BCE165)		Sheet No: 11

2nd Floor Bottom Reinforcement Plan

Drawing labels and dimensions (as visible on the structural plan):

Grid labels (top): 7, 6, 5, 4, 3', 3, 2', 2, 1
Grid labels (right): A, B, C, D, E, F
Dimension line (top): 29525

Top dimensions: 3165 | 7085 | 3764 | 3234 | 4231 | 2496 | 7536 | 3637
Sub-dimensions: 1140 | 1988 | 3727 | 3358 | 7465 | 5041 | 7536

Right-side dimensions: 4826 | 2876 | 3779 | 9187 | 7733

Left-side dimensions: 4790 | 7650 | 2861 | 2080 | 2586 | 25409 | 1231 | 2463 | 1234 | 1749 | 6818

Bottom dimensions: 2167 | 1656 | 5420 | 4305 | 2824 | 3697 | 5255 | 2204
Sub-dimensions: 7116 | 7476 | 25365

Beam labels:
- Beam 525 x 700 (multiple)
- Sec Beam 400 x 600 (multiple)
- Sec Beam 600 x 400
- Shear Wall 500
- Shear Wall 400
- Lift Core

Grid circles: X, Y, A, B, C, D, E, F, 8, 9, 9', 10, 11, 12, 13, 14, 15

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: 2nd Floor Top Reinforcement Plan	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		Sheet No: 2
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

Section Along X-X

All dimensions are in mm

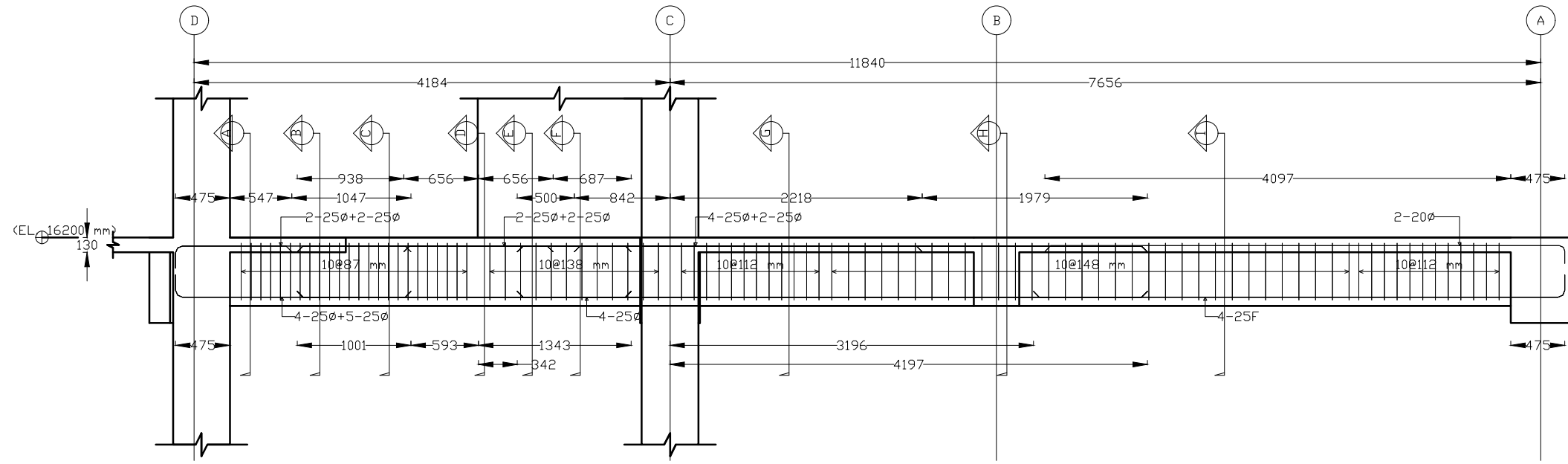| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Slab X-X Section	Group Members:	Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota (075BCE004) Abhinav Shrestha (075BCE009) Akhilesh Prasad Gupta (075BCE011) Anil Chapagain (075BCE018) Shreeya Khanal (075BCE158) Sophiya Shrestha (075BCE165)		Sheet No: 3

SLAB SECTION ALONG Y-Y

A — 1470

B — 1470 — 1235

C — 1235 — 1235

D — 1235 — 786

E — 786 — 2101

F — 2105

10 mm bars @ 300 mm c/c

735

735
486

486 — 795

795 — 486

486 — 1115

1125

1495 — 877

1054 — 1267

1293 — 912

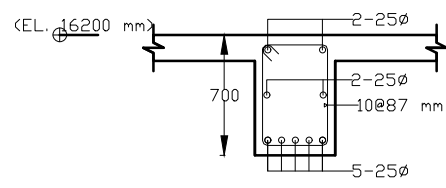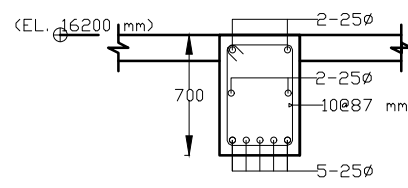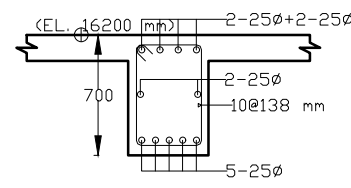862 — 1800

1852

4905

2939

4094

2357

5572

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Slab Y-Y Section	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		Sheet No: 4
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

7482

847

Shear Wall 400 mm thickness

Curved Beam 525 x 700

Y

499

10 mm bars @
300 mm c/c

10 mm bars @
300 mm c/c

3396

10 mm bars @
225 mm c/c

4416

Beam 525 x 700

1519

Y

542

6092

TOP VIEW OF RAMP SLAB

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Reinforcement Detailing Of Ramp Slab	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		
			Abhinav Shrestha	(075BCE009)		Sheet No:
			Akhilesh Prasad Gupta	(075BCE011)		5
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

130

700

525

10 mm bars @
225 mm c/c

700

10 mm bars @
300 mm c/c

525

Y-Y SECTION OF RAMP SLAB

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **"Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"**	Sheet Title: **Reinforcement Detailing Of Ramp Slab**	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota (075BCE004) Abhinav Shrestha (075BCE009) Akhilesh Prasad Gupta (075BCE011) Anil Chapagain (075BCE018) Shreeya Khanal (075BCE158) Sophiya Shrestha (075BCE165)			Sheet No: **6**

9CB1:Elevation
(Scale 1:50)

9CB1:Section A
(Scale 1:15)

9CB1:Section B
(Scale 1:15)

9CB1:Section C
(Scale 1:15)

9CB1:Section D
(Scale 1:15)

9CB1:Section E
(Scale 1:15)

| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Reinforcement Detailing of Beams	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		Sheet No: **8**
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

Section F — (EL 16200 mm), 700, 525, 4-25∅+2-22∅, 4-25∅, 10@150 mm, 5-25∅
9CB1:Section F (Scale 1:15)

Section G — (EL 16200 mm), 700, 525, 4-25∅+2-22∅, 4-25∅, 10@100 mm, 5-25∅
9CB1:Section G (Scale 1:15)

Section H — (EL 16200 mm), 700, 525, 4-25∅, 4-25∅, 10@147 mm, 5-25∅
9CB1:Section H (Scale 1:15)

Section I — (EL 16200 mm), 700, 525, 4-25∅+2-22∅, 4-25∅, 10@147 mm, 4-25∅
9CB1:Section I (Scale 1:15)

Section J — (EL 16200 mm), 700, 525, 4-25∅+2-22∅, 4-25∅, 10@100 mm, 4-25∅
9CB1:Section J (Scale 1:15)

Section K — (EL 16200 mm), 700, 525, 4-25∅, 4-25∅, 10@198 mm, 4-25∅
9CB1:Section K (Scale 1:15)

Section L — (EL 16200 mm), 700, 525, 4-25∅+2-22∅, 4-25∅, 10@198 mm, 4-25∅
9CB1:Section L (Scale 1:15)

Section M — (EL 16200 mm), 700, 525, 4-25∅+2-22∅, 4-25∅, 10@150 mm, 4-25∅
9CB1:Section M (Scale 1:15)

Section N — (EL 16200 mm), 700, 525, 4-25∅+2-22∅, 4-25∅, 10@150 mm, 4-25∅
9CB1:Section N (Scale 1:15)

Section O — 700, 525, 2-25∅+2-14∅, 4-25∅, 10@150 mm, 2-25∅
9CB1:Section O (Scale 1:15)

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Reinforcement Detailing of Beams	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		Sheet No: 9
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

9CB10:Elevation
(Scale 1:25)

9CB10:Section B
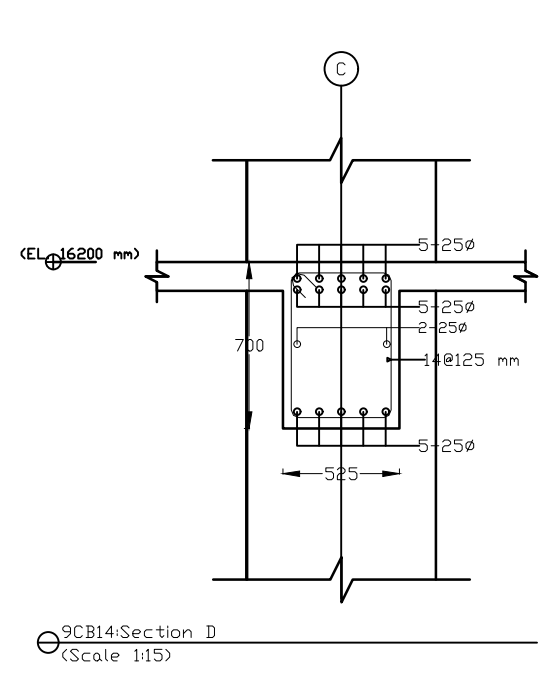(Scale 1:15)

9CB10:Section C
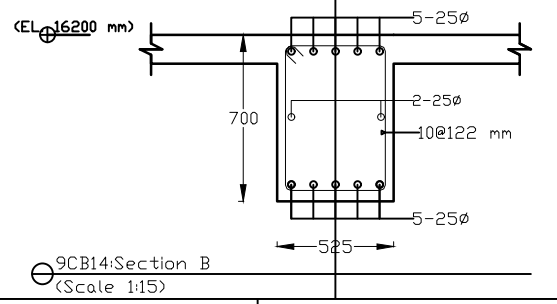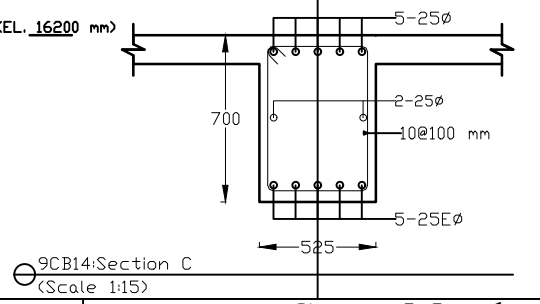(Scale 1:15)

9CB10:Section D
(Scale 1:15)
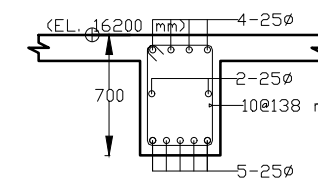
9CB10:Section E
(Scale 1:15)
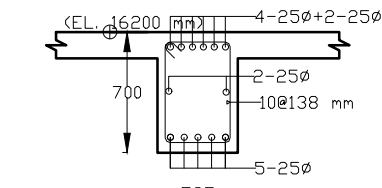
9CB10:Section F
(Scale 1:15)
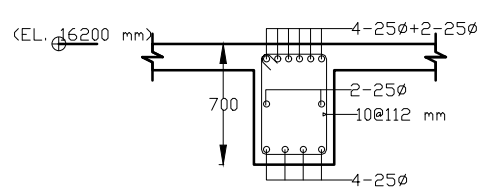
9CB10:Section G
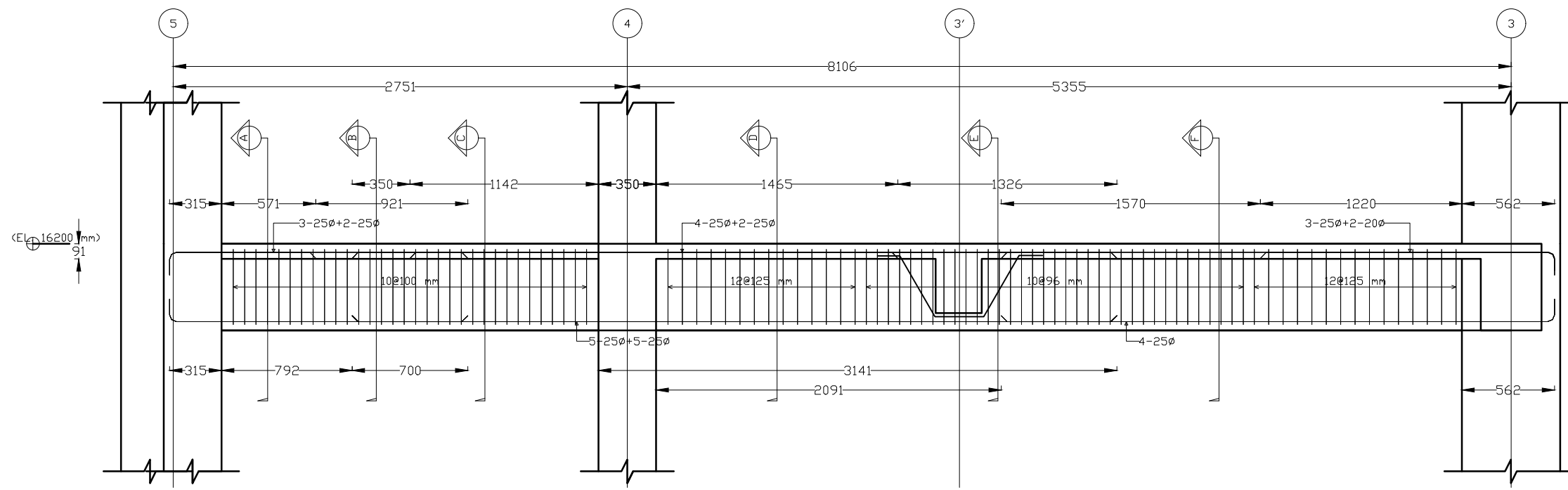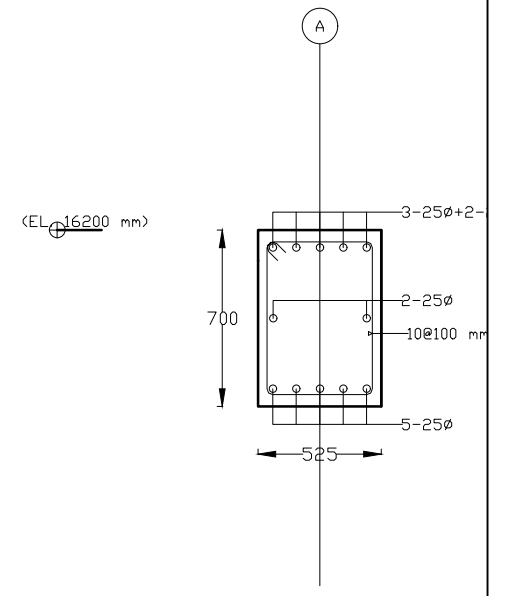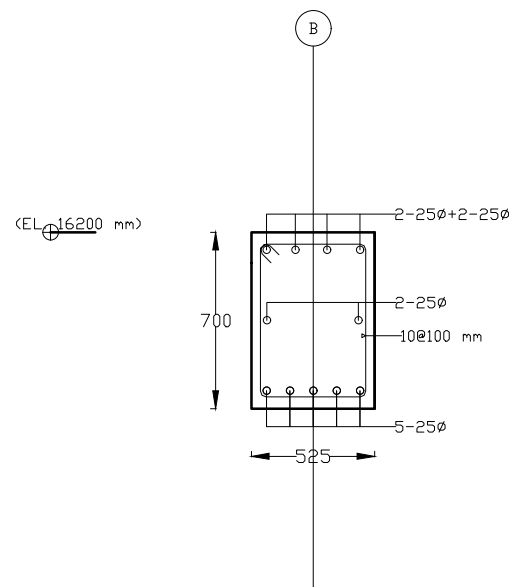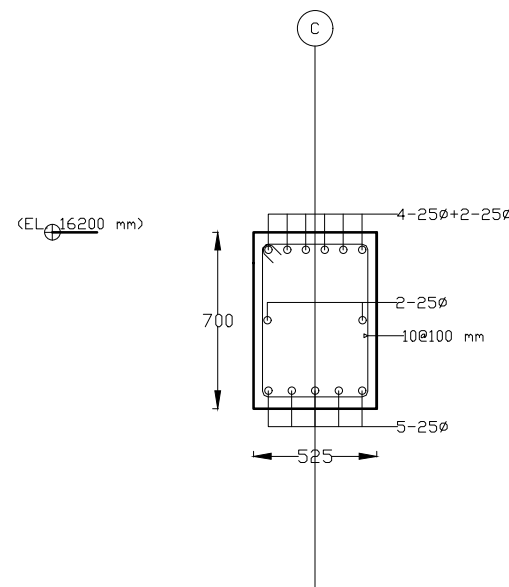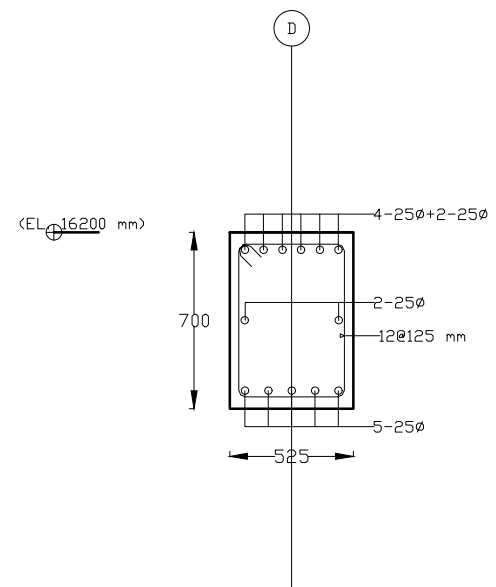(Scale 1:15)
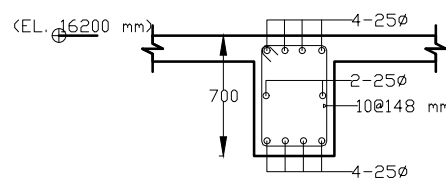
9CB10:Section H
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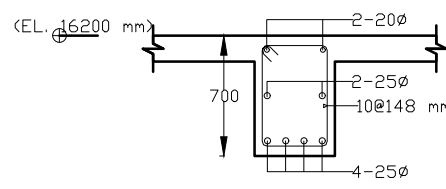
9CB10:Section I
(Scale 1:15)
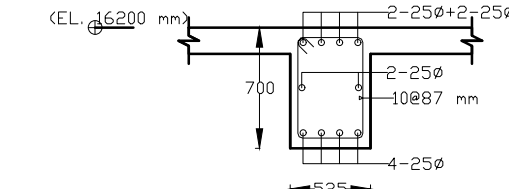
9CB10:Section A
(Scale 1:15)

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Reinforcement Detailing of Beams	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		Sheet No:
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		10
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		
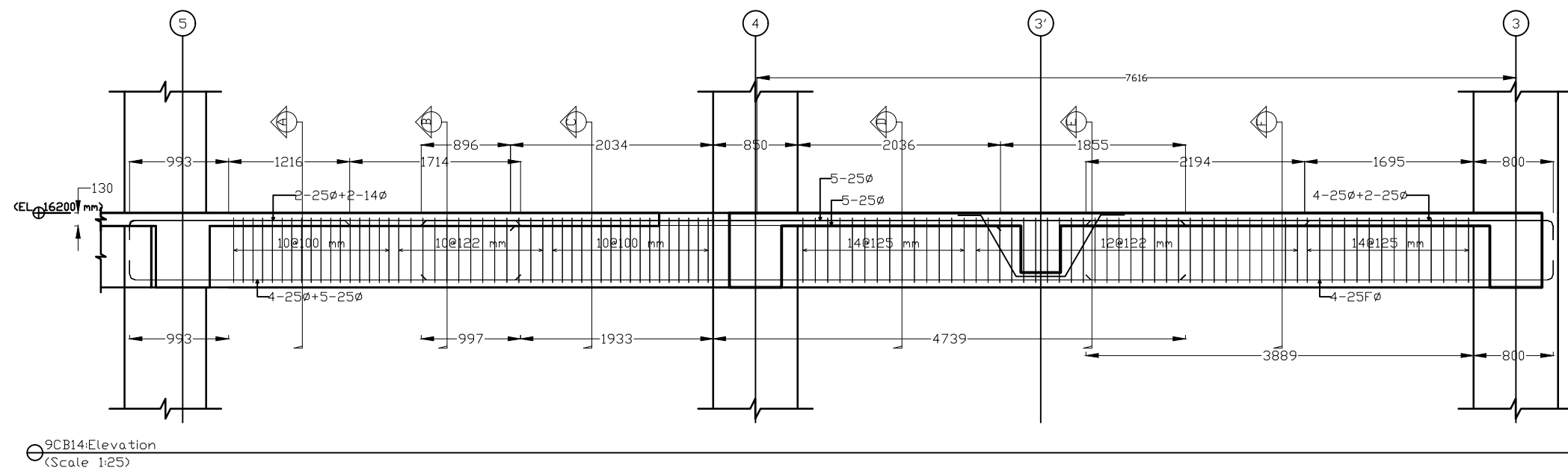
9CB14:Elevation
(Scale 1:25)
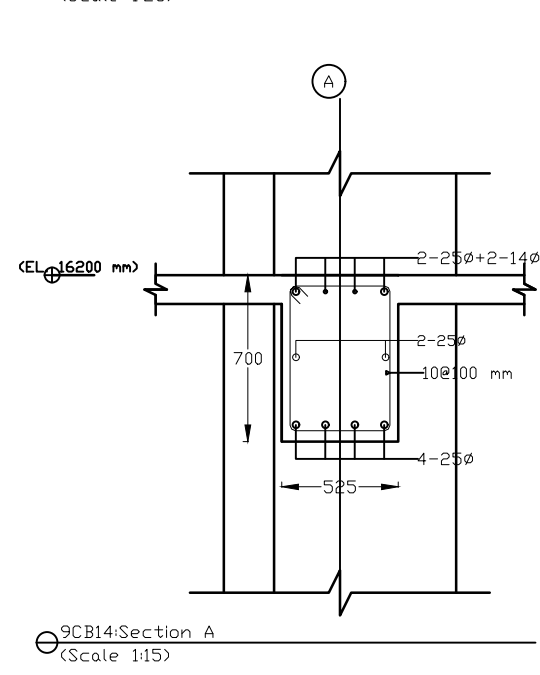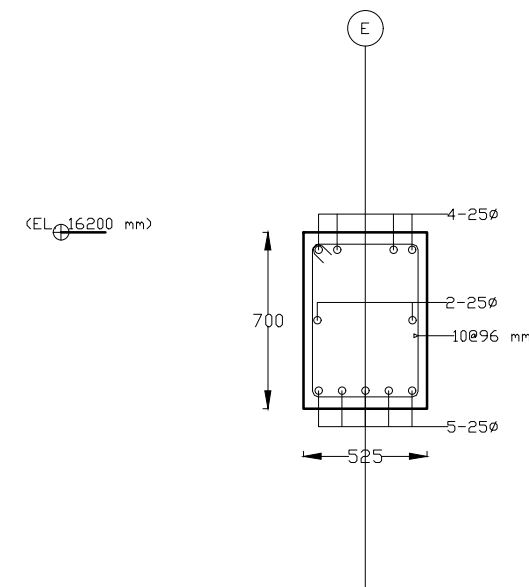
9CB14:Section A
(Scale 1:15)
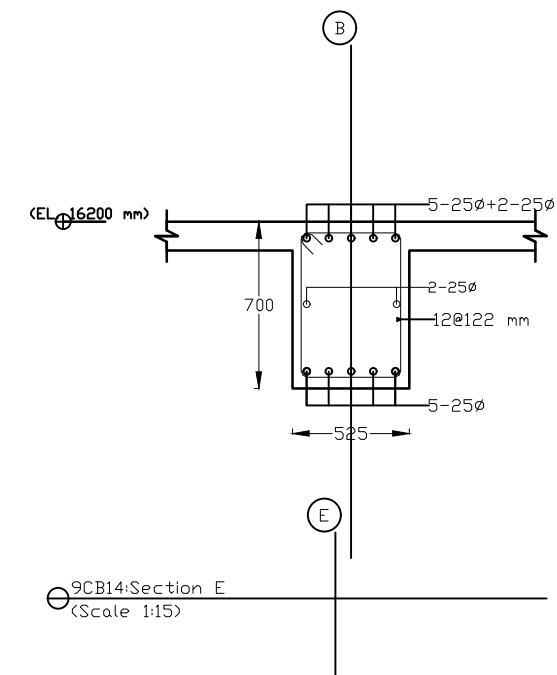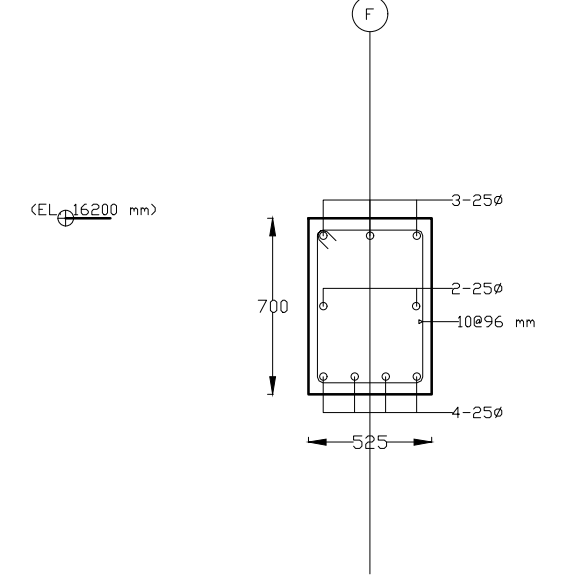
9CB14:Section E
(Scale 1:15)
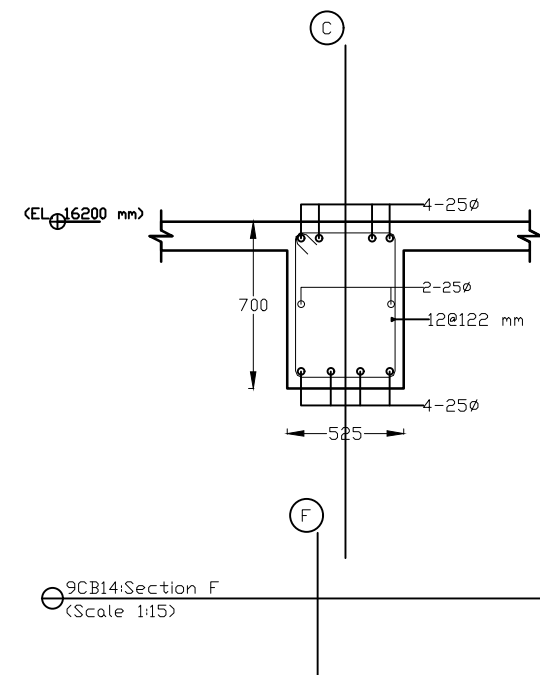
9CB14:Section F
(Scale 1:15)

9CB14:Section D
(Scale 1:15)

9CB14:Section B
(Scale 1:15)

9CB14:Section C
(Scale 1:15)

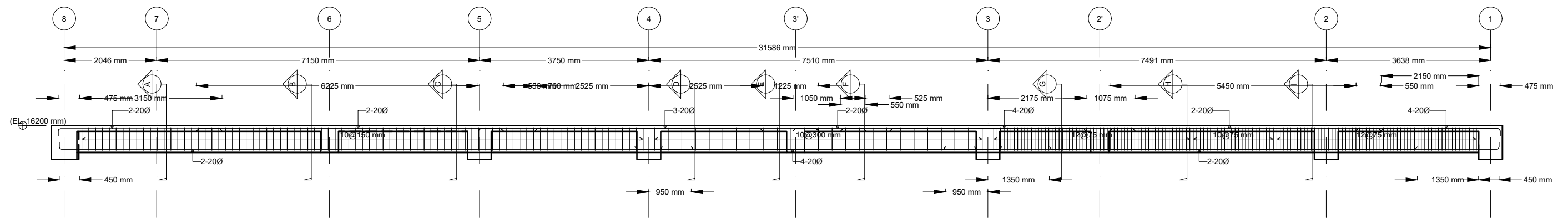| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Reinforcement Detailing of Beams	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		Sheet No: 11
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

9CB5:Elevation (Scale 1:25)

9CB5:Section A (Scale 1:15)

9CB5:Section B (Scale 1:15)
9CB5:Section C (Scale 1:15)
9CB5:Section D (Scale 1:15)
9CB5:Section E (Scale 1:15)
9CB5:Section F (Scale 1:15)

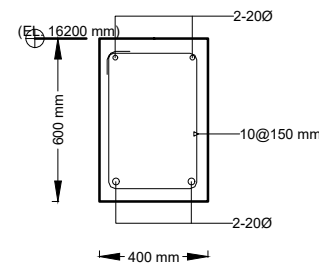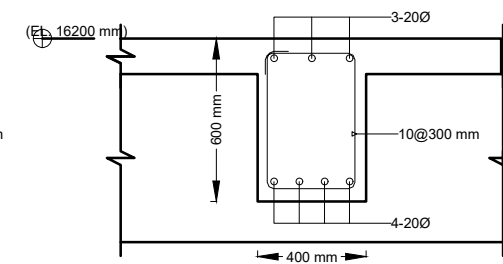| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Reinforcement Detailing of Beams	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		Sheet No: 12
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

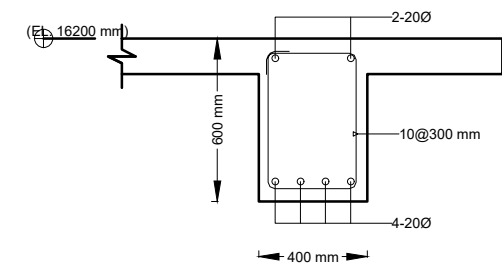9CB20:Elevation
(Scale 1:50)

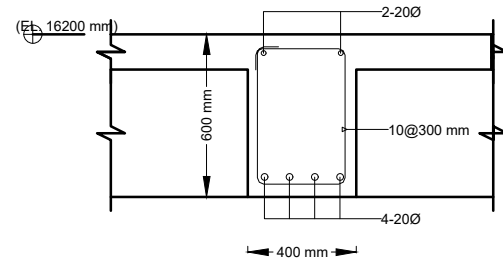9CB20:Section A
(Scale 1:15)
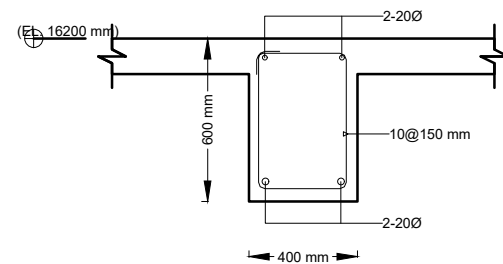
9CB20:Section B
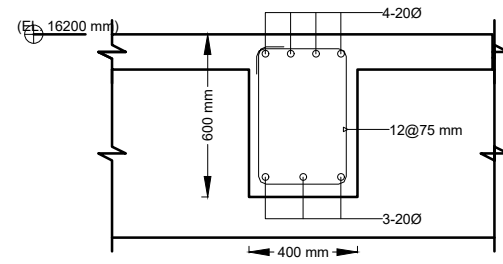(Scale 1:15)

9CB20:Section C
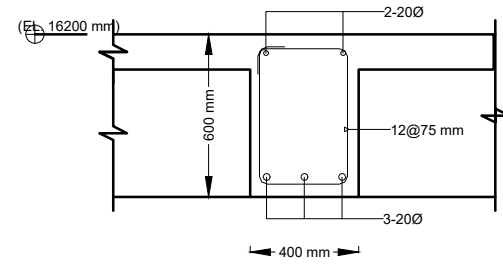(Scale 1:15)

9CB20:Section D
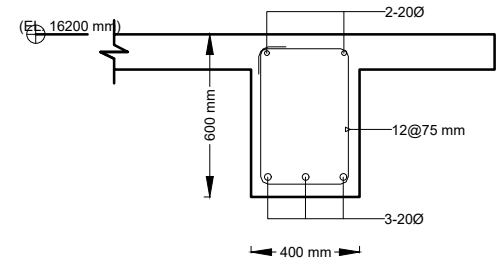(Scale 1:15)

9CB20:Section E
(Scale 1:15)

9CB20:Section F
(Scale 1:15)

9CB20:Section G
(Scale 1:15)

9CB20:Section H
(Scale 1:15)

9CB20:Section I
(Scale 1:15)

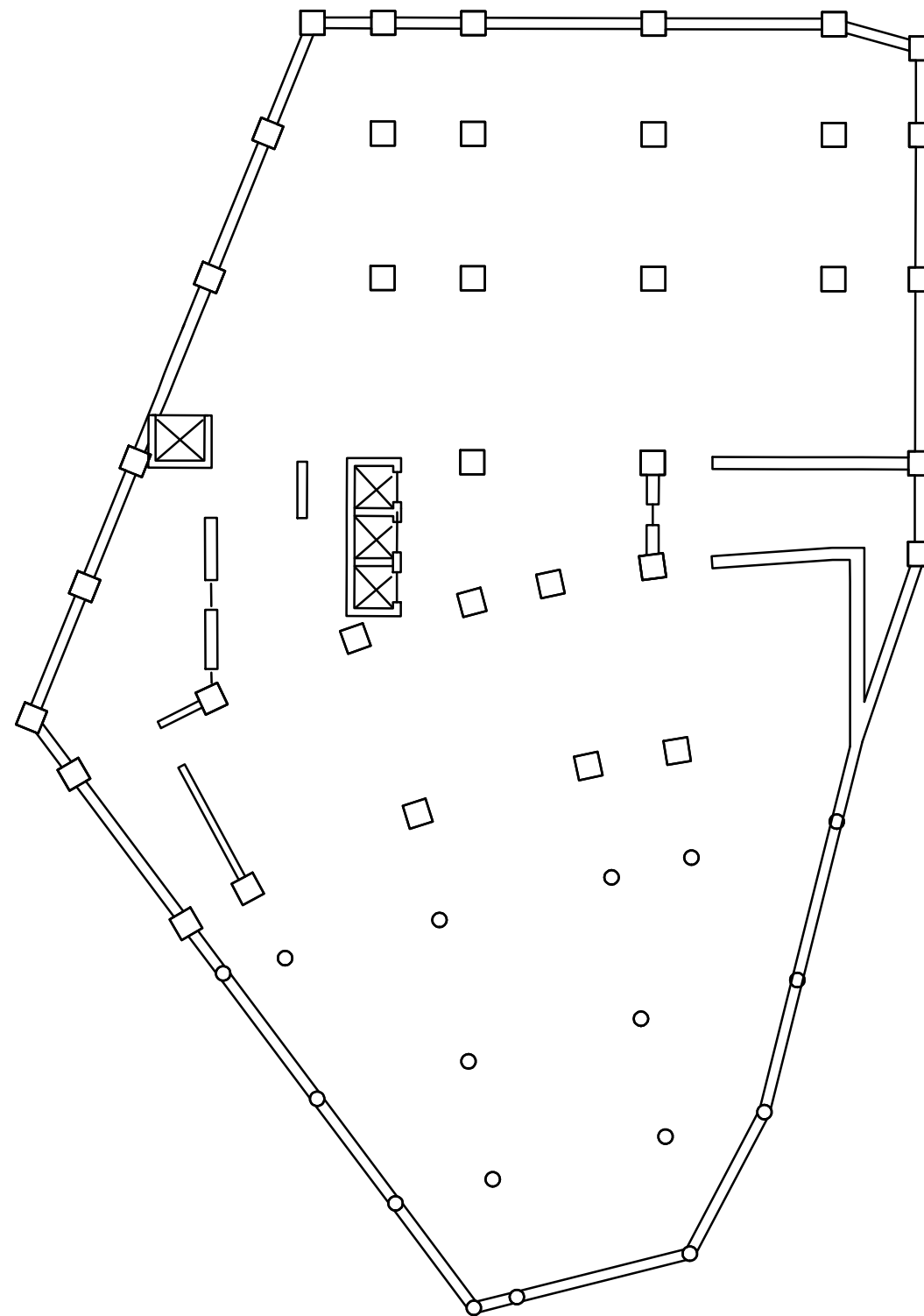| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Reinforcement Detailing of Secondary Beams	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		Sheet No: 13
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **"Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"**	Sheet Title: **Concrete Column Layout Plan**	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota (075BCE004) Abhinav Shrestha (075BCE009) Akhilesh Prasad Gupta (075BCE011) Anil Chapagain (075BCE018) Shreeya Khanal (075BCE158) Sophiya Shrestha (075BCE165)			Sheet No: 14

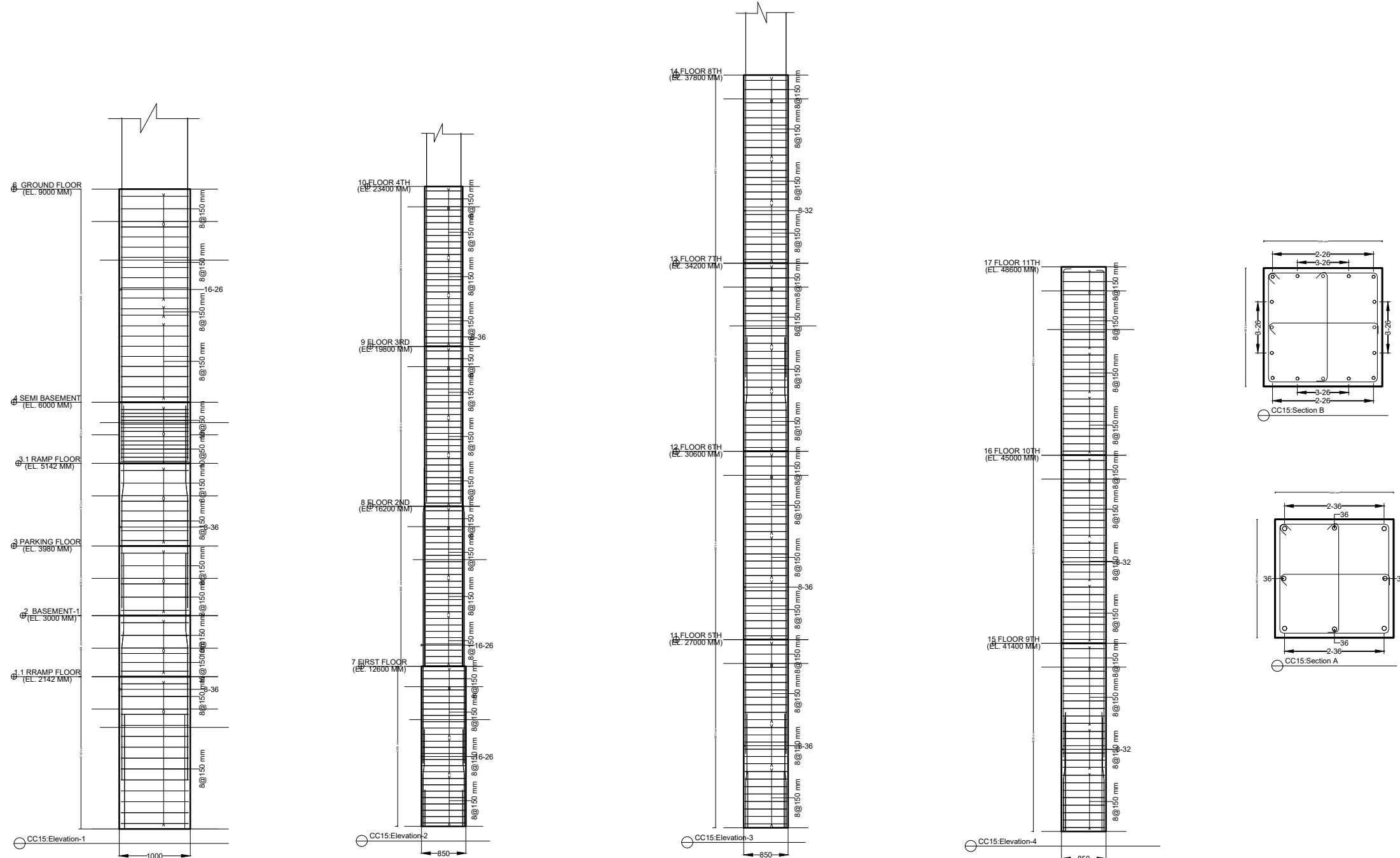| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering | Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building" | Sheet Title: Reinforcement Detailing Of Column | Group Members: Aashish Sapkota (075BCE004) Abhinav Shrestha (075BCE009) Akhilesh Prasad Gupta (075BCE011) Anil Chapagain (075BCE018) Shreeya Khanal (075BCE158) Sophiya Shrestha (075BCE165) | Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha | Scale: Fit to scale Sheet No: 15 |

CC15:Section C

CC15:Section D

CC15:Section E

CC15:Section F

CC15:Section G

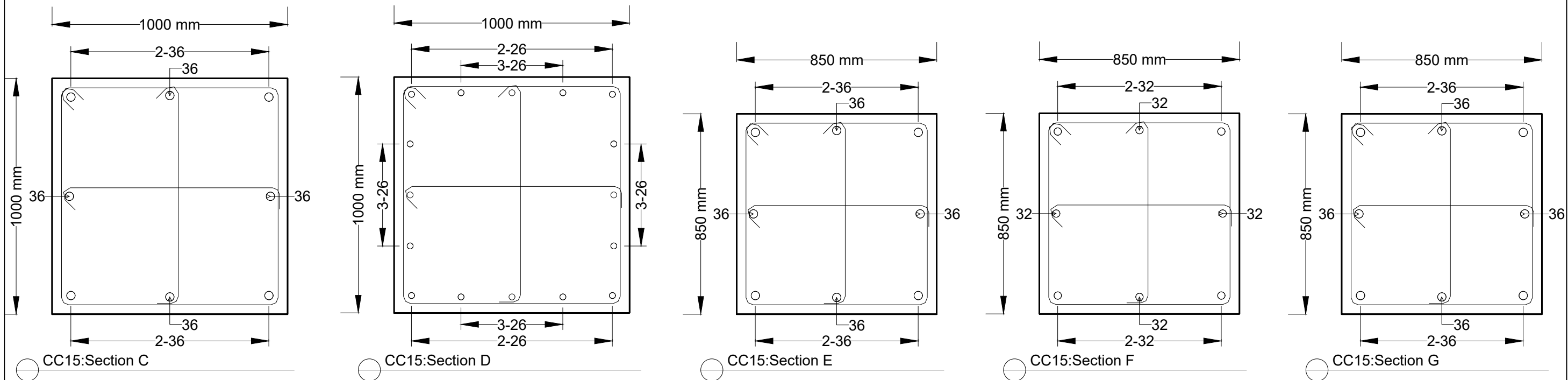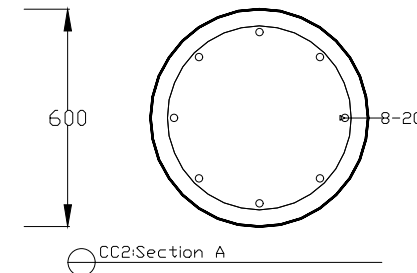| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Reinforcement Detailing Of Column	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota (075BCE004) Abhinav Shrestha (075BCE009) Akhilesh Prasad Gupta (075BCE011) Anil Chapagain (075BCE018) Shreeya Khanal (075BCE158) Sophiya Shrestha (075BCE165)			Sheet No: 16

CC3:Elevation

5 PARKING INLET FLOOR
(EL. 7800 MM)

8@75 mm

1800

8@75 mm

8-28

4 SEMI BASEMENT
(EL. 6000 MM)

858

8@100 mm

3.1 RAMP FLOOR
(EL. 5142 MM)

8@100 mm

1162

8@100 mm

3 PARKING FLOOR
(EL. 3980 MM)

8@100 mm

8-25

3980

8@125 mm

8@100 mm

600

600

CC3:Section C

8-28

600

CC3:Section B

8-20

600

CC3:Section A

8-25

5 PARKING INLET FLOOR
(EL. 7800 MM)

8@100 mm

1800

8@100 mm

8-20

4 SEMI BASEMENT
(EL. 6000 MM)

858

8@100 mm

3.1 RAMP FLOOR
(EL. 5142 MM)

8@100 mm

1162

8@100 mm

3 PARKING FLOOR
(EL. 3980 MM)

8@100 mm

8-20

3980

8@125 mm

8@100 mm

600

CC2:Elevation

600

CC2:Section A

8-20

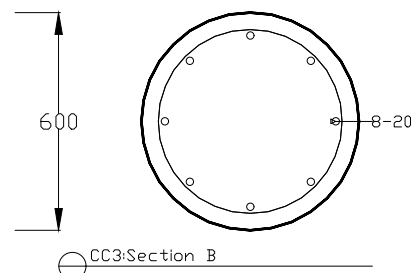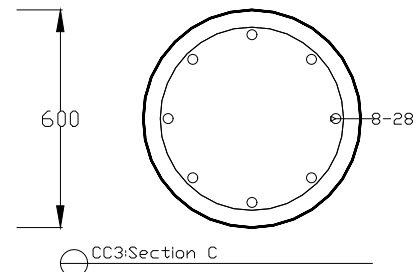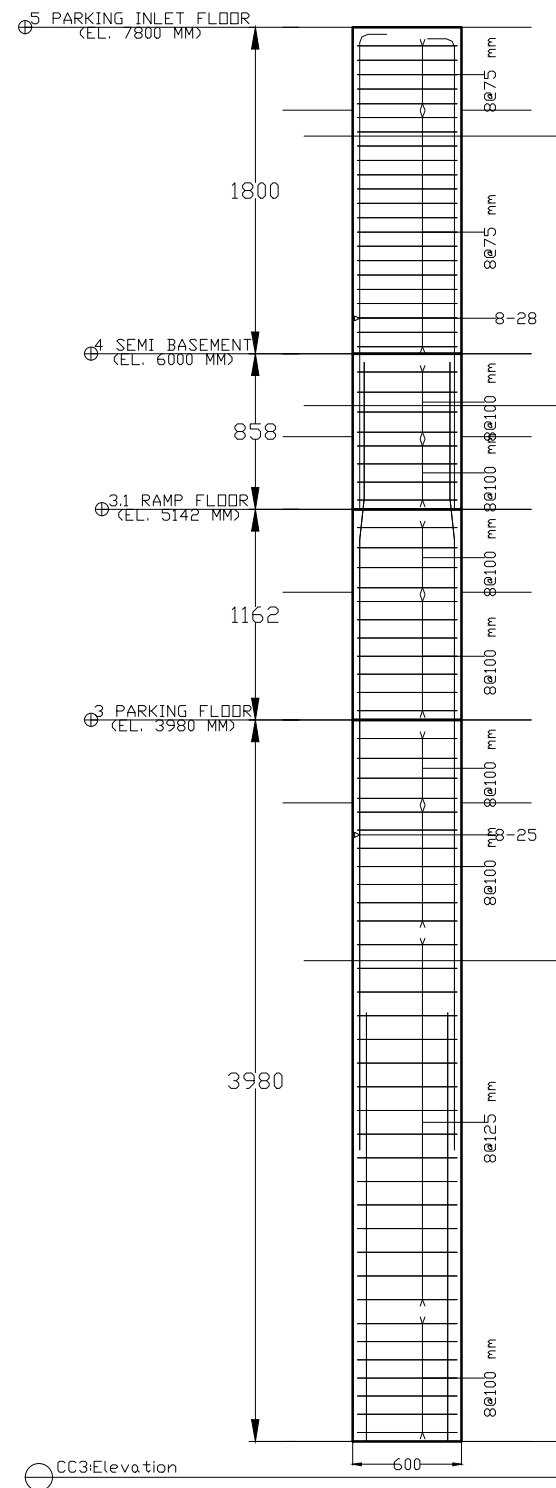| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Reinforcement Detailing Of Circular Column	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		Sheet No: 17
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		
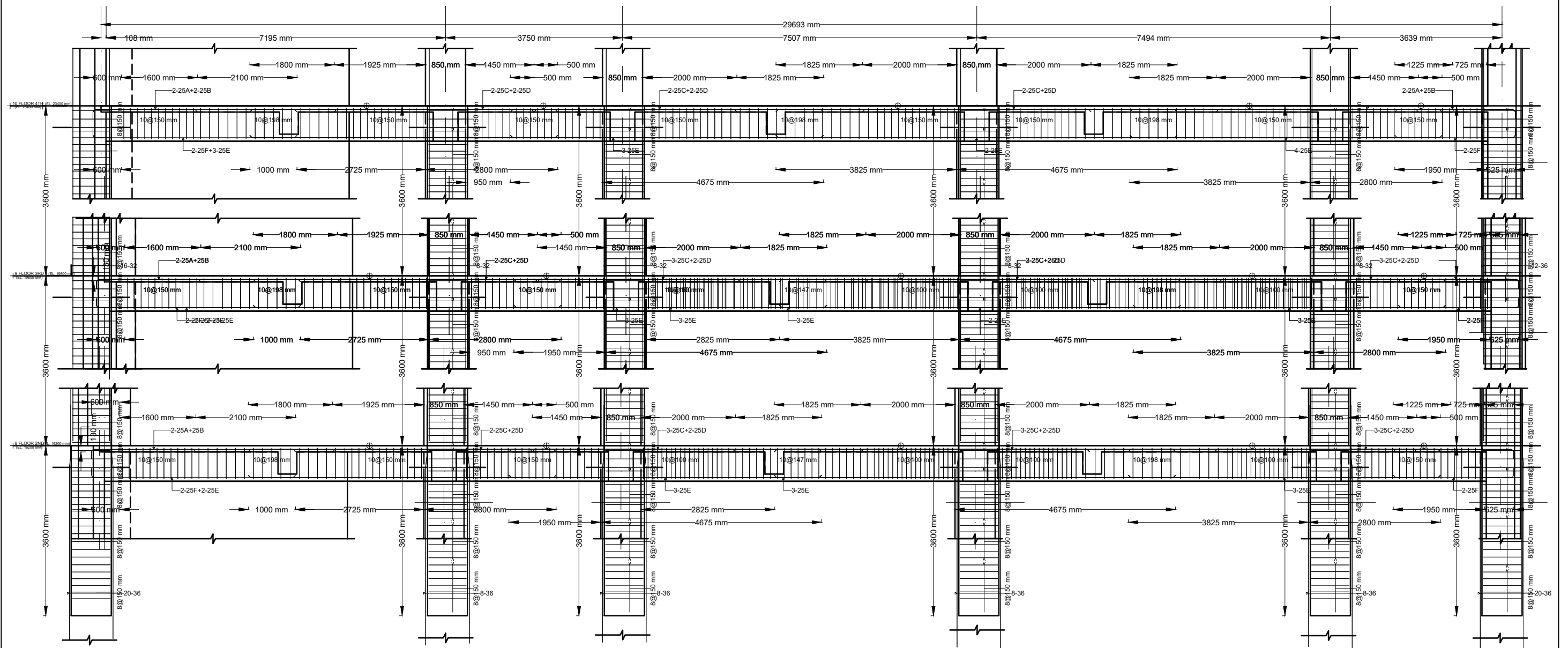
Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Beam-Column Detailing in Grid-3 along Elevation	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		Sheet No: 18
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		

Plan of Mat Foundation

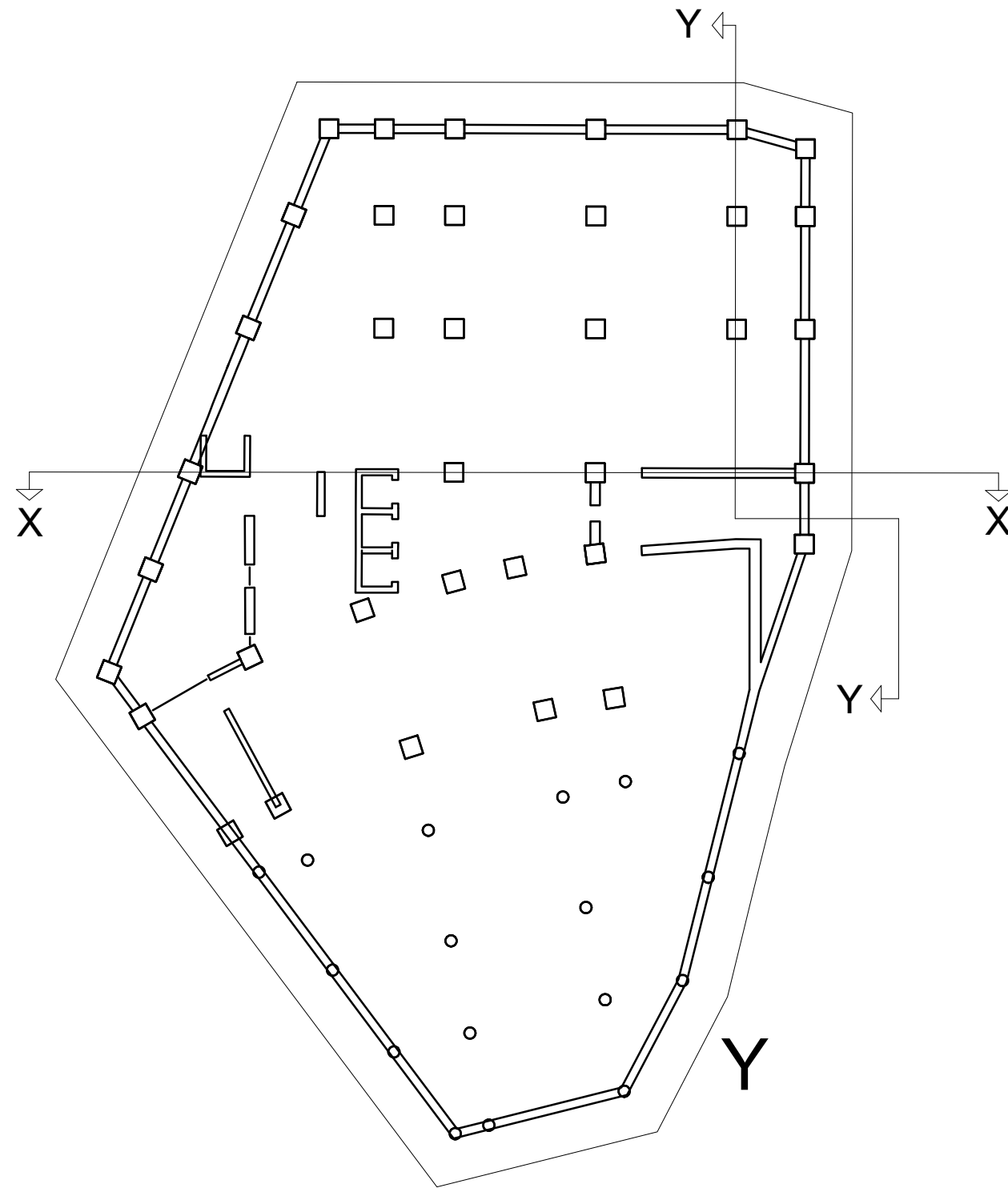| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **"Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"**	Sheet Title: **Mat Foundation Plan**	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota (075BCE004) Abhinav Shrestha (075BCE009) Akhilesh Prasad Gupta (075BCE011) Anil Chapagain (075BCE018) Shreeya Khanal (075BCE158) Sophiya Shrestha (075BCE165)			Sheet No: 19

Longitudinal Reinforcement 16-26 Ø

20 mm @ 150 mm c/c

10 mm @ 75 mm c/c

20 mm @ 150 mm c/c

20 mm @ 200 mm c/c

20 mm @ 150 mm c/c

20 mm @ 175 mm c/c

2220

20 mm @ 150 mm c/c

Chair bars 16 mm @ 480 mm c/c

Ld=1134

X-X Section

20 mm @ 150 mm c/c

Chair bars 16 mm @ 480 mm c/c

20 mm @ 150 mm c/c

20 mm @ 150 mm c/c

Longitudinal Reinforcement 16-26 Ø

10 mm @ 75 mm c/c

20 mm @ 175 mm c/c

20 mm @ 150 mm c/c

20 mm @ 150 mm c/c

Y-Y Section

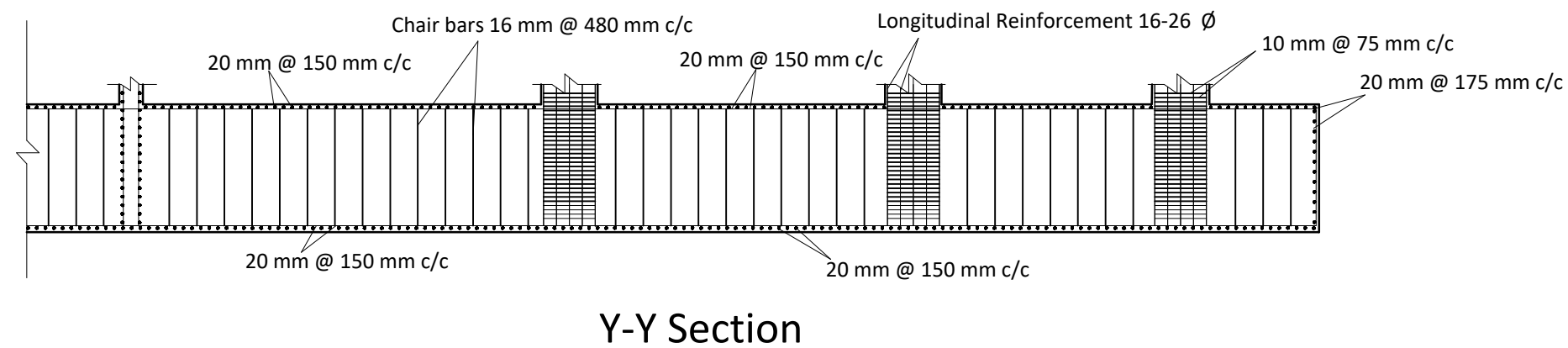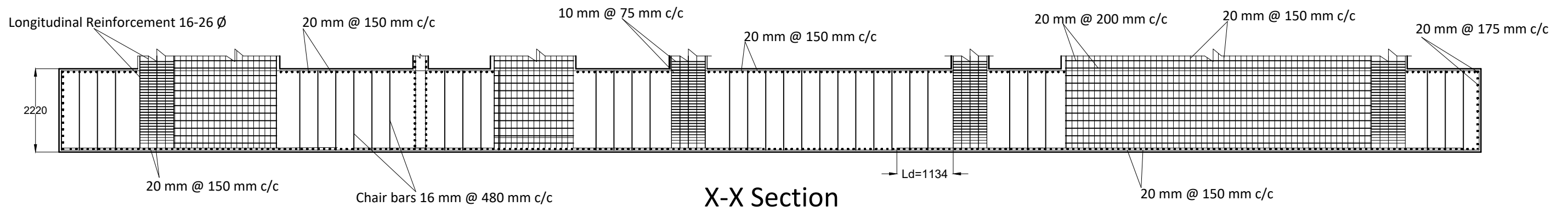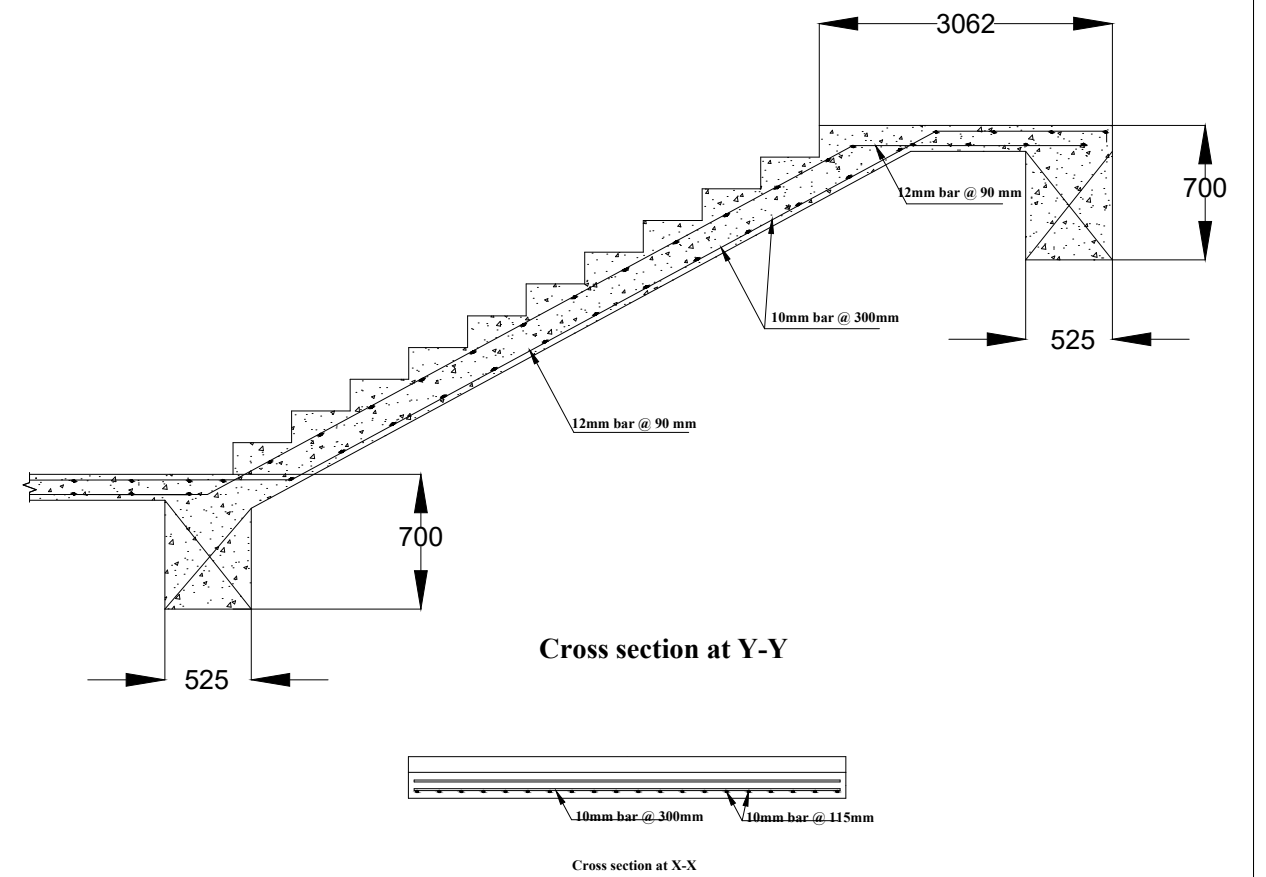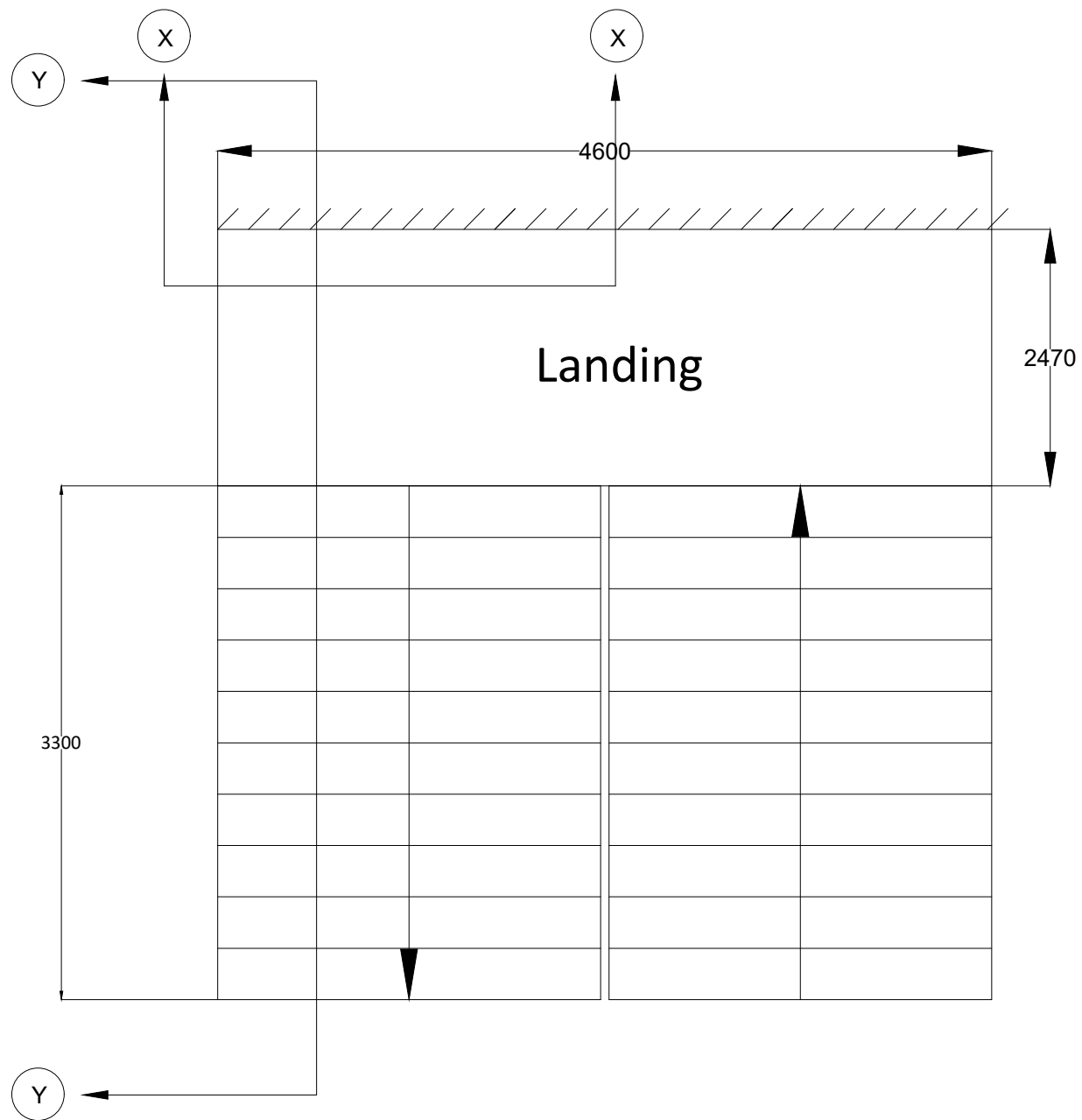| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **"Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"**	Sheet Title: **Reinforcement Detailing Of Mat Foundation**	Group Members: Aashish Sapkota (075BCE004) Abhinav Shrestha (075BCE009) Akhilesh Prasad Gupta (075BCE011) Anil Chapagain (075BCE018) Shreeya Khanal (075BCE158) Sophiya Shrestha (075BCE165)	Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
					Sheet No: **20**

Landing

4600

2470

3300

3062

700

12mm bar @ 90 mm

10mm bar @ 300mm

12mm bar @ 90 mm

525

700

525

Cross section at Y-Y

10mm bar @ 300mm   10mm bar @ 115mm

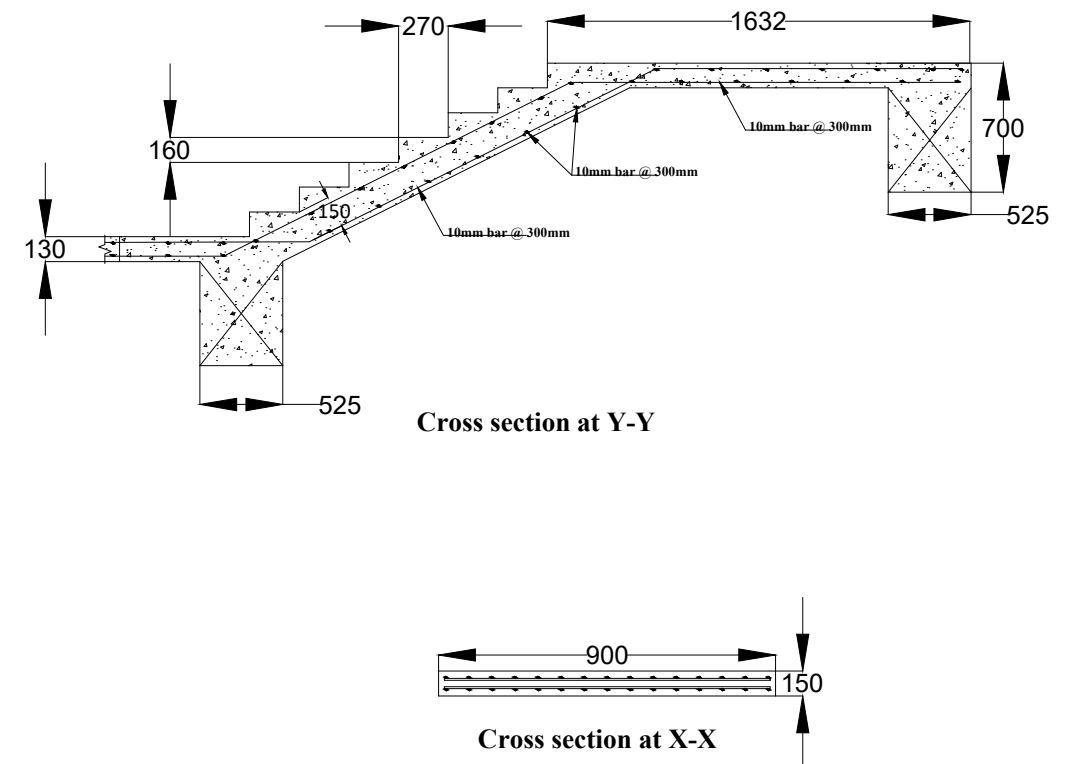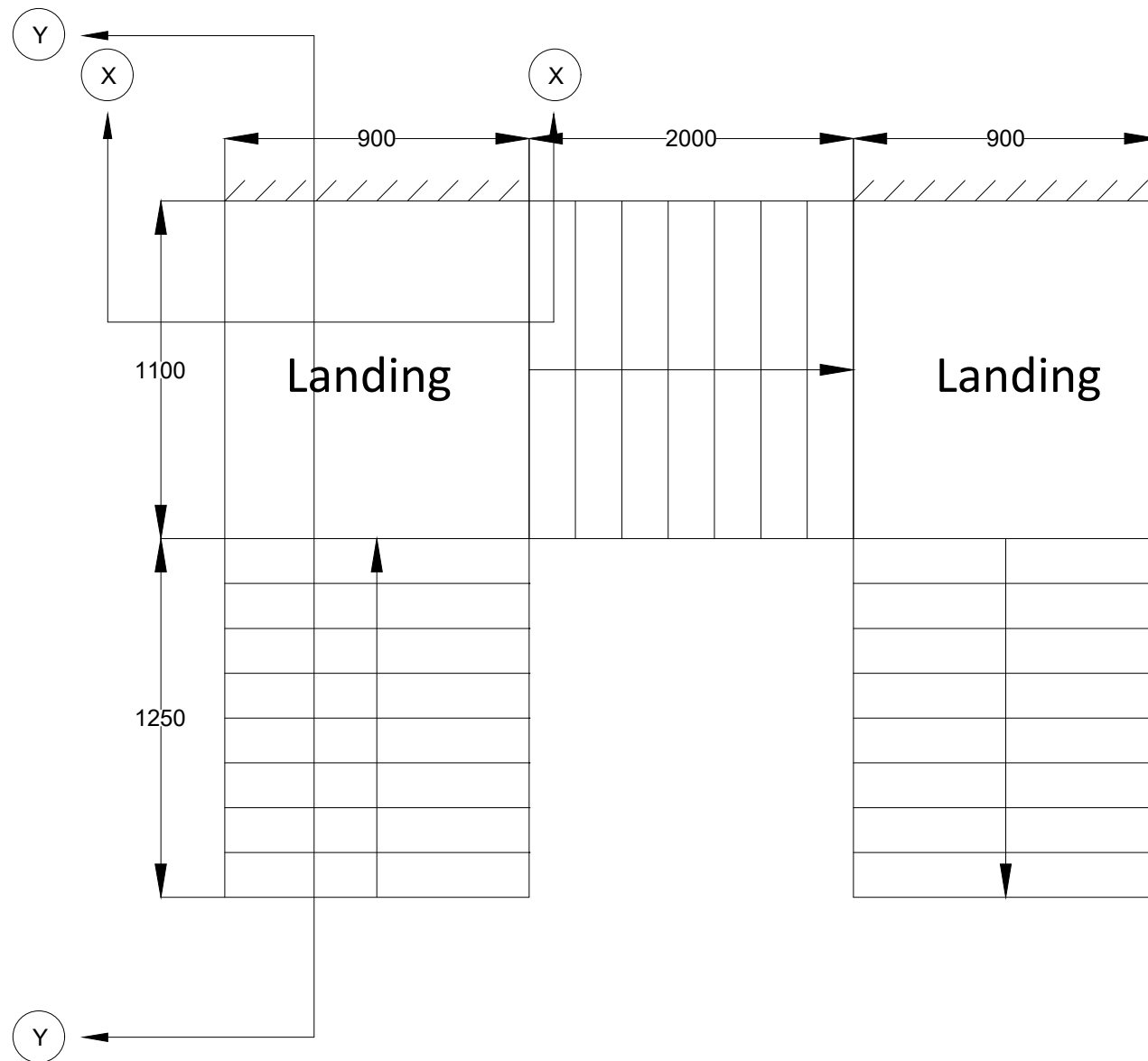**Cross section at X-X**

Dog-legged Staircase

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **"Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"**	Sheet Title: **Reinforcement Detailing of Dog-legged Staircase**	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota (075BCE004) Abhinav Shrestha (075BCE009) Akhilesh Prasad Gupta (075BCE011) Anil Chapagain (075BCE018) Shreeya Khanal (075BCE158) Sophiya Shrestha (075BCE165)			Sheet No: **21**

Plan View

900	2000	900	

1100

Landing

Landing

1250

Cross section at Y-Y

270   1632

160

150

130

10mm bar @ 300mm

10mm bar @ 300mm

10mm bar @ 300mm

700

525

525

Cross section at X-X

900

150

Open Newel Staircase

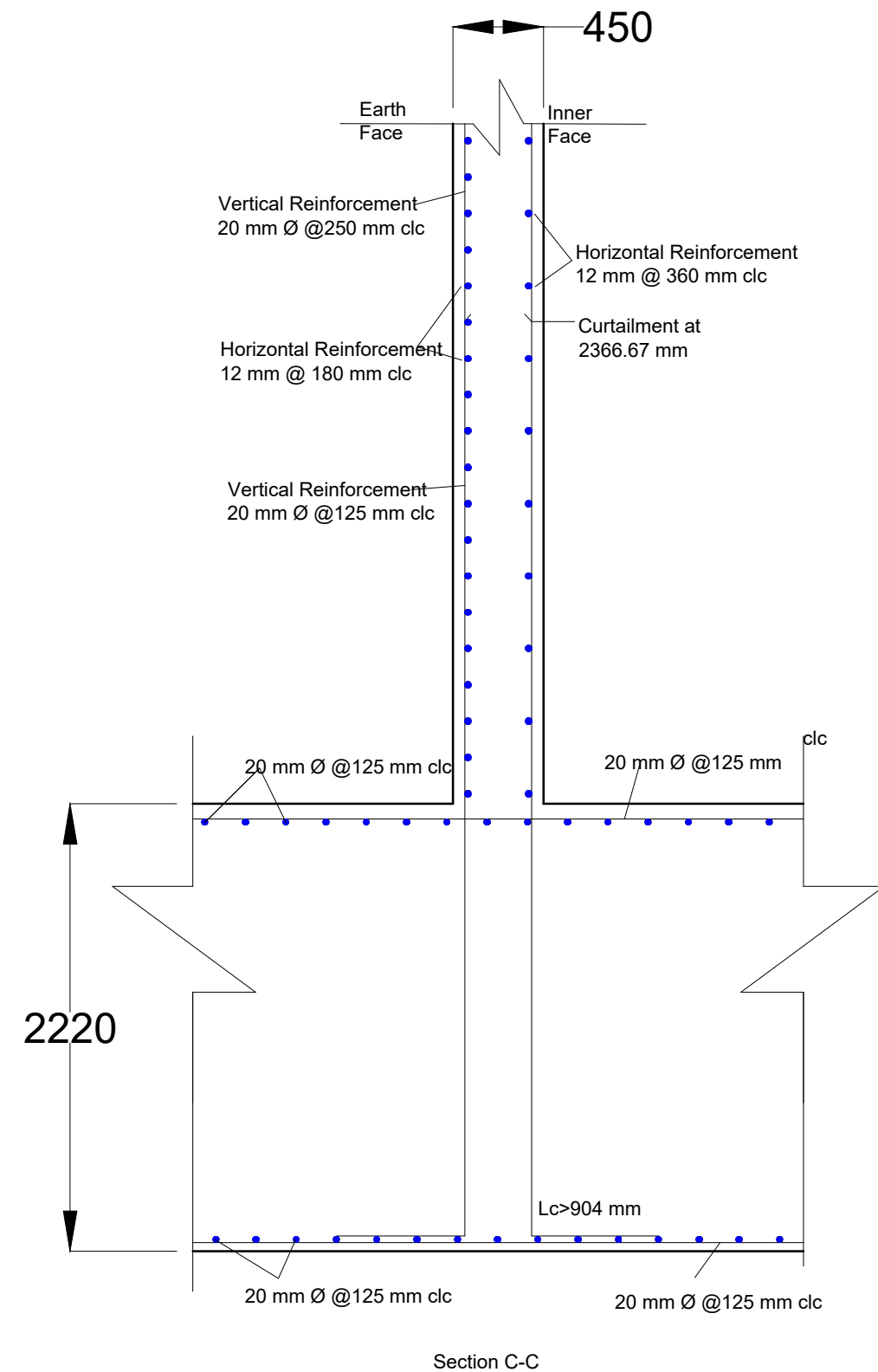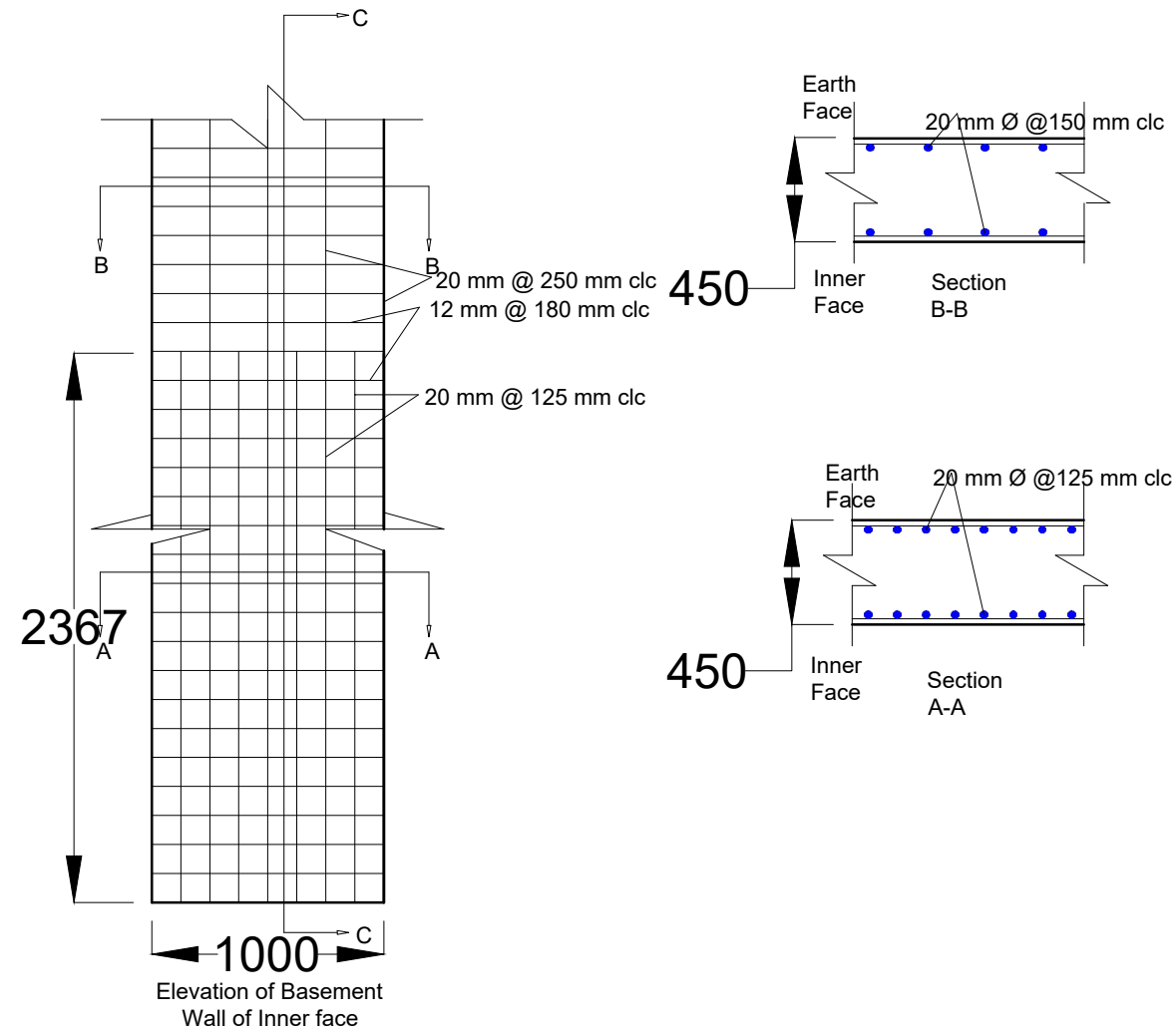| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Reinforcement Detailing in Open-Newel Staircase	Group Members:	Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota (075BCE004) Abhinav Shrestha (075BCE009) Akhilesh Prasad Gupta (075BCE011) Anil Chapagain (075BCE018) Shreeya Khanal (075BCE158) Sophiya Shrestha (075BCE165)		Sheet No: **22**

Elevation of Basement
Wall of Inner face

2367

1000

Section B-B

Earth Face
Inner Face
450
20 mm Ø @150 mm clc

20 mm @ 250 mm clc
12 mm @ 180 mm clc
20 mm @ 125 mm clc

Section A-A

Earth Face
Inner Face
450
20 mm Ø @125 mm clc

Section C-C

450

Earth Face
Inner Face

Vertical Reinforcement
20 mm Ø @250 mm clc

Horizontal Reinforcement
12 mm @ 180 mm clc

Vertical Reinforcement
20 mm Ø @125 mm clc

Horizontal Reinforcement
12 mm @ 360 mm clc

Curtailment at
2366.67 mm

20 mm Ø @125 mm clc

20 mm Ø @125 mm clc

2220

Lc>904 mm

20 mm Ø @125 mm clc

20 mm Ø @125 mm clc

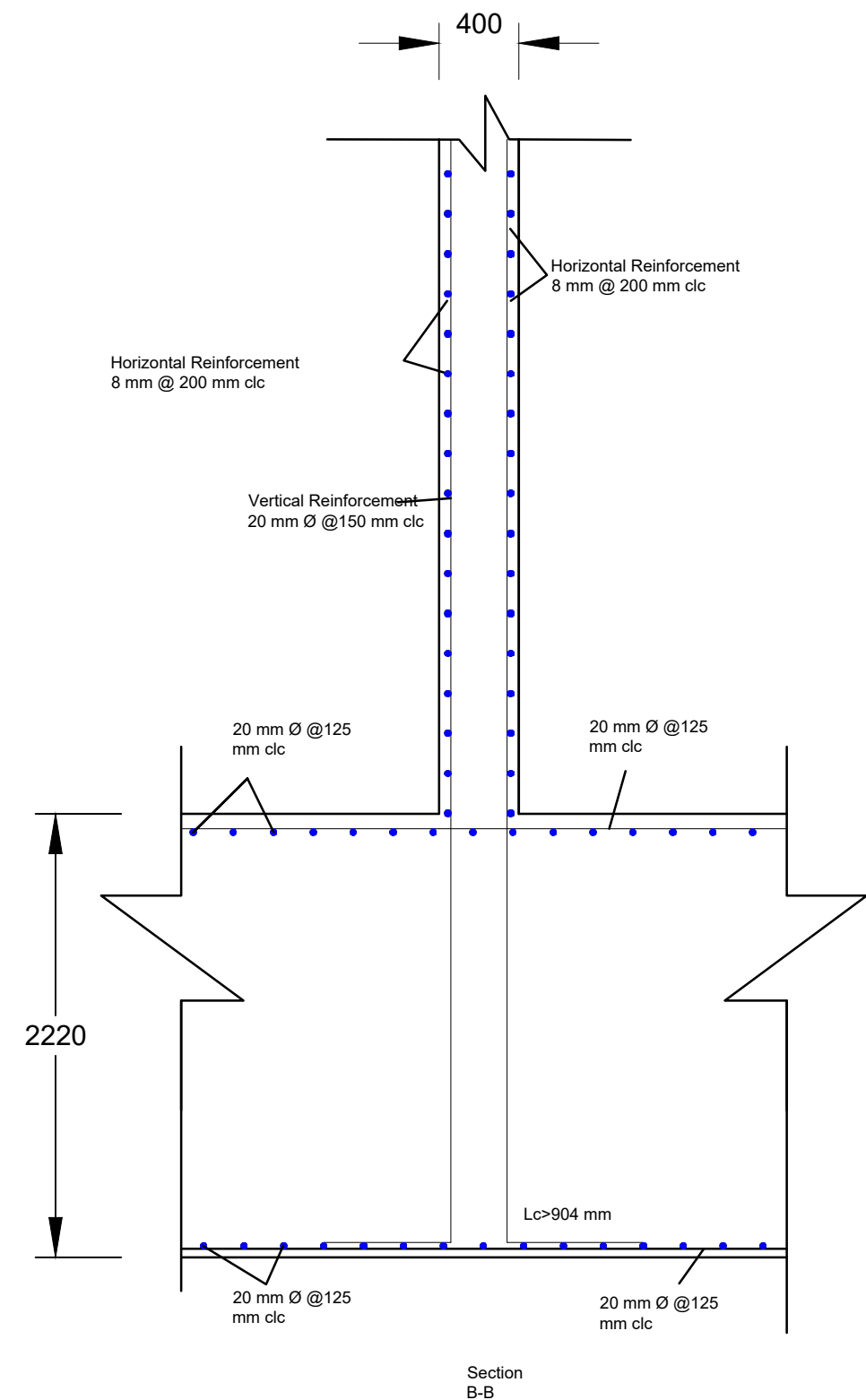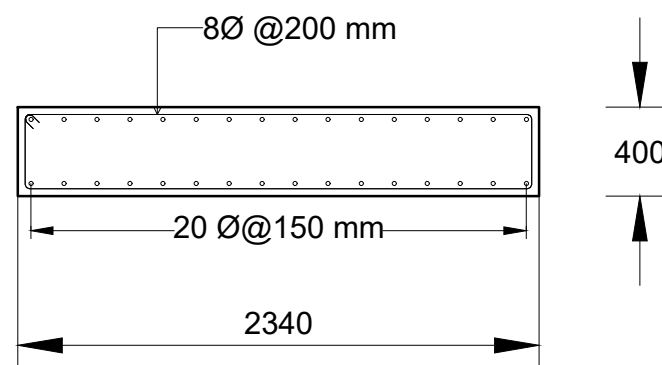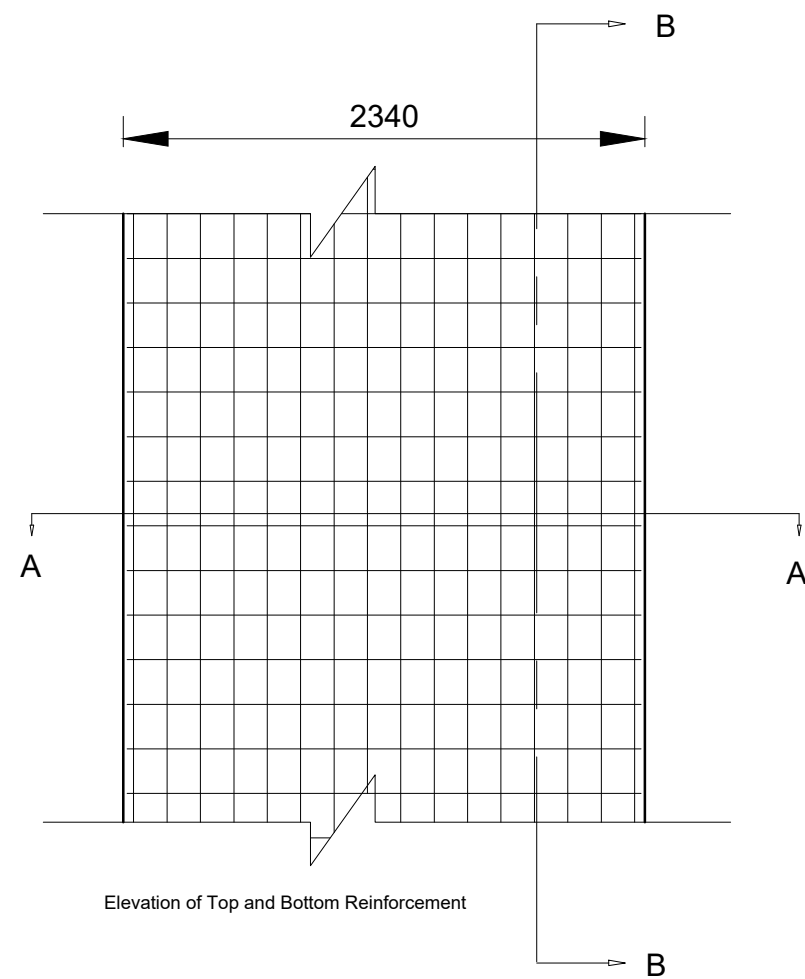| Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Reinforcement Detailing of Basement Wall	Group Members:	Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota (075BCE004) Abhinav Shrestha (075BCE009) Akhilesh Prasad Gupta (075BCE011) Anil Chapagain (075BCE018) Shreeya Khanal (075BCE158) Sophiya Shrestha (075BCE165)		Sheet No: 23

2340

B

A ————————————— A

B

Elevation of Top and Bottom Reinforcement

8Ø @200 mm

20 Ø@150 mm

400

2340

Section: A - A

400

Horizontal Reinforcement
8 mm @ 200 mm clc

Horizontal Reinforcement
8 mm @ 200 mm clc

Vertical Reinforcement
20 mm Ø @150 mm clc

20 mm Ø @125
mm clc

20 mm Ø @125
mm clc

2220

Lc>904 mm

20 mm Ø @125
mm clc

20 mm Ø @125
mm clc

Section
B-B

Tribhuvan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: "Earthquake Resistant Design of Irregular Multi-storied RCC Commercial Building"	Sheet Title: Reinforcement Detailing Of Shear Wall	Group Members:		Project Supervisor: Assoc. Prof. Dr. Kshitij C. Shrestha	Scale: Fit to scale
			Aashish Sapkota	(075BCE004)		
			Abhinav Shrestha	(075BCE009)		
			Akhilesh Prasad Gupta	(075BCE011)		Sheet No: 24
			Anil Chapagain	(075BCE018)		
			Shreeya Khanal	(075BCE158)		
			Sophiya Shrestha	(075BCE165)		