Affiliated to Pokhara University

POKHARA ENGINEERING COLLEGE

Phirke, Pokhara-8

A

Project Report

On

Comparative Study of Behaviour of Buildings using NBC 105:2020 and IS 1893:2016 based on Horizontal Irregularities

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<u>Submitted to:</u>

Research Management Cell

Pokhara Engineering College

2025

ACKNOWLEDGEMENT

We would like to express our sincere gratitude to all those who have supported and guided us throughout the course of this research project titled "Comparative Study of Behaviour of Buildings Using NBC 105:2020 and IS 1893:2016 Based on Horizontal Irregularities."

First and foremost, we are deeply thankful to our supervisor, **Er. Krishna Ghimire**, for his continuous support, encouragement, and valuable guidance throughout the project with much more attention and care.

We would also like to extend our appreciation to Er. Bimal Bhandari, Head of Department, civil and Er. Yam Bahadur Thapa for their insightful suggestions and feedback, which greatly helped in refining the scope and direction of our project.

Finally, we are grateful to our friends, faculty members, and everyone who are directly or indirectly involved for their constant motivation and support during the entire research process.

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PREFACE

This report, titled **"Comparative Study of Behaviour of Buildings Using NBC 105:2020 and IS 1893:2016 Based on Horizontal Irregularities,"** has been prepared as a part of our academic coursework and as a contribution to the understanding of seismic code applications in structural engineering.

The primary motivation for undertaking this research stems from the observation that, despite codal discouragement, horizontally irregular buildings such as L-shaped, U-shaped, and E-shaped structures are frequently constructed in practice due to functional, architectural, or site constraints. Such irregularities significantly influence a building's seismic performance, often making them more vulnerable during earthquakes.

This study aims to analyse and compare how two major seismic codes NBC 105:2020 (Nepal) and IS 1893:2016 (India) address these irregularities in terms of design provisions, structural behaviour, and performance outcomes. Through this comparative approach, we hope to contribute valuable insights into the role of code-based design in improving the safety and resilience of irregular buildings.

We extend our sincere thanks to our supervisor Er. Krishna Ghimire sir, faculty members, and everyone who guided and supported us during this study. We have tried our best to make the report error free. We apologize for any mistakes and misprints. Any comments and critics are highly appreciated.

ABSTRACT

Seismic performance of structures is highly influenced by building configuration and the design code adopted. Irregular buildings, especially those with L-, C- and E-shaped plans, are known to exhibit complex dynamic behavior compared to regular-shaped structures. This study focuses on evaluating the seismic response of such irregular configurations using two widely adopted design standards: NBC 105:2020 (Nepal) and IS 1893:2016 (India). A comparative analysis was conducted through equivalent static method using ETABS software, assessing key parameters such as lateral displacement, inter-storey drift, and torsional irregularity. Four different building models—regular, L-shaped, C-shaped, and E-shaped—were analyzed under identical loading, material, and geometric conditions. The results revealed that irregular buildings consistently exhibited higher lateral displacement and drift, particularly at the re-entrant corners where stress concentrations were most severe. Among the codes, NBC 105:2020 produced notably higher drift values in all configurations compared to IS 1893:2016, indicating a more conservative approach. Torsional irregularities were also more pronounced in L- and C-shaped buildings under NBC, due to its stricter consideration of eccentricity and mass irregularity. Overall, the study highlights the critical impact of both structural configuration and seismic code selection on the response of buildings. It is concluded that while NBC 105:2020 provides a higher margin of safety, especially for irregular buildings, careful consideration of building geometry during the design phase remains essential for seismic resilience. The findings underscore the need for enhanced code-specific design provisions for irregular structures in high-seismic zones.

Keywords: Code comparison, horizontal irregularities, base shear, drift, torsion, displacement

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ACRONYMS

SYMBOLS	MEANING
A_c	Area of concrete
A_g	Gross area of the section
A_{sc}	Area of compression steel
A_{st}	Area of the tensile section
A_{st1}	Area of balanced tensile steel
A_{st2}	Area of tensile steel in excess of the balanced steel
A_{sv}	Area of vertical stirrups
B	Breadth of beam or shorter dimension of rectangular column
BM	Bending moment
B_w	Breadth of beam
D	Overall depth of the beam or slab longer dimension of column
D	Effective depth of the bar
d'	Effective cover
DL	Dead load
E_c	Young's modulus of elasticity of concrete
EL	Earthquake load
E_{min}	Minimum eccentricity
E_s	Young's modulus of elasticity of steel
E_x, E_y	Eccentricity about X and Y axis respectively
I	Importance factor of the structure
I_{xx}, I_{yy}	Moment of inertia about X and Y axis respectively
K	Performance factor depending upon the structural framing system and for brittleness or ductility of the construction
l	Unsupported length or clear span of elements
L_{eff}	Effective length of element
LL	Live load
L_x	Span of the slab in the shorter direction
L_y	Span of the slab in the longer direction
M_u	Factored moment, designed moment for limit state design
M_{ux}	Factored moment axis about X-X axis
M_{ux1}	Maximum uni-axial moment capacity of the section with axial load, bending about X-axis

M_{uy}	Factored moment axis about Y-Y axis
M_{uy1}	Maximum uni-axial moment capacity of the section with axial load, bending about Y- axis
P_c	Percentage of compression reinforcement
P_t	Percentage of tension reinforcement
P_{uz}	factored axial load, designed axial load for the limit state design
P_z	Axial load in the element
Q_i	Base shear distributed in the i^{th} floor
S	Spacing of the main bar
S_v	Spacing of stirrup
T_a	Estimated natural or fundamental time period of the building in seconds
V	Shear force
V_b	Total base shear
V_u	Design shear force for limit state, factored shear force
W_i	Lump load on the i^{th} floor
$X_{u,lim}$	Depth of the natural axis at the limit state of the collapse
A_h	Design horizontal seismic coefficient
α_x, α_y	Coefficient for moment in slab
σ_{ck}	Characteristics compressive strength of concrete
σ_{max}	Maximum stress
σ_{min}	Minimum stress
σ_y	Characteristics yield strength of steel
τ_{bd}	Design bond stress
τ_c	Shear strength of concrete
ϕ	Diameter of the bar
T	Target
P	Projected

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Seismic forces significantly impact the safety and performance of buildings, especially in earthquake-prone countries like Nepal and India. One of the key factors influencing seismic response is horizontal irregularity, which includes non-uniform building shapes in plan such as L-shaped, U-shaped, and E-shaped structures. These irregular configurations cause uneven distribution of stiffness and mass, which results in stress concentrations and enhanced torsional effects during ground motion (Agarwal and Shrikhande, 2010).

To address these concerns, national seismic design codes such as NBC 105:2020 in Nepal and IS 1893 (Part 1):2016 in India provide criteria for identifying and analysing horizontal irregularities. Both codes define types of plan irregularities, including torsional irregularity, re-entrant corners, diaphragm discontinuities, and non-parallel lateral force-resisting systems. For torsional irregularity, both codes state that it exists when the maximum storey displacement at one end exceeds 1.5 times the minimum displacement at the other end of the same storey (NBC 105:2020 and IS 1893:2016).

Previous studies highlight that buildings with horizontal irregularities often experience higher seismic demand compared to regular buildings. According to Adhikari et al. (2020), L- and U-shaped buildings show significantly increased lateral displacement and torsional response. Goyal and Choudhury (2019) emphasized that torsional effects are especially critical when such buildings are analysed using simplified methods like the equivalent static approach. Sharma and Khandelwal (2017) found that NBC 105:2020 generally leads to higher base shear and more conservative designs than IS 1893, due to differences in seismic zoning and design assumptions.

Despite these observations, limited research has been conducted that directly compares NBC 105:2020 and IS 1893:2016 in the context of horizontally irregular structures under a consistent modelling approach. Therefore, this study aims to fill that gap by evaluating and comparing the seismic behaviour of regular, L, U, and E-shaped buildings as per the respective provisions of both codes.

Site and Building's Description

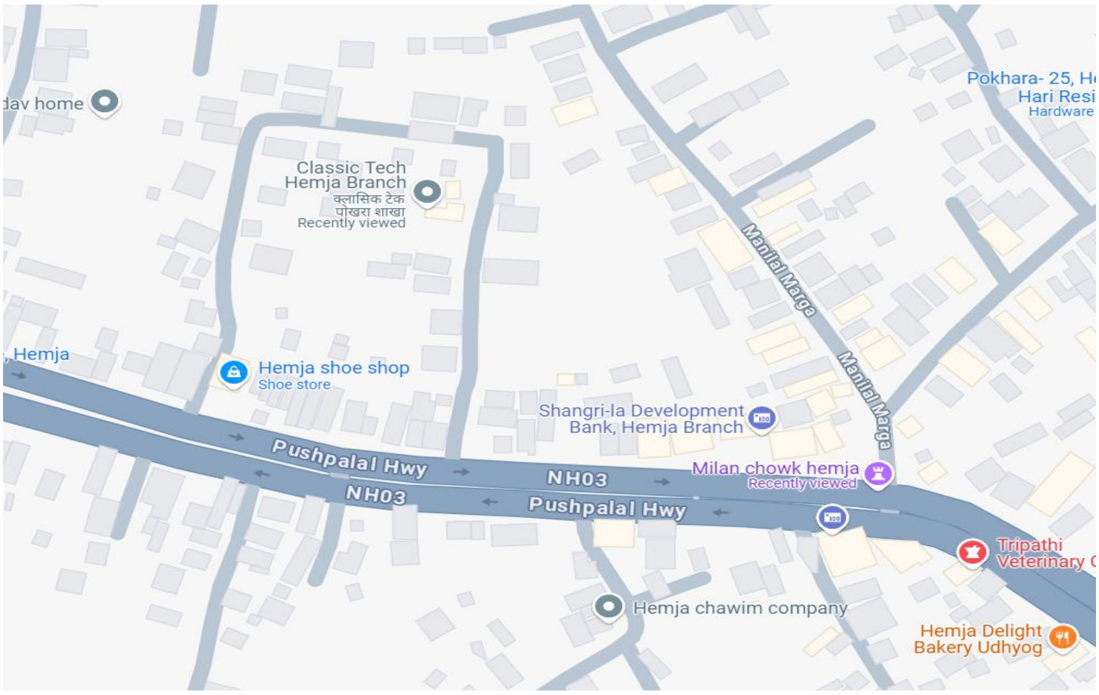

Fig 1: Site Location source: Google maps

Apartment

The site of the project is selected in Milan Chowk, Pokhara where our proposed building is designed as a multi-storied apartment building. It is a sixth storey building with earthquake resisting property in focus. The salient features of our project are as follows:

- Type of building : Medium Rise Apartment Building (as per NBC 105:2020)
- Location : Structure system: Special RC Moment Resisting frame structure (SMRF)
- Plinth area : 407.63 sq.m
- No. of storey : 6 storey (5 storey + staircase cover)
- Floor to floor height: 3m for all floors
- Materials : Cement, Brick, Sand, Rebar etc.
- Characteristic strength of Concrete: M25 for column, beam, slab, foundation
- Steel grade (f_y) : Fe500
- Soil Type : Medium Soil with SBC of 150 KN/m^2

ARCHITECTURAL REQUIREMENTS FOR APARTMENT

Table 1: Architectural requirement for apartment

PARTICULARS	DESCRIPTION
Maximum Floor Area Allowances per Occupant	
Max. area per occupant (Sq.m)	18
Minimum Occupants per 100 Sq.m	6
Exit Door/s width	
Minimum Width (m)	1
Occupant per unit 500mm width	75
No of occupants when exit medium width exceeds the min limit	150
Width of the corridors, Passageways, Staircase and Ramps	
Minimum Width (m)	1.2
Occupant per 500mm width (For ramps add 10 occupants)	25
No of Occupants when stairway width exceeds the min limit	60
Min floor height (m)	2.4
Min height of parapet (m)	1
Min height of basement floor (m)	2.4

1.2 STATEMENT OF PROBLEM

Although seismic codes like NBC 105:2020 and IS 1893:2016 discourage the use of horizontally irregular buildings due to their poor seismic performance, such irregular forms are still commonly used in practice for functional or architectural reasons. This mismatch between codal intent and construction practice can compromise structural safety.

Moreover, the two codes differ in their treatment of irregularities, which may lead to variations in design and seismic response. There is limited comparative analysis of how these codes address horizontal irregularities in terms of drift, displacement, and torsion. This study aims to fill that gap by analysing and comparing the behaviour of irregular buildings under both codes.

1.3 OBJECTIVE

General Objective

The main objective of the project is to compare seismic codes NBC 105:2020 and IS 1893:2016 based on horizontal irregularities.

Specific Objectives

1. To prepare four architectural plans, one regular shaped apartment building and three buildings with horizontal irregularities (L shaped, C shaped and E shaped).
2. To prepare structural plan for all buildings (modelling).
3. To analyse and compare key parameters such as base shear, torsion, storey drift, displacement, reinforcement percentage etc. of regular and irregular buildings based on NBC and IS codes.

1.4 SCOPE OF PROJECT

The scope of the study encompasses various aspects in functional planning and structural analysis of building with NBC and IS using ETABS.

The comparative analysis using NBC and IS will be for the four buildings (5 storey + staircase cover); one is the designed to be building named Regular building and others are the buildings having horizontal irregularities, i.e, L shaped building, C shaped building and E shaped building. The research uses Equivalent Static Method for analysis and it will evaluate how each code handles torsional irregularities. The soil type is considered "soil type D" for NBC and "soil type III" for IS. This project outlines the approach to utilizing different plans, for seismic analysis and comparison (NBC and IS) along with detailed design of an apartment building.

1.5 LIMITATIONS

Soil test is not carried out for the bearing capacity of soil. (Bearing capacity = 150KN/m^2, assumed)

CHAPTER 2: LITERATURE REVIEW

2.1 GENERAL

Building means a place where we live. It provides us the space to do works and reflects the social, economic, cultural and other aspects of the whole area. Apartment buildings provide housing facilities to different families living independently of each other in a same building. Since shelter is one of basic human needs, apartment buildings are solely focused on providing shelter accommodations and related services to many families simultaneously. Lack of development pace in rural parts of Nepal has resulted people to migrate into urban areas to get better opportunities and improved quality of life. Better opportunities of self-employment, health facilities, education and other facilities is attracting majority of rural population to migrate. Although the population keeps increasing in urban areas, there is limited land to provide housing facilities. In order to cater growing population's housing needs, multi-storied buildings in the form of apartment housings are being developed.

Furthermore, Nepal is situated in sub-duction zone of two tectonic plate called Indian and Tibetan plate. The Indian plate is moving towards the Tibetan plate 2cm per year. The existence of Himalayan range with the world's highest peak is the evidence of high tectonics beneath the country. As a result, Nepal lies in an active seismic zone. Thus, Pokhara is prone to earthquake hazards too since Pokhara lies on the zone V, the severest one. Hence, the effect of earthquake is pre-dominant than the wind load. So, the design of earthquake resistance structure is mandatory.

An earthquake is a sudden movement of the Earth caused by the abrupt release of strain that has accumulated over a long period of time. Earthquakes last a few seconds but, in that time, it can change people's life forever. We cannot prevent an earthquake from happening but we can prepare for one.

SOME TERMS

- Base shear: Total design lateral force or shear force due to earthquake at the base of a structure
- Ductility: Capacity of a structure, or its members to undergo large inelastic deformations without significant loss of strength.
- Ductility factor: The ratio of ultimate displacement demand to yield displacement demand.
- Eccentricity: The distance between the centre of mass and centre of stiffness.
- Intensity of earthquake: It is a measure of the severity of ground shaking at a particular site due to an earthquake
- Interstorey drift: Relative displacement of adjacent floors.
- Peak ground acceleration (PGA): Maximum acceleration of a particular direction of the ground motion.
- Centre of mass (CM): The point in a floor through which the resultant of the mass passes.
- Centre of stiffness/rigidity (CR): The point in a floor at which the resultants of the resisting forces in the two orthogonal directions intersect.

Seismic design codes provide standardized guidelines to ensure structural safety against earthquake forces. India's IS 1893:2016 (Part 1) and Nepal's NBC 105:2020 are the most up-to-date seismic codes in their respective countries. Both codes have undergone significant revisions in recent years to incorporate modern earthquake engineering principles and region-specific seismic risks. This literature review summarizes key comparative research on these two codes, focusing on their seismic design provisions and structural behaviour predictions.

IS 1893:2016 is part of India's updated seismic design framework. It emphasizes:

- Zone factors based on Deterministic Seismic Hazard Analysis (DSHA)
- Site-specific response spectra
- Importance factor categories tailored to occupancy risk
- Ductile detailing (linked with IS 13920)
- Introduction of vertical irregularities and torsional irregularities explicitly

NBC 105:2020 is a significant overhaul of the earlier NBC 105:1994. Its highlights include:

- Updated seismic zoning map of Nepal with five zones based on Probabilistic Seismic Hazard Analysis (PSHA)
- Use of spectral acceleration based on site class (Soil Type I, II, III)
- Introduction of ductility-based design concepts.
- Refined response spectra and design base shear equations.
- Revised load combinations aligning with performance-based design.

Adhikari & Poudel (2023) conducted a direct comparison of IS 1893:2016 and NBC 105:2020 on two and three-storey RC buildings using Equivalent Static Analysis (ESA) in ETABS. They concluded:

- NBC 105:2020 produced higher storey displacements and inter-storey drifts.
- Time periods calculated using NBC were longer, reflecting the updated site-dependent response spectra.
- Reinforcement demands were higher under NBC due to increased base shear and ductility requirements.

These findings show NBC 105:2020 to be more conservative in estimating seismic demand than IS 1893:2016, particularly for mid-rise buildings.

Pokharel et al. (2023) compared the seismic design of a three-storey residential building using the Response Spectrum Method under both IS 1893:2016 and NBC 105:2020. Key outcomes:

- Base shear from NBC 105:2020 was consistently 10–25% higher than IS 1893:2016 for the same building configuration and site class.
- Drift and displacement values were higher in NBC, reflecting the greater spectral accelerations considered in Nepal’s updated seismic zones.
- Despite increased structural demand, cost increase was only 5.5%, indicating better safety-to-cost efficiency with NBC 105:2020.

In a G+8 RC building comparison, Gwaccha et al. (2024) analysed IS 1893:2002, NBC 105:1994, and NBC 105:2020. Although the IS version was older, findings provide valuable context:

- NBC 105:2020 yielded 2x higher values for base shear and inter-storey drift compared to IS 1893:2002.
- The increase was due to the revised seismic coefficients, soil amplification factors, and ductility modifications in NBC 2020.
- The study emphasized that NBC 2020 better addresses local ground motion behaviour and soil types typical of the Kathmandu valley.

Dhital & Rathore (2022) studied a G+7 mass irregular RC frame using both IS 1893:2016 and NBC 105:2020:

- NBC predicted greater torsional irregularities and lateral displacement under the same loading.
- IS 1893 was found to be less conservative, possibly due to less detailed soil classification and simplified response spectrum.

This highlights the importance of soil-structure interaction and mass distribution in determining the accuracy of code-based predictions.

Pandit (2023) compared a high-rise (G+21) building using both codes under three soil types:

- On soft soil (Type III): NBC 105:2020 resulted in higher base shear and drift values.
- On medium or stiff soil: IS 1893:2016 sometimes produced higher demands due to differences in base shear formula.
- NBC 2020 showed higher ductility-based detailing, requiring greater reinforcement in beams and columns

This variation emphasizes how site class definition and local hazard zonation affect structural response more prominently in NBC 105:2020.

Sah (2020) studied buildings of various heights and observed:

- NBC 105:2020 uses updated load combinations more aligned with international standards (e.g., IBC, ASCE 7).
- IS 1893:2016 is simpler in form but lacks Nepal-specific seismic details, especially for soil amplification.
- NBC produced longer time periods (up to 56%) due to use of more refined response spectra for flexible structures.

In conclusion, the reviewed studies consistently show that NBC 105:2020 is more conservative than IS 1893:2016 in predicting seismic demands such as base shear, displacement, drift, and reinforcement. NBC's updated zoning, local soil classifications, and enhanced ductile detailing provisions result in safer, though slightly more expensive, structural designs. IS 1893:2016 remains a robust and globally respected code, but may underpredict demands in regions with high seismic risk and complex soil conditions like Nepal.

As seismic hazard awareness grows, adopting context-specific codes like NBC 105:2020 especially for critical and irregular structures ensures resilience against future earthquakes.

2.2 PRINCIPLES OF EARTHQUAKE-RESISTANT DESIGN OF RC MEMBERS

The lateral loads used in seismic design are highly unpredictable. In case of strong earthquakes, the magnitudes of lateral loads experienced by buildings are so large that an elastic design under these loads yields very large size of members. Thus, the structural members cost so much more that the risk of buildings going beyond their elastic limit is accepted with the stipulation that they do not fall down or collapse. The collapse of RC buildings is generally preventable if the following principles of earthquake-resistant design are observed.

- Failure should be ductile rather than brittle—ductility with large energy dissipation capacity (with less deterioration in stiffness) must be ensured.
- Flexure failure should precede shear failure.
- Beams should fail before columns.
- Connections should be stronger than the members that fit into them.

The structure should have the following characteristics:

- have a direct and continuous load path
- be simple and symmetrical
- not be too elongated in plan or elevation, i.e., the size should be moderate
- have uniform and continuous distribution of strength, mass, and stiffness
- have horizontal members which form hinges before the vertical members
- have sufficient ductility
- have stiffness related to the sub-soil properties.
  source: earthquake resistant design of structures (Shasikant K. Duggal)

2.3 RCC Design Philosophy

For the design of RC structures, there are three major design philosophies namely:

2.3.1 WORKING STRESS METHOD

The Working Stress Method is a method of designing reinforced concrete structures that assumes that the concrete and steel behave in a linear elastic manner. This means that the stresses in the materials are directly proportional to the strains, and that the materials do not exhibit any plastic or brittle behavior.

The WSM is a relatively simple method to use, and it is still widely used in practice today. However, it has some limitations, such as the fact that it does not consider the nonlinear behavior of concrete and steel.

Assumption of WSM

I. **Plane section remains plane after bending:** This assumption means that the cross-section of a beam does not change shape after it is bent.

II. **Strain compatibility:** This assumption means that the strains in the concrete and steel are equal at any point in the beam.

III. **Linear elastic behavior of concrete and steel:** This assumption means that the stress-strain relationship for concrete and steel is linear up to the point of failure.

IV. **Tensile strength of concrete is ignored:** This assumption means that the tensile strength of concrete is considered to be zero.

The WSM is a relatively conservative method of design, which means that it tends to overestimate the stresses in the materials. This can lead to the use of more reinforcement than is actually necessary, which can increase the cost of the structure.

Steps in the WSM Design Process

The WSM design process can be summarized in the following steps:

I. Determine the loads acting on the structure.
II. Calculate the stresses in the concrete and steel due to the applied loads.
III. Compare the calculated stresses to the permissible stresses for the materials.
IV. If the calculated stresses are greater than the permissible stresses, then the reinforcement must be increased.
V. Repeat steps 1- 4 until the calculated stresses are within the permissible limits.

Factors of Safety

The WSM uses factors of safety to ensure that the structure is safe. The factors of safety are applied to the permissible stresses to account for the uncertainty in the material properties and the loading conditions.

The factors of safety used in the WSM are typically as follows:

I. Concrete: 1.5
II. Steel: 1.2

However, the WSM is a simple and easy-to-use method, and it is still widely used in practice today. Here are some of the advantages and disadvantage of using the WSM:

Advantage:

I. It is a simple and easy-to-use method.
II. It is relatively conservative, which means that it tends to overestimate the stresses in the materials.
III. It is still widely used in practice today.

Disadvantage:

I. It does not consider the nonlinear behavior of concrete and steel.

II. It can lead to the use of more reinforcement than is actually necessary.

III. It is not as accurate as other methods of design, such as the Limit State Method.

2.3.2 ULTIMATE LOAD METHOD

The Ultimate Load Method (ULM) is a method of designing reinforced concrete structures that assumes that the structure will not fail under any combination of loads that is likely to occur during its lifetime.

The ULM considers the nonlinear behavior of concrete and steel, which means that the stresses in the materials are not directly proportional to the strains. This makes the ULM more accurate than the Working Stress Method (WSM), which assumes that the behavior of concrete and steel is linear elastic.

Assumption of ULM

I. Concrete and steel behave in a non-linear elastic manner.

II. The plane section before bending remains plane after bending.

III. There is strain compatibility between steel and concrete.

IV. The ultimate load is calculated by multiplying the working load by a load factor.

The ultimate load of the structure is calculated by multiplying the working load by a load factor. The load factor is a number that is greater than 1 and that accounts for the uncertainty in the material properties and the loading conditions.

Steps in the ULM Design Process

The ULM design process can be summarized in the following steps:

I. Determine the loads acting on the structure.

II. Calculate the ultimate load of the structure.

III. Compare the ultimate load to the working load.

IV. If the ultimate load is greater than the working load, then the structure is safe.

V. If the ultimate load is less than the working load, then the reinforcement must be increased.

Factors of Safety

The ULM uses load factors to ensure that the structure is safe. The load factors are applied to the working load to account for the uncertainty in the material properties and the loading conditions.

The load factors used in the ULM are typically as follows:

I. Dead load: 1.2
II. Live load: 1.5
III. Wind load: 1.6
IV. Earthquake load: 2.0

The ULM is a more accurate method of designing reinforced concrete structures than the WSM. However, it is also more complex to use. The choice of which method to use depends on the specific project and the level of accuracy required. Here are some of the advantages and disadvantage of using the ULM:

Advantage:

I. It considers the nonlinear behaviour of concrete and steel.
II. It is more accurate than the WSM.
III. It can lead to the use of less reinforcement than is necessary.

Disadvantage:

I. It is more complex to use than the WSM.
II. It can be more expensive to design a structure using the ULM.

2.3.3 LIMIT STATE METHOD

The Limit Load Method (LLM) is a method of designing reinforced concrete structures that is based on the concept of limit states. A limit state is a condition in which the structure is no longer able to perform its intended function. The LLM ensures that the structure will not reach any limit state under any combination of loads.

The LLM considers the nonlinear behavior of concrete and steel, which means that the stresses in the materials are not directly proportional to the strains. This makes the LLM more accurate than the Working Stress Method (WSM), which assumes that the behavior of concrete and steel is linear elasti

Assumptions of LLM

I. Concrete and steel behave in a non-linear elastic manner.
II. The plane section before bending remains plane after bending.
III. There is strain compatibility between steel and concrete.
IV. The limit load is calculated by considering the possibility of failure of the structure.

The limit loads are calculated by considering the possibility of failure of the structure. This means that the limit loads are calculated by considering the maximum stresses that the materials can withstand.

Steps in the LLM Design Process

The LLM design process can be summarized in the following steps:

I. Determine the loads acting on the structure.
II. Calculate the limit load of the structure.
III. Compare the limit load to the working load.
IV. If the limit load is greater than the working load, then the structure is safe.
V. If the limit load is less than the working load, then the reinforcement must be increased.

Factors of Safety

The LLM uses load factors and partial safety factors to ensure that the structure is safe. The load factors are applied to the working load to account for the uncertainty in the material properties and the loading conditions. The partial safety factors are applied to the material properties to account for the uncertainty in the material properties.

The load factors and partial safety factors used in the LLM are typically as follows:

I. Dead load: 1.2
II. Live load: 1.5
III. Wind load: 1.6
IV. Earthquake load: 2.0
V. Concrete strength: 1.5
VI. Steel strength: 1.2

The LLM is a more complex method to use than the WSM, but it is also more accurate. The LLM can lead to the use of less reinforcement than is necessary, which can be more economical in the long run. Here are some of the advantages and disadvantage of using the LLM:

Advantages of LLM:

I. It considers the nonlinear behaviour of concrete and steel.
II. It is more accurate than the WSM.
III. It can lead to the use of less reinforcement than is necessary.
IV. It is more rational and scientific than the WSM.

Disadvantages of LLM:

I. It is more complex to use than the WSM.
II. It can be more expensive to design a structure using the LLM. There are two main types of limit states in structural design:

1. Ultimate/Collapse limit state (ULS):

This is the condition in which the structure is no longer able to carry the loads that are acting on it. This can happen due to the failure of one or more structural members, or due to the overall instability of the structure. This ultimate limit state may correspond to:

a. Flexure,
b. Compression,
c. Shear, and
d. Torsion

2. Serviceability limit state (SLS):

This is the condition in which the structure no longer performs its intended function. This can happen due to excessive deflection, cracking, or vibration. The ULS is the most critical limit state, and it is the one that is most likely to occur. The SLS is less critical than the ULS, but it is still important to ensure that the structure meets the requirements of its intended use.

This ultimate limit state may correspond to:

a. Deflections
b. Cracking

c. Vibration

In addition to the ULS and SLS, there are other limit states that may be considered in structural design, such as:

i. Fatigue limit state:

This is the condition in which the structure fails due to repeated loading.

3. Progressive collapse limit state:

This is the condition in which the failure of one structural member leads to the failure of other members, resulting in the collapse of the entire structure.

Environmental limit state:

This is the condition in which the structure fails due to environmental factors, such as wind, snow, or earthquakes.

The choice of which limit states to consider depends on the specific project and the level of risk that is acceptable. For example, a bridge that carries heavily trafficked roads would need to be designed to meet more stringent limit states than a pedestrian bridge.

2.4 SEISMIC DESIGN PHILOSOPHY

Seismic design philosophy is the set of principles that guide the design of structures to withstand earthquakes. The goal of seismic design is to ensure that structures remain standing and functional after an earthquake, even if they experience some damage.

There are two main seismic design philosophies:

a. Strength-based design

Strength-based design is based on the principle of ensuring that the structure is strong enough to withstand the forces of an earthquake without collapsing. This is achieved by using strong materials and by designing the structure to be able to distribute the forces of an earthquake evenly.

b. Ductile design

Ductile design is based on the principle of allowing the structure to deform (or bend) during an earthquake without collapsing. This is achieved by using ductile materials, such as steel, and by designing the structure to have a certain amount of flexibility. When a ductile structure is subjected to an earthquake, it will bend and absorb the energy of the earthquake, preventing it from collapsing.

The choice of which seismic design philosophy to use depends on the specific project and the level of risk that is acceptable. For example, a critical infrastructure facility, such as a nuclear power plant, would need to be designed using a strength-based design philosophy, while a less critical structure, such as a residential building, could be designed using a ductile design philosophy.

In addition to the two main seismic design philosophies, there are also a number of other factors that can be considered in the design of earthquake-resistant structures.

These factors (Also Known as Principle of Earthquake Resistance Design) include:

a. Lightness:

Since the earthquake force is a function of mass, the building shall be as light as possible consistent with structural safety and functional requirements. Roofs and upper stores of buildings, in particular, should be designed as light as possible.

b. Good Structural Configuration

The behavior of a building during earthquake depends on its shape, size and geometry. A good building configuration can result in less damage during earthquakes. The inertial forces need to be transferred in a smooth and direct manner without any bends in the forces. Buildings with rectangular plans and regular verticality have a good chance of withstanding an earthquake. The various components of building configuration are:

i. **Symmetry:**

The building should have a simple rectangular plan and be symmetrical both with respect to mass and rigidity so that the centers of mass and rigidity of the building coincide with each other in which case no separation sections other than expansion joints are necessary. If symmetry of the structure is not possible in plan, elevation or mass, provision shall be made for torsional and other effects due to earthquake forces in the structural design or the parts of different rigidities may be separated through crumple sections. The length of such building between separation sections shall not preferably exceed three times the width.

ii. **Simplicity and Regularity:**

The building should have a simple rectangular plan. It is seen that simple shapes behave better during earthquake than complex shapes like L, T, E, H, U, T, etc. It is seen that during earthquakes the buildings with re-entrant comers have suffered great damage. These types of buildings can be broken into rectangular blocks which are separated properly. Thus, separation of a large building into smaller blocks can lead to symmetry and irregularity. For preventing pounding
or hammering between blocks, a separation of 3 to 4 cm throughout the height above plinth level is required. This separation section is just like an expansion joint or it may be filled with a weak material which can easily crush during earthquake shaking.

iii. **Simple Building without much projections and suspended parts behave well during earthquake:**

Long cornices, vertical or horizontal projections, facial stones etc. should be avoided and are dangerous during earthquakes. If these parts cannot be avoided, they should be reinforced properly and tied firmly to the main structure. Ceiling plaster should not be done and if done, it should be thin as possible.

iv. **Size of the Building:**

In tall buildings, the horizontal movement of the floors during ground shaking is large. In short, but very long buildings the damaging effects of earthquake are more. Buildings with one of their dimensions much larger or smaller than the other two do not perform well during earthquakes. Thus, the buildings length should not be more than three times its width. If longer lengths are needed, two separate blocks with separation should be provided.

v. **Enclosed Area:**

A small building with properly interconnected walls acts like a rigid box and more earthquake resistant. Therefore, it is advisable to have separate small rooms than one long room.

vi. **Lateral Strength**

Lateral strength is the maximum resistance that a building can give when subjected to lateral loading without collapse.

vii. **Adequate Stiffness**

Lateral stiffness is the initial stiffness of a building and must be adequate enough such that low to medium shaking doesn't lead to damage.

viii. **Good Ductility**

Ductility is the ratio of maximum deformation to the idealized yield deformation. It is a measure of the capacity to undergo large plastic deformations under severe excitation. Ductile behavior allows for structures to dissipate large amounts of energy.

ix. **The location of the structure.**

Structures that are located in areas with a high seismic risk will need to be designed to withstand more severe earthquakes than structures that are located in areas with a low seismic risk.

x. **The materials used in the construction of the structure.**

Some materials, such as steel, are more ductile than other materials, such as concrete. This means that structures that are made of steel are more likely to survive an earthquake than structures that are made of concrete.

xi. **The type of structure.**

Some types of structures, such as tall buildings, are more vulnerable to earthquakes than other types of structures, such as single-family homes.

xii. **Stability of Slope:**

Hill side slopes are liable to slide during an earthquake. Hence buildings should not be constructed on them, only stable slopes should be chosen. Similarly, buildings with unequal members will also twist underground movement and may result in damage and collapse.

xiii. **Foundation**:

The buildings should not be constructed on loose soils. These soils will compact and subsidies and result in unequal settlement of building and damage it. Although such soil can be compacted properly for small buildings. But for large buildings this operation is costly. For large buildings, rigid raft foundation can be used.

In conclusion, the seismic design philosophy that is chosen for a particular structure will depend on a number of factors, including the location of the structure, the type of structure, and the materials used in the construction of the structure. The goal of seismic design is to ensure that structures remain standing and functional after an earthquake, even if they experience some damage.

2.5 Earthquake Design Philosophy

Every building structure is expected to experience moderate to major earthquake during its life period. Design of building structure should follow the principle of earthquake resistant design. A well design earthquake resistant building should be able to-

- Structure should possess at least a minimum strength to resist minor earthquake without damage.
- Structure should be able to resist moderate earthquake without significant structural damage through some non-structural damage.
- Structure should be able to resist major earthquake without collapse through structural damage and non-structural damage.
  as per IS 1893: part 1 2016 (Cl 6.1)

2.6 Seismic Analysis Methods

I. Equivalent Static Method:

ESM maybe used when at least one of the following criteria is satisfied:

i. The height of structure is less than or equal to 15m.

ii. The natural time period of the structure is less than 0.5 secs.

iii. The structure is not categorized as irregular and the height is less than 40m.

II. Modal Response Spectrum Method:

i. For buildings >40m or with irregular geometry.

ii. Considers dynamic behaviour and higher mode effects.

iii. Based on response spectrum curves provided in NBC 206.

III. Elastic Time History Analysis:

i. For critical structures or complex buildings.

ii. Uses actual/seismic ground motion data for detailed analysis.

iii. Requires advanced software and data.

IV. Push-Over Analysis:

i. Performance-based analysis for seismic capacity.

ii. Identifies weak points under increasing lateral loads.

iii. Used for retrofit or detailed evaluation.

Equivalent Static method is used for further analysis for our research.

2.7 Load Cases

For our project on apartment, the most important load cases to consider are the dead load, the live load, and the earthquake load. The dead load and the live load will be determined by the size and type of building, as well as the number of people who will occupy the building. The earthquake load will be determined by the seismic zone in which the building is located.

2.8 Load Patterns

Loading pattern from slab to beam is obtained by drawing 45-degree offset line from each comer. Then obtained trapezoidal as well as triangular loading and convert into equivalent UDL as described before.

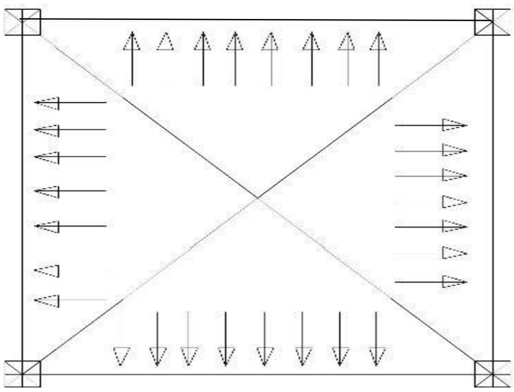

Figure 2:Load Distribution

2.9 Load Combination

Different load cases and load combinations are considered to obtain the most critical element stress in the structure in the course of analysis, there are altogether four load cases considered for the structural analysis and are mentioned as below:

I. Dead Load (DL)

II. Live Load (LL)

III. Earthquake Load in X Direction

IV. Earthquake Load in Y Direction

All the component and the structural frame should be designed/ checked for the following load combination each separately:

For Static Equilibrium (as per NBC 105:2020) (Cl. 3.6.1)

For parallel systems,

a. 1.2DL + 1.5 LL
b. DL + 0.3LL + EQX / ULS
c. DL + 0.3LL - EQX / ULS
d. DL + 0.3LL + EQY / ULS
e. DL + 0.3LL - EQY / ULS
f. DL + 0.3LL + EQX / SLS
g. DL + 0.3LL - EQX / SLS
h. DL + 0.3LL + EQY / SLS
i. DL + 0.3LL - EQY / SLS

As per IS 1893:2016, for parallel systems, (cl 6.4.3.1)

a. 1.2 (DL + LL + EQX)
b. 1.2 (DL + LL - EQX)
c. 1.2 (DL + LL + EQY)
d. 1.2 (DL + LL - EQY)
e. 1.5 (DL + EQX)
f. 1.5 (DL - EQX)
g. 1.5 (DL + EQY)
h. 1.5 (DL - EQY)
i. 0.9DL + 1.5EQX
j. 0.9DL - 1.5EQX
k. 0.9DL + 1.5EQY
l. 0.9DL - 1.5EQY

2.10 Codal Provisions

2.10.1 Base Shear

Base shear as per NBC 105:2020

According to NBC 105:2020 Base Shear is Defined as “Total design lateral force or shear force due to earthquake at the base of a structure.

Simply, base shear is the maximum expected lateral force that will occur due to seismic ground acceleration at the base of a structure. It is calculated using the seismic zone, soil material, and building code lateral force equations. Base shear is an important design consideration for all structures in seismically active areas, as it can cause significant damage or even collapse if not properly resisted.

The purpose of base shear is to distribute the lateral forces of an earthquake throughout the height of a structure. This helps to prevent the structure from overturning or collapsing. Base shear is typically applied at the base of a structure, but it can also be distributed to other parts of the structure, such as the roof or the foundation.

The amount of base shear that a structure is subjected to depends on a number of factors, including the seismic zone, the soil material, the building's height, and the building's structural system. Seismic zones are classified according to the likelihood of an earthquake occurring. Soil material can also affect the amount of base shear, as softer soils are more likely to amplify seismic waves. The height of a structure also plays a role, as taller structures are more susceptible to lateral forces. Finally, the structural system of a building can also affects the amount of base shear, as some structural systems are more effective at resisting lateral forces than others.

Base shear is an important design consideration for all structures in seismically active areas. By properly designing for base shear, engineers can help to ensure that structures will remain standing during an earthquake.

From clause 6.2; The horizontal seismic base shear, V, acting at the base of the structure in the direction being considered shall be calculated as:

$V_i = C_d(T_i) \times W$

where,

$C_d(T_i)$ = horizontal base shear co-efficient for each mode given by

$=\frac{C(T_i)}{R_\mu * \Omega_u}$

where,

$C(T_i)$ = elastic site spectra at period Ti as per NBC 105 Clause 4.1.1

Ti = fundamental period of the ith mode of vibration

R_μ= Ductility factor as per 5.3

Ω_u = Over Strength factor for ULS as per 5.4

W= Seismic weight= Dead Load + 30% of live load

Elastic Site Spectra

The elastic site spectra for horizontal loading shall be as given by

$C(T)=C_h(T)\ Z\ I$

where,

$C_h(T)$ = Spectral Shape factor

Z = Seismic Zoning factor (0.3 for Pokhara)

I = Importance factor

$T = k_t H^{\frac{3}{4}}$

T should be factored by 1.5

where, k_t=

0.075 for Moment resisting concrete frame

0.085 for Moment resisting structural steel frame

0.075 for Eccentrically braced structural steel frame

0.05 for all other structural systems

H = Height of the building from foundation or from top of a rigid basement.

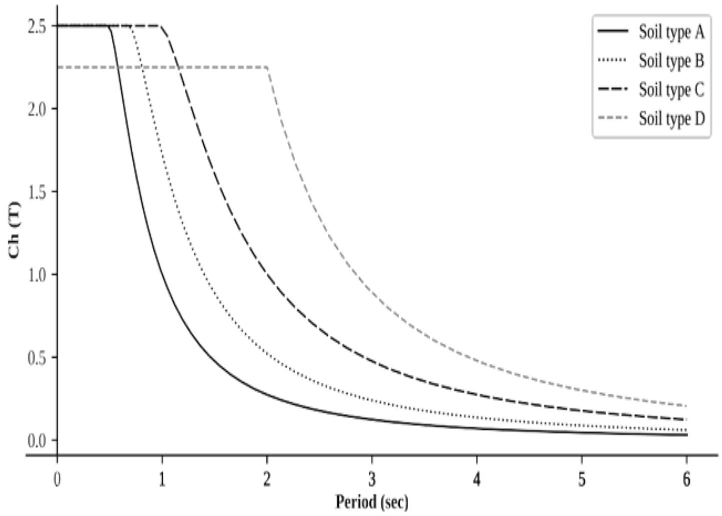

Figure 3: Spectral Shape Factor Source: NBC 105:2020

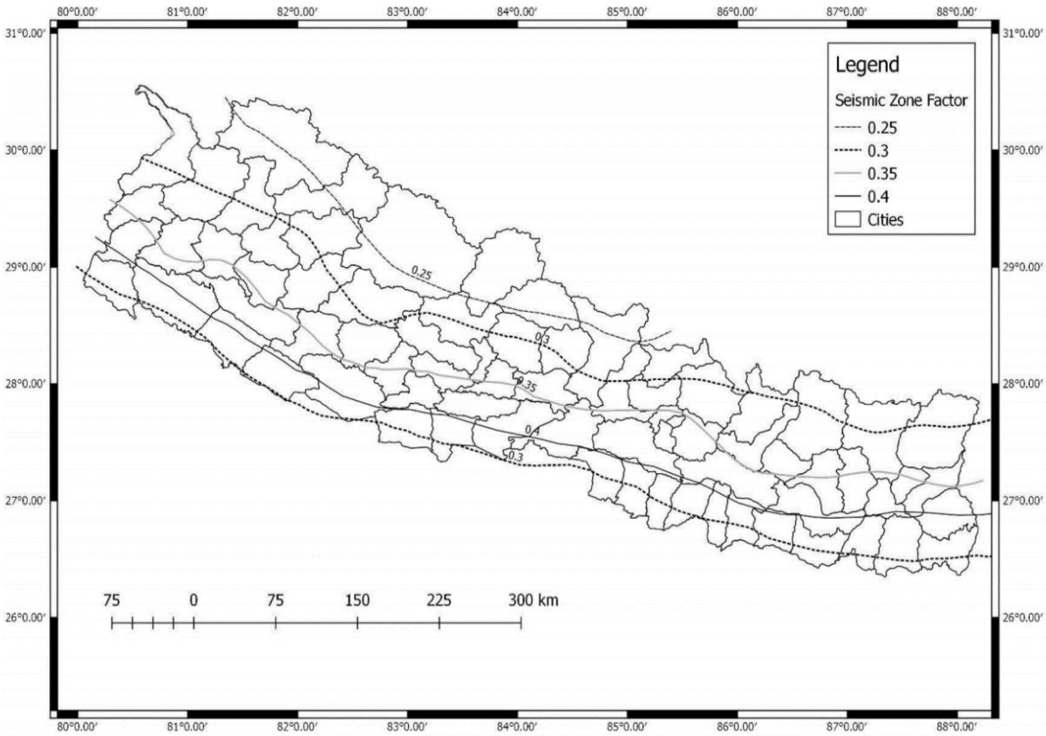

Fig 4: Seismic Zoning Map of Nepal Source: NBC 105:2020

Base shear as per IS 1893 (Part 1): 2016 (Cl 7.6.1)

From clause 7.6.1; The total design lateral force or design seismic base shear (V_B) along any principal direction shall be determined by the following expression:

$V_B = A_h \times W$

Where,

A_h= Design horizontal acceleration spectrum value as per 6.4.2, using the fundamental natural period T, as per 7.6 in the considered direction of vibration

$A_h = \frac{ZI}{R}(\frac{S_a}{g})$

Z= Zone Factor (as per Table 2, IS1893 Part 1:2002)

Table 2: Seismic Zone

Seismic Zone	**II**	**III**	**IV**	**V**
Seismic Intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

I= Importance Factor (as per Table 6)

Table 3: Importance Factor

SI No.	**Structure**	**Importance Factor**
1	Important service and community buildings, such as hospitals; schools; monumental structures; emergency buildings like telephone exchange, television stations, radio stations, railway stations, fire station buildings; large community halls like cinemas, assembly halls and subway stations, power stations	1.5
2	All other buildings	1.0

R= Response Reduction Factor (as per table 7, Cl 6.4.2).

$\frac{S_a}{g}$= Average Response Acceleration Coefficient

W= Seismic weight of the building as per 7.4
W= Full dead load + appropriate amount of imposed load

Table 4: Percentage of imposed load to be considered in calculation of seismic weight

SI. No	Imposed Uniformly Distributed Floor Loads (KN/m^2)	Percentage of Imposed Load
i)	Up to and including 3.0	25
ii)	Above 3.0	50

For $\frac{S_a}{g}$, use Cl 6.4.2

2.10.2 Centre of mass

According to NBC 105:2020, Center of mass is defined as "The point in a floor through which the resultant of the mass passes."

Simply, the center of mass (CM) of a structure is the point where the entire mass of the structure is assumed to be concentrated. In seismic analysis, the center of mass is used to determine the location of the inertia forces that are generated during an earthquake. These inertia forces are proportional to the mass of the structure and the acceleration of the ground. The location of the center of mass is important because it affects the distribution of the inertia forces throughout the structure.

If the center of mass is located near the base of the structure, the inertia forces will be concentrated near the base. This can lead to large bending moments and shear forces in the columns and foundation. On the other hand, if the center of mass is located near the top of the structure, the inertia forces will be concentrated near the top. This can lead to large bending moments and shear forces in the beams and slabs.

In general, it is desirable to have the center of mass located as low as possible in a structure. This is because it will help to distribute the inertia forces more evenly throughout the structure and reduce the risk of damage during an earthquake.

Here are some of the factors that can affect the location of the center of mass in a structure:

- The distribution of mass throughout the structure.

- The shape of the structure.
- The location of the lateral load resisting system.

The center of mass can be determined using a variety of methods, including:

I. Analytical methods.
II. Graphical methods.
III. Computer-aided methods.

The method that is used to determine the center of mass will depend on the complexity of the structure and the accuracy that is required. By calculating the static moments around a point or set of reference lines, the center of mass can be found.

$$x_m = \frac{\sum w_i L_i}{\sum W_i}$$

$$Y_m = \frac{\sum w_i B_i}{\sum W_i}$$

Where,

W = W (R) + W (N) + W (S) + We

2.10.3 Centre of Stiffness / Rigidity

According to NBC 105:2020, Center of stiffness or rigidity is defined as "the point in a floor at which the resultants of the resisting forces in the two orthogonal directions intersect."

The center of rigidity is the point in a structure where all the stiffness of the structure is concentrated. It is the point where, if a lateral load is applied, the structure will deflect the least. The center of rigidity is also known as the center of stiffness or the center of resistance.

The center of rigidity is calculated by taking the weighted average of the positions of all the rigid elements in the structure. The weights are determined by the stiffness of each element. For example, a column that is twice as stiff as a beam will have twice the weight in the calculation of the center of rigidity.

The center of rigidity is an important concept in structural engineering because it can be used to determine the distribution of lateral loads in a structure. If the center of rigidity is not located at the center of mass of the structure, then the lateral loads will cause the structure to rotate. This can lead to the structure becoming unstable.

In a beam, the center of rigidity is located at the centroid of the beam's cross-section. This is because the stiffness of a beam is proportional to the area of its cross-section. In a frame, the center of rigidity is located at the intersection of the centerlines of the most rigid members.

The center of rigidity is an important concept to understand for anyone who is involved in the design or analysis of structures. It can help to ensure that structures are designed to be stable and to resist lateral loads.

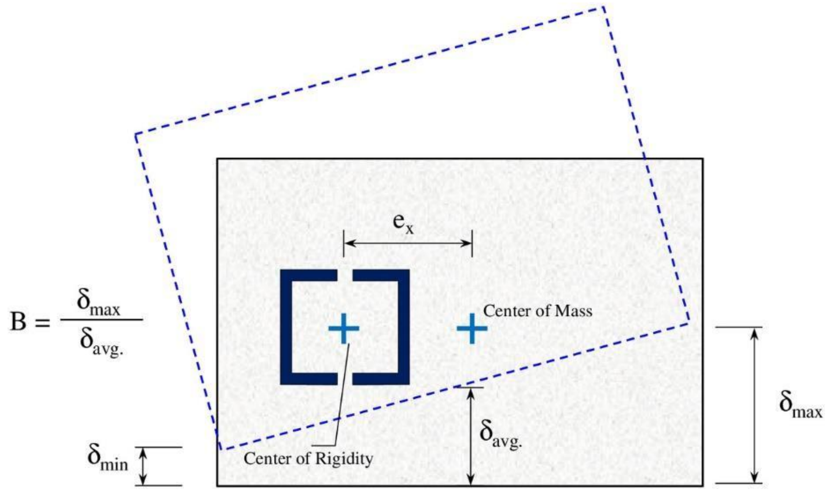

Figure 5: Centre of Mass and Centre of Stiffness/Rigidity

2.10.4 Storey Drift

Storey drift is the relative lateral displacement between two consecutive stories of a structure. It is a measure of the flexibility of a structure and its ability to withstand lateral loads, such as wind and earthquakes.

Story drift is calculated by dividing the lateral displacement of one story by the height of that story. The resulting value is typically expressed as a percentage or a ratio. For example, a story drift of 0.02 would mean that the lateral displacement of one story is 2% of the height of that story.

The calculation of story drift is relatively straightforward. However, the specific method used to calculate story drift can vary depending on the structural analysis software being used. In general, the following steps are involved in calculating story drift:

I. The lateral displacements of each story are calculated using a structural analysis software.
II. The story heights are determined.
III. The story drift is calculated by dividing the lateral displacement of each story by the height of that story.

The story drift limits for a structure are typically specified in the building code for the area where the structure is located. These limits are based on the seismic hazard of the area and the type of structure. For example, a structure in a high-seismic area will have lower story drift limits than a structure in a low-seismic area.

2.10.5 Torsion

Torsion is a twisting moment that can be applied to a structure. It is caused by lateral loads that are not evenly distributed across the plan of a structure. Torsion can cause significant damage to a structure, especially in seismically active areas.

In IS code, torsional irregularity is defined as a condition in which the plan of a structure is not symmetrical and the distribution of lateral loads is not uniform. Torsional irregularity can be caused by a number of factors, including the shape of the structure, the location of openings, and the presence of cantilevers.

There are two types of torsional irregularity:

Plan irregularity:

This type of irregularity is caused by asymmetries in the plan of a structure. For example, a building with a T-shaped plan would be considered torsionally irregular.

Elevation irregularity:

This type of irregularity is caused by asymmetries in the elevation of a structure. For example, a building with a sloping roof would be considered torsionally irregular.
Torsional irregularity can increase the seismic demand on a structure. This is because torsion can cause the structure to twist, which can lead to shear failures and damage to the structural elements.

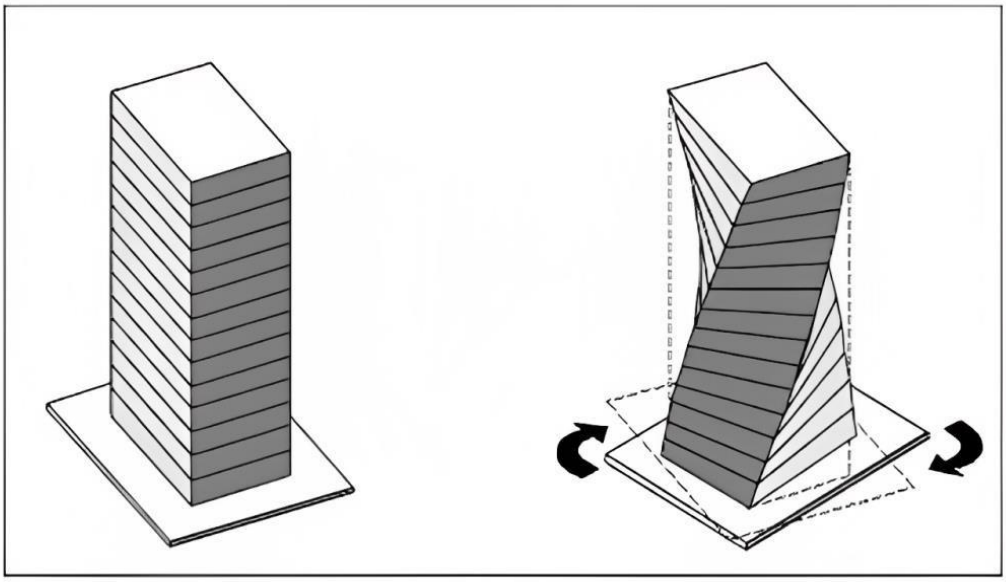

Figure 6: Torsion

2.10.6 Modal Mass Participating Ratio

According to NBC 105:2020, Modal Mass is defined as "Part of the total seismic mass of the structure that is effective in a specific mode of vibration."

Modal mass participating ratio (MMPR) is a measure of the relative contribution of a mode of vibration to the total mass of a structure. It is calculated as the ratio of the modal mass to the total mass of the structure. MMPR is used in structural analysis to determine the dynamic response of a structure to a given loading condition. The higher the MMPR, the more significant the contribution of the mode to the total response of the structure.

For example, if a structure has a MMPR of 0.5 for a particular mode, then that mode accounts for 50% of the total mass of the structure. This means that the mode will have a significant impact on the dynamic response of the structure to loads that excite that mode.

MMPR is also used to assess the seismic performance of structures. Structures with high MMPRs are more susceptible to damage in earthquakes, as they are more likely to vibrate in the modes with high MMPRs. The calculation of MMPR is relatively straightforward. However, the specific method used to calculate MMPR can vary depending on the structural analysis software being used. In general, the following steps are involved in calculating MMPR:

I. The mode shapes of the structure are calculated.
II. The modal masses of the structure are calculated.
III. The MMPR' s is calculated by dividing the modal masses by the total mass of the structure.

2.10.7 Lateral Load Resisting System

The first step in building architectural planning is to choose a lateral load resisting system. The load resisting system must have closed loops in order to transfer all vertical and horizontal forces to the ground. In the code IS 1893 (Part 1), the Bureau of Indian Standards (BIS) has approved three major types of lateral force resisting system: 2016. There are three types of systems: moment resisting building frame systems
bearing wall systems, and dual systems.

These systems are further subdivided into the materials used in construction. Further it can be classified into following types.

I. Moment Resisting Frame
II. Building with Shear Wall or Bearing Wall System
III. Building with Dual System

2.10.8 HORIZONTAL IRREGULARITY

Horizontal irregularities refer to discontinuities or irregular configurations in a building's horizontal (plan) layout, which may lead to torsional effects or concentration of seismic forces during an earthquake.

Types of Horizontal Irregularities (as per IS 1893 / IBC)

a) Torsional Irregularity:

- Occurs when the lateral displacement of one edge of the structure is significantly more than the opposite edge (typically, if the max drift is >1.2 times the average drift).
- Results in twisting or rotation during seismic motion.

b) Re-entrant Corners:

- Buildings with L, T, U, or similar shapes have re-entrant corners that can cause stress concentration and separation during seismic shaking.

c) **Diaphragm Discontinuity:**

- Occurs when the floor diaphragm (usually the slab) has abrupt changes in stiffness or has large openings, reducing its ability to distribute seismic forces.

d) **Out-of-Plane Offsets:**

- Discontinuity where lateral-force-resisting elements (like shear walls or frames) are not aligned vertically in the plan, leading to stress concentration.

e) **Nonparallel Systems:**

- Lateral-force-resisting systems not aligned with the major orthogonal axes of the building; causes torsional behaviour due to oblique force transmission.

f) **Impact on Seismic Performance:**

- Leads to uneven distribution of seismic forces, making the structure more susceptible to damage.
- Causes torsional motion, increasing demands on structural and non-structural elements.
- Requires detailed dynamic .

2.10.9 Torsion Irregularity:

As per NBC 105:2020 (Clause 5.5.2.1)

Torsion irregularity is considered to exist where the maximum horizontal displacement of any floor in the direction of the lateral force (applied at the center of mass) at one end of the story is more than 1.5 times its minimum horizontal displacement at the far end of the same story in that direction.

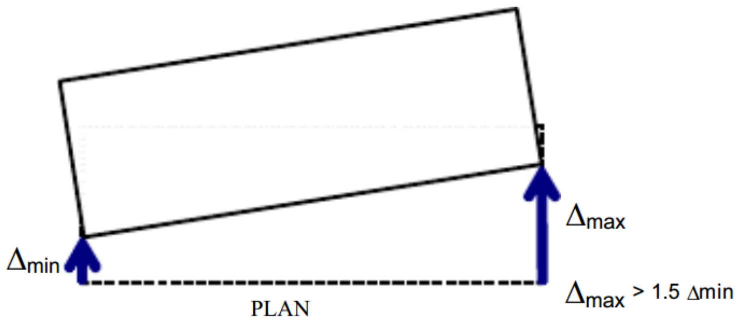

Fig 7 : Torsion Irregularity *Source: NBC 105:2020(Cl 5.5.2.1)*

As per IS 1893 Part I:2016 (Clause 7.6)

A building shall be considered to have torsional irregularity, if the following condition is satisfied:

I. The maximum relative displacement at any floor level in the direction of applied lateral force shall exceed 0.05 times the maximum displacement at the base (for buildings with a torsional irregularity).

II. The distance between the center of mass (CM) and the center of rigidity (CR) of the building (in any plan) is greater than 0.05 times the building dimension in the direction of the applied force.

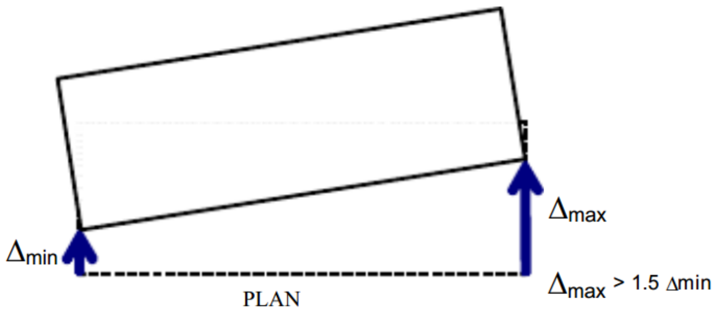

Fig 8: Torsion Irregularity *Source: IS 1893 Part I:2016 (Cl 7.6)*

2.11 DRIFTS AND DISPLACEMENTS

2.11.1 As per NBC 105:2020 (cl 5.6.1)

Determination of Design Horizontal Deflections

I. The design horizontal deflections shall be determined by multiplying the

Horizontal deflection found from Equivalent Static Method or Modal Response Spectrum Method by the Ductility factor (Ru).

II. Serviceability limit state

The design horizontal deflection for serviceability limit state shall be taken as equal to the horizontal deflections calculated either by Equivalent Static Method or Modal Response Spectrum Methods.

Inter-Story Deflections

The ratio of the inter-story deflection to the corresponding story height shall not exceed:
0.025 at ultimate limit state
0.006 at serviceability limit state (cl. 5.6.3)

2.11.2 As per IS 1893 Part I :2016 (Cl 7.11.1)

Maximum Drift:

The maximum interstory drift at any floor level, when subjected to the design lateral forces (earthquake loads), shall not exceed the following limits:

For buildings with reinforced concrete or steel frames, the interstory drift should not exceed 0.004 times the story height.

For buildings with other types of framing systems (e.g., masonry or unreinforced walls), the interstory drift should not exceed 0.005 times the story height.

2.12 STRUCTURAL COMPOMENTS

2.12.1 BEAMS

Requirements of this section shall apply to beams resisting earthquake-induced effects, in which the factored axial compressive stress does not exceed 0.08fck.

Beams, in which the factored axial compressive stress exceeds 0.08fck, shall be designed as per requirements of 4.2.

Dimensional Limits

- Beams shall preferably have width-to-depth ratio of more than 0.3.
- Beams shall not have width less than 200 mm.
- Beams shall not have depth D more than 1/4th of clear span.

- Width of beam shall not exceed the width of supporting member.

Longitudinal Reinforcement

- Beams shall have at least two 12 mm diameter bars each at the top and bottom faces.
- Minimum longitudinal steel ratio min required on any face at any section is:

$$\rho_{min} = 0.24\frac{\sqrt{f_{ck}}}{f_y}$$

- Maximum longitudinal steel ratio max provided on any face at any section is 0.025.

Transverse Reinforcement

- Only vertical links/stirrups shall be used in beams. Inclined links/stirrups shall not be used.
- Links/stirrups are permitted to be made of two pieces of bars also, namely a U-link with a 135° hook with an extension of 6 times diameter (but not less than 65 mm) at each end, embedded in the core concrete, and a cross-tie.
  (as per Cl 4.1 NBC 105:2020)

2.12.2 COLUMNS

Requirements of this section shall apply to columns resisting earthquake-induced effects, in which the factored axial compressive stress due to gravity and earthquake effects exceeds 0.08fck.

Dimensional Limits

- The minimum dimension of a column shall not be less than 20 do, where do is diameter of the largest diameter of longitudinal reinforcement bar in the beam passing through or anchoring into the column at the joint.
- The minimum dimension of column shall be 300 mm.
- Columns shall preferably have width-to-depth ratio of more than 0.45.

Longitudinal Reinforcement

- Circular columns shall have a minimum of 6 numbers of bars and rectangular columns shall have a minimum of 8 numbers of bars.
- Minimum longitudinal steel ratio ρ_{min} shall be 0.01.
- Maximum longitudinal steel ratio ρ_{max} shall be 0.04.
- Minimum diameter of the longitudinal bar shall be 12 mm.

Transverse reinforcement

- Transverse reinforcement shall consist of closed loops of spiral/ circular links/hoops for circular columns, and rectangular links/hoops for rectangular columns.
- The closed links/hoops shall have 135° hook ends with an extension of 6 times its diameter (but not < 65 mm) at each end, which are embedded in the confined core of the column.
- The minimum diameter of a link/hoop shall be 8 mm.
  (as per Cl 4.2 NBC 105:2020)

2.12.3 SLAB

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

i. One-way slab spanning in one direction - Length is more than twice the breadth.
ii. Two-way slab spanning in both directions.
iii. Circular slabs.
iv. Flat slabs (resting directly on columns).
v. Grid floor and ribbed slab.
vi. Slabs are designed using the same theories as beams, i.e., theories of bending and shear.

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

- The minimum span of a slab should not be less than four times the overall depth, and slabs are much thinner than beams.

- Slabs are analysed and designed as having unit width, i.e., 1m.
- Compressional reinforcement is used only in exceptional cases.
- Shear stress is very low in slab hence shear reinforcement is not provided, if needed, depth is increased rather than providing shear reinforcement.
  (as per NBC 105:2020)

CHAPTER 3: METHODOLOGY

This study involves a systematic comparison of the Nepal National Building Code (NBC) with Indian Standards on the basis of seismic analysis along with the detailed design of an apartment building using Nepal Building Code. The methodology comprises:

I. Architectural planning

The plans were collected from earlier batches as a reference and modifications were done as per need. The Regularbuilding was further modified to create plan irregularities (i.e C shaped, E shaped and L shaped). Functional planning was done following NBC 206 using AUTOCAD 2020.

II. Preliminary Design

Preliminary design of structural elements was done with the help of literature review and other project reports. The preliminary size of slab, beam and column were designed with reference IS 456:2000 following deflection criteria.

III. Structural Planning

Structural model of all four buildings were prepared using software (ETABS).

IV. Structural Analysis

Key parameters such as torsion, drift, deflection, displacement etc were calculated using both codes.

V. Comparison of key parameters

Different parameters of irregular buildings and the Regularbuilding were compared between both codes.

V. Discussion of Result

The discussion covered the results from comparative analysis using NBC 105:2020 and IS 1893 (Part I): 2016, followed by a comparative assessment of key results such as base shear, story drift, and torsion for irregularities.

VI. Detailed Design

The calculated and designed details of beam, column, slab and other components were drawn with the help of ETABS. The ductile detailing will be carried out as per NBC 105:2020 with reference to the books EARTHQUAKE RESITANT DESIGN OF STRUCTURES (Manish Agarwal and Pankanj Shrikhande, 2006) and REINFORCED CONCRETE LIMIT STATE OF DESIGN (Ashok K. Jain, 2016).

VII. Reporting

This process will ensure that all key elements, including methodology, analysis, and recommendations are effectively conveyed to stakeholders.

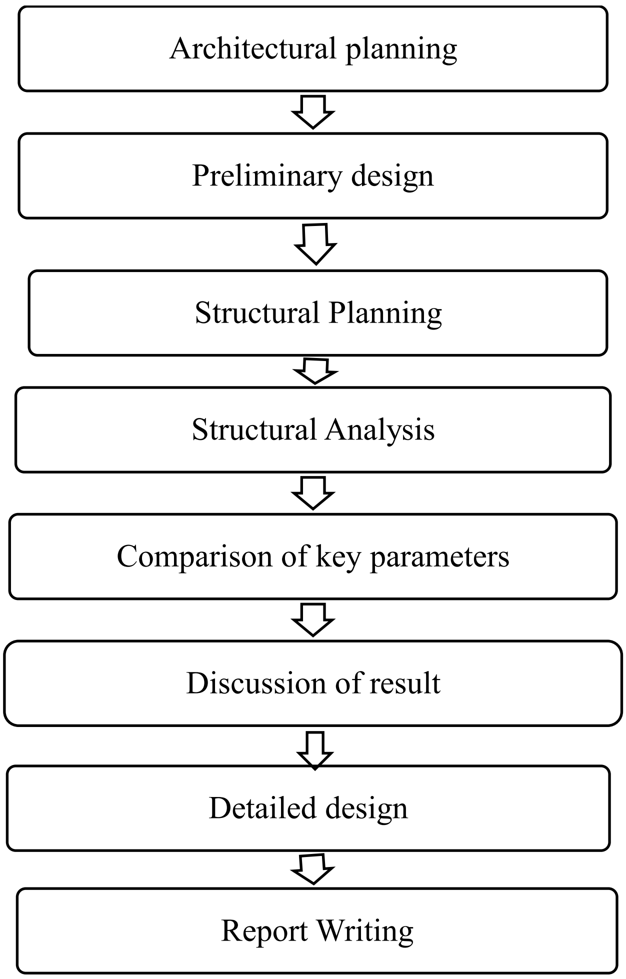

Fig 9: Flowchart for Methodology

CHAPTER 4: PRELIMINARY DESIGN

4.1 Preliminary design

Preliminary design results serve as basic dimensions to be used during the initial modeling of the building in structural analysis software. These dimensions can then be adjusted based on the structural response parameters obtained. During this analysis, the most critical location of each element is identified and its dimensions are calculated using IS 456:2000.

4.1.1 Slab

Table 5: Preliminary Design of Slab

DESCRIPTION	UNITS	VALUE	Remarks
Largest of two dimension of slab,Ly	mm	5350	Largest dimension adopted
Smallest of the two dimension of slab,Lx	mm	4250	smallest dimension adopted
Ly/Lx		1.25882353	Ratio of largest to smallest <2
Types of slab		Two-way slab	From Clauses 22.2 of Code IS 456:2000
Grade of concrete	Mpa	25	M20
Grade of steel,fy	Mpa	500	Fe500
α		26	For continous Slab
β		1	For span below 10m
Fs		290	Area of cross section of steel required /Area of cross section of steel provided=1(adopted)
ϒ		1.3	Adopting 0.35% reinforcement of tension steel and from figure 4
λ		1	
δ		1	
Minimum Effective Depth,d		125.739645	From clause 23.2.1 of code IS 456:2000
Adopted d		130.739645	Adding 5 to d
Clear Cover .CC	mm	25	Adopted more than 15 mm minimum required
Reinforcement Bar ,ø	mm	10	Adopted

Total Depth, D= d+CC+ø/2	mm	165.739645	
This depth is highly uneconomical. So, let us take total depth of slab = 125mm as of now as preliminary design.			

4.1.2 BEAM

Table 6: Preliminary Design of Beam

Description	**Unit**	**Value**	**Remarks**
Length of longest beam,L	mm	7475.00	Largest length adopted
Grade of concrete	Mpa	20.00	M20
Grade of steel,Fy	Mpa	500.00	Fe500
α		26.00	For continous beam
β		1.00	For span below 10m
Fs		290.00	Area of cross section of steel required of steel/Area of cross section of steel provided =1(adopted)
ϒ		0.75	Adopting 2% reinforcement of tension steel and figure 4
λ		1.00	
δ		1.00	
Minimum depth,d	mm	383.33	From clauses 23.2.1 of code IS 456:2000
Adopted d	mm	388.33	Adding 5 to d
Clear Cover,CC	mm	25.00	Adopted (0+10) more than 25 mm minimum required
Reinforcement Bar,φ	mm	16.00	Adopted
Total Depth,D = d+CC+φ/2	mm	421.33	
Adopted D	mm	450.00	Depth of beam
Width of beam	mm	135.00	From clauses 6.1.1 of code IS 1390:2016 (width of beam/depth of beam>0.3)
From clauses 6.1.1 of code IS 13920:2016, minimum dimension of the member should not be less than 200mm			
ThicKNess of external wall is of 230mm (brick			

work) we cannot take width less than 230mm			
Adopt beam size	mm^2	450×300	

4.1.3 COLUMN

Table 7: Preliminary Design of Column

S.No	Description	Notation	Value	Unit
	No of storey		5.00	Nos
1	**Load Calculation**			
i.	Unit wt. of Slab	γ	25.00	KN/m^3
	Effective area covered by column	Aeff	25.22	m^2
	Thickness of Slab	Ts	0.13	m
	Self wt. of Slab	Wslab	78.81	KN
	Total weight of slab in each floor	**Wslab**	**394.06**	KN
ii.	Unit wt. of Brick Masonary	γ	19.20	KN/m^3
	Length	L	10.05	m
	Floor height	h	2.70	m
	Wall Thickness	Twall	0.14	m
	assume 30% opening		0.70	
	Self wt. of Brick wall	Wwall	49.23	KN
	Total weight of brick wall in each floor		**196.93**	KN
iii.	Unit wt. of plaster	γ	20.40	KN/m^3
	Length	L	10.05	m
	Floror height	h	2.70	m
	Wall Thickness	Tplaster	0.03	m
	assume 30% opening		0.70	
	Self wt. of plaster	Wwall	9.69	KN
	Total weight of plaster		**38.75**	KN
iv.	Unit wt. of Granite	γ	26.40	KN/m^3
	Thickness of granite	Tm	0.02	m
	Area for Floor Finish	Af	25.22	m
	Unit wt. of Mortar	γ	20.40	KN/m^3
	Thickness of Mortar	Tmo	0.04	m

a.	**Floor Finish Load**	**Wf**	**156.16**	**KN**
b.	**Live Load**	**LL**	**252.20**	**KN**
	Unit wt of RCC	γ	25.00	KN/m^3
	Floor Height		3.00	m
	Assume size of column		0.20	m^2
c.	**Self wt of Column**	**Wc**	**91.13**	KN
	Total Load	Wcol	1129.23	KN
	Total factored load		**1693.85**	KN
2	**Calculation of Area**			
	Characteristics Strength of Concrete	fck	25.00	MPA
	Yield Strength of Steel	fy	500.00	MPA
	Assume Percentage of steel =2%		0.02	
	from clause 39.6 of code IS 456:2000,			
	Pu= 0.4×fck×Ac+0.67×fy×Asc	Ag	102657.59	m^2
	Area of Steel As = 2% of Ag	Asc		
	area of concrete = 98% of Ag			
	Area of Column		102657.59	mm^2
		L=B=	320.40	mm
			which is less than the assumed size ie, 450mmx450mm, hence OK.	
	Therefore, Adopt the size of column as 450mm x 450mm.			

CHAPTER 5: STRUCTURAL ANALYSIS AND COMPARISON

5.1 Material specification

Table 8: Unit weight of Material used (Source IS 456:2000)

S.N.	Material Used	Unit Weight	Type of Member
1	Cement Concrete for RCC	25 KN/m^3	Beam, Column, Slab
2	Masonry wall	19.2 KN/m^3	Infill & Partition Walls
3	Motar Screed on floor	20.4 KN/m^3	All Flooring Spaces
4	Granite	25.9-27.45(26.4) KN/m^3	Staircase and corridor
5	Tile	22 KN/m^3	All rooms floor
6	Cement plaster	20.4 KN/m^3	Wall and plastering.

- Thickness of Granite Used = 16 mm
- Thickness of Tile Used = 12 mm
- Thickness of Mortar Screeding = 40 mm
- Thickness of Plastering = 12.5 mm and 10 mm in external and internal wall in each side respectively.
- Thickness of External wall = 230 mm
- Thickness of Internal wall = 115 mm

5.2 Software Verification

5.2.1 Calculation of Seismic Weight

Table 10: Beam Load Calculation

BEAM LOAD CALCULATION		
S.N.	**Floor**	**Load on each floor (KN)**
1	staircase cover	250
2	terrace roof	511.875
3	3rd Floor	511.875
4	2nd Floor	642.625
5	1st floor	642.625
6	Ground floor	642.625
OVERALL WEIGHT FROM BEAM		3201.625

Table 9: External Wall Load Calculation

EXTERNAL WALL LOAD CALCULATION		
S.N.	**Floor**	**Load on each floor (KN)**
1	staircase cover	158.49
2	terrace roof	332.564
3	3rd Floor	665.128
4	2nd Floor	665.128
5	1st floor	665.128
6	Ground floor	665.128
OVERALL WEIGHT FROM WALL		3151.566

Table11: Internal Wall Load Calculation

INTERNAL WALL LOAD CALCULATION		
S.N.	**Floor**	**Load on each floor (KN)**
1	staircase cover	0
2	terrace roof	232.47
3	3rd Floor	464.94
4	2nd Floor	464.94
5	1st floor	464.94
6	Ground floor	464.94
OVERALL WEIGHT FROM WALL		2092.23

Table 12: Column Load Calculation

COLUMN LOAD CALCULATION		
S.N.	**Floor**	**Load on each floor (KN)**
1	staircase cover	75.625
2	terrace roof	240.75
3	3rd Floor	481.5
4	2nd Floor	481.5
5	1st floor	481.5
6	Ground floor	481.5
OVERALL WEIGHT FROM COLUMN		2242.375

Table 13: Slab Load Calculation

SLAB LOAD CALCULATION		
S.N.	**Floor**	**Load on each floor (KN)**
1	staircase cover	93.75
2	terrace roof	1981.317
3	3rd Floor	1981.317
4	2nd Floor	1981.317
5	1st floor	1981.317
6	Ground floor	1981.317
OVERALL WEIGHT FROM SLAB		10000.335

Table 14: Staircase Load Calculation

STAIRCASE LOAD		
S.N.	**Floor**	**Load on each floor (KN)**
1	staircase cover	35.15625
2	terrace roof	70.3125
3	3rd Floor	70.3125
4	2nd Floor	70.3125
5	1st floor	70.3125
6	Ground floor	70.3125
OVERALL WEIGHT FROM STAIRCASE		386.71875

Table 15: Others Load

WATER TANK LOAD
103.005
PARAPET WALL LOAD
475.338
FLOOR FINISH IN STAIR
0

Table 16: Live Load Calculation for NBC

S.N.	Floor	Load on each floor (KN) (taking 30%)
1	staircase cover	0
2	terrace roof	174.96
3	3rd Floor	254.88
4	2nd Floor	254.88
5	1st floor	254.88
6	Ground floor	254.88
OVERALL WEIGHT		1194.48

Table 17: Live Load Calculation for IS

S.N.	Floor	Load on each floor(KN) (taking 25%)
1	staircase cover	0
2	terrace roof	145.75
3	3rd Floor	212.4
4	2nd Floor	212.4
5	1st floor	212.4
6	Ground floor	212.4
OVERALL WEIGHT		995.35

Total seismic weight from NBC=Total dead load+ 30% of Live Load

= 25368.472KN

Total seismic weight from IS= Total dead load+ 25% of Live Load

= 25075.592KN

5.2.2 Base Shear calculation

Table 18: Calculation of Base Shear Coefficient and Base Shear as Per NBC

CALCULATION OF HORIZONTAL BASE SHEAR COEFFICIENT				
Notation and Calculation	**Value**	**Unit**	**Description**	**Info.**
k_t	0.075		for Moment resisting concrete frame	From clause 5.1.2 of NBC 105:2020
H	18.000	m	Height of the building from foundation or from top of a rigid basement	All Clauses and Tables From NBC 105:2020
$T_1 = k_t H^{\frac{3}{4}}$	0.655	sec	Approximate fundamental period of vibration	from clause 5.1.2
T	0.819	sec	T_1 is increased by 1.25 times	from clause 5.1.3
Considering Soil Type D				
T_a	0.000		lower periods of the flat part of the spectrum	from Table 4-1
T_c	2.000		upper periods of the flat part of the spectrum	from Table 4-1
α	2.250		peak spectral acceleration normalized by PGA	from Table 4-1
k	0.800		Controls the descending branch of the spectrum	from Table 4-1
C_hT	2.250			
Z	0.300		Zoning factor (POKHARA)	from clause 4.1.4
I	1.000		Importance Factor	from clause 4.1.5
$C(T) = C_h(T)\, Z\, I$	0.675			from cluase 4.1.1
$C(T_1)$	0.675			
R_μ	4.000		Ductility Factor For ULS	from table 5-2

Ωu	1.500		Over Strength Factor For ULS	from table 5-2
Ωs	1.250		Over Strength Factor For SLS	from table 5-2
FROM ULS				
$C_d(T_1) = \frac{C(T_1)}{R\mu\, \Omega u}$	0.113		Horizontal Base Shear Coefficient	from clause 6.1.1
FROM SLS				
$Cs(T_1)$	0.135			
$C_d\ (T_1) = \frac{C_S\ (T_1)}{\Omega s}$	0.108		Horizontal Base Shear Coefficient	from clause 6.1.1
W	25368.472	KN	Seismic Weight	
V_b(ULS)	2853.953	KN	Horizontal Base Shear for ULS	From Clause 6.2
V_b(SLS)	2739.795	KN	Horizontal Base Shear for SLS	From Clause 6.2

Table 19: Calculation of Base Shear Coefficient and Base Shear as Per IS

Calculation of Horizontal Base Shear Coefficient			
Notation and Calculation	**Values**	**Units**	**Information**
Height of Building (h)	18.000	meter	Ht of Building from ground to top
Approximate Natural Period (T_a)	0.655	Second	Cl 7.6.2(a) for RC MRF Building
Seismic Zone Factor (Z)	0.36		Cl 6.4.2 Table 3
Design Accleration coefficient (S_a/g)	2.5		Cl 6.4.2 (a) For Soft soil Sites
Response Reduction Factor (R)	5		Cl 7.2.6 Table 9
Importance Factor (I)	1.0		Cl 7.2.3 Table 8

Design Horizontal Base Shear Coefficient $A_h = \frac{\left(\frac{Z}{2}\right)\left(\frac{S_A}{g}\right)}{\left(\frac{R}{I}\right)}$	0.090		Cl 6.4.2
Base Shear (Vb)	2256.803	KN	Cl 7.5.3

Lateral Load Distribution as per NBC 105:2020 (ULS)

Table 20: Lateral Load Distribution

Concenteration of mass at each floor, Determination of design lateral loads at each floor: ULS						
S. N.	**Floor level**	**Total load (KN)**	**hi(m)**	**Wihi$^{1.159}$**	**(Wihi$^{1.159}$)/(∑Wihi$^{1.159}$)**	**Q_i**
1	staircase cover	706.77125	18	20143.7215	0.060172797	172.494
2	terrace roof	4577.3415	15	105609.5237	0.315474001	904.35
3	3rd Floor	4884.7025	12	87018.01707	0.259937939	745.148
4	2nd Floor	5015.4525	9	64014.3279	0.191221922	548.164
5	1st floor	5019.7025	6	40045.64361	0.119623297	342.917
6	Ground floor	5019.7025	3	17933.35365	0.053570044	153.566
SUM		25368.47275		334764.5875	1.00	2866.64

5.2.3 Base Shear Comparison (Manual vs Etabs)

Table 21: Base Shear Comparison for Regular Building (Manual vs Etabs)

Codes	Manual	Etabs
NBC ULS	2853.953	2842.287
NBC SLS	2739.795	2716.5223
IS	2256.803	2245.1472

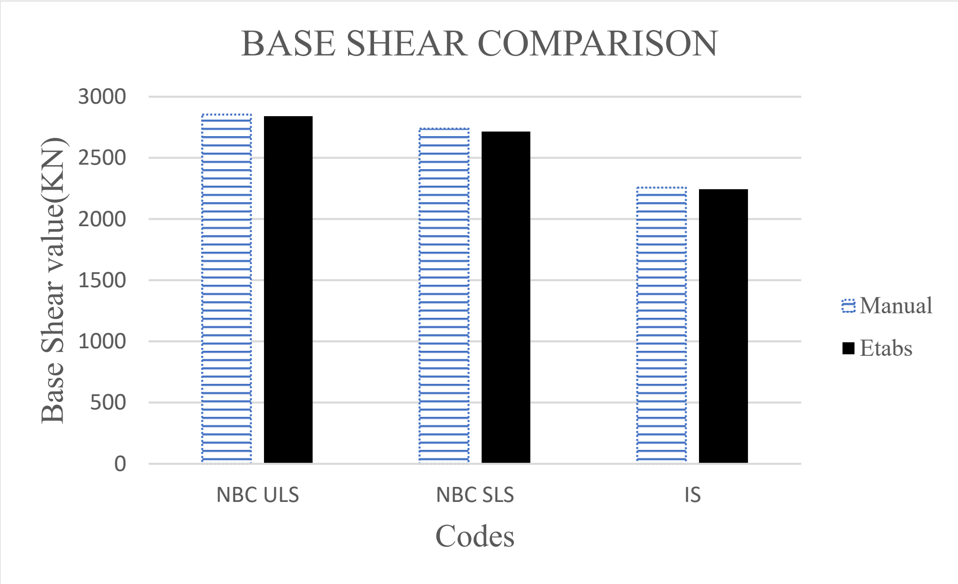

Fig 10: Base Shear Comparison Graph for Regular Building (Manual vs Etabs)

The above chart shows that the manual calculations for base shear and base shear from software (ETABS) are approximately equal.

5.2.4 Centre of Mass Calculation

Manual Calculation

Table 22: Centre of Mass of Each Floors from Manual Calculation

FIRST FLOOR								
S. N	**Structural Components**	**W_i (KN)**	**Xi (m)**	**Yi (m)**	**$W_i \times X_i$**	**$W_i \times Y_i$**	**X (m)**	**Y (m)**
1	Slab	2145.94	13.12	7.316	28165.49	15699.7		
2	Beam	664.713	13.52	7.108	8990.901	4724.77		
3	Column	382.863	13.56	7.286	5193.530	2789.53		
4	Ext Wall	963.144	13.50	7.200	13002.44	6934.63		
5	Int Wall	578.629	13.12	6.885	7594.506	3983.86		
	TOTAL	**4735.29**			**62946.87**	**34132.5**	**13.2**	**7.208**

SECOND FLOOR								
S. N	**Structural Components**	**W_i (KN)**	**Xi (m)**	**Yi (m)**	**Wi× X_i**	**W_i× Y_i**	**X (m)**	**Y (m)**
1	Slab	2145.94	13.12	7.316	28165.49	15699.71		
2	Beam	664.713	13.52	7.108	8990.901	4724.776		
3	Column	383.553	13.56	7.287	5202.896	2794.951		
4	Ext Wall	963.144	13.50	7.200	13002.44	6934.637		
5	Int Wall	578.629	13.12	6.885	7594.506	3983.861		
	TOTAL	**4735.98**			**62956.24**	**34137.94**	**13.29**	**7.20**

THIRD FLOOR								
S. N	**Structural Components**	**W_i (KN)**	**Xi (m)**	**Yi (m)**	**Wi× X_i**	**W_i× Y_i**	**X (m)**	**Y (m)**
1	Slab	2145.9	13.12	7.316	28165.4	15699.71		
2	Beam	636.43	13.52	7.115	8609.09	4528.253		
3	Column	385.87	13.56	7.293	5233.96	2814.164		
4	Ext Wall	963.14	13.50	7.200	13002.4	6934.637		
5	Int Wall	578.62	13.12	6.885	7594.50	3983.861		
	TOTAL	**4710.0**			**62605.5**	**33960.6**	**13.29**	**7.21**

FOURTH FLOOR								
S. N	**Structural Components**	**W_i (KN)**	**Xi (m)**	**Yi (m)**	**Wi× X_i**	**W_i× Y_i**	**X (m)**	**Y (m)**
1	Slab	2145.94	13.12	7.316	28165.4	15699.715		
2	Beam	485.563	13.52	7.121	6569.17	3457.691		
3	Column	388.125	13.56	7.286	5264.52	2827.879		
4	Ext Wall	963.144	13.50	7.200	13002.4	6934.637		
5	Int Wall	578.62	13.12	6.885	7594.50	3983.861		
	TOTAL	**4561.4**			**60596.1**	**32903.78**	**13.28**	**7.21**

FIFTH FLOOR								
S. N	**Structural Components**	**W_i (KN)**	**Xi (m)**	**Yi (m)**	**$Wi \times X_i$**	**$W_i \times Y_i$**	**X (m)**	**Y (m)**
1	Slab	1528.91	13.27	7.160	20290.24	10947.0		
2	Beam	585.563	13.52	7.121	7922.075	4169.79		
3	Column	231.875	13.55	8.039	3142.834	1864.04		
4	Ext Wall	132.300	13.50	11.90	1786.050	1574.37		
5	Parapet Wall	465.055	13.50	6.734	6278.243	3131.52		
	TOTAL	**2943.7**			**39419.44**	**21686.7**	**13.39**	**7.36**

SIXTH FLOOR								
S.No.	**Structural Components**	**W_i (KN)**	**Xi (m)**	**Yi (m)**	**$Wi \times X_i$**	**$W_i \times Y_i$**	**X (m)**	**Y (m)**
1	Slab	98.525	13.5	11.90	1330.08	1172.44		
2	Beam	56.250	13.5	11.90	759.375	669.375		
3	Column	37.813	13.5	11.90	510.469	449.969		
	TOTAL	**192.58**			**2599.93**	**2291.79**	**13.5**	**11.90**

Table 23: Centre of Mass of Building from Manual Calculation

Centre of Mass of Building								
S. N	**FLOOR**	**W_i (KN)**	**Xi (m)**	**Yi (m)**	**$Wi \times X_i$**	**$W_i \times Y_i$**	**X (m)**	**Y (m)**
1	6	192.588	13.50	11.900	2599.931	2291.791		
2	5	2943.70	13.39	7.367	39419.444	21686.766		
3	4	4561.40	13.28	7.214	60596.148	32903.782		
4	3	4710.02	13.29	7.210	62605.501	33960.629		
5	2	4735.98	13.29	7.208	62956.243	34137.940		
6	1	4735.29	13.29	7.208	62946.876	34132.525		
	TOTAL	**21878.9**			**291124.14**	**159113.43**	**13.30**	**7.27**

Table 24: Centre of Mass of Building from ETABS

CENTRE OF MASS FROM ETABS									
Story	**Diaphragm**	**Mass X**	**Mass Y**	**XCM**	**YCM**	**Cum Mass X**	**Cum Mass Y**	**XCCM**	**YCCM**
		kg	kg	m	m	kg	kg	m	m
Story6	D1	12129.67	12129.67	13.75	12.15	12129.67	12129.67	13.75	12.15
Story5	D1	197384.64	197384.64	14.9477	8.6747	209514.31	209514.31	14.8783	8.8759
Story4	D1	323720.58	323720.58	14.8771	7.9687	533234.89	533234.89	14.8776	8.3252
Story3	D1	320771.55	320771.55	14.8764	7.991	854006.43	854006.43	14.8771	8.1997
Story2	D1	328954.58	328954.58	14.8781	7.9868	1182961.02	1182961.02	14.8774	8.1405
Story1	D1	328954.58	328954.58	14.8781	7.9868	1511915.6	1511915.6	14.8775	8.1071

5.2.5 Centre of Rigidity

Table 25: Centre of Rigidity from Manual calculation

Centre of Rigidity for Column						
SN	**Kx**	**Ky**	**Ky×X**	**Kx×Y**	**Xcr**	**Ycr**
Ground floor	1496319.44	1496319.44	20292905.09	11174256.94	13.56	7.46
First Floor	1496319.44	1496319.44	20292905.09	11174256.94	13.56	7.46
Second Floor	1496319.44	1496319.44	20292905.09	11174256.94	13.56	7.46
Third floor	1496319.44	1496319.44	20292905.09	11174256.94	13.56	7.46
Fourth Floor	1496319.44	1496319.44	20292905.09	11174256.94	13.56	7.46
Staircase Cover	338912.037	338912.037	4405856.481	4033053.241	13	11.9

Table 26: Centre of Rigidity from ETABS

Centre of Rigidity from Etabs									
Story	Diaphragm	Mass X	Mass Y	XCM	YCM	Cum Mass X	Cum Mass Y	XCR	YCR
		kg	kg	m	m	kg	kg	m	m
Story6	D1	12129.67	12129.67	13.75	12.15	12129.67	12129.67	13.5763	11.1882
Story5	D1	197384.64	197384.64	14.9477	8.6747	209514.31	209514.31	13.5701	8.3307
Story4	D1	323720.58	323720.58	14.8771	7.9687	533234.89	533234.89	13.5631	8.283
Story3	D1	320771.55	320771.55	14.8764	7.991	854006.43	854006.43	13.5689	8.2041
Story2	D1	328954.58	328954.58	14.8781	7.9868	1182961.02	1182961.02	13.5891	8.0582
Story1	D1	328954.58	328954.58	14.8781	7.9868	1511915.6	1511915.6	13.6645	7.7704

Result: From above the table it is seemed that centre of rigidity obtained from manual calculation is nearly equal to the centre of rigidity obtained from Etabs.

5.2.5 Load Calculation on Beam

Table 27: load Calculation on Beam Using Yield Line Theory

Calculation of Load on Beam Using Yield Line Theory				
S.No.	**Notation and Calculation**	**Value**	**Unit**	**Description**
	Effected Area (A_1)	6.75	m^2	from Autocad
	Effected Area (A_2)	6.6975	m^2	from Autocad
	Total Effected Area (A_3) = A_1+A_2	13.4475	m^2	
1	SLAB			
	Unit Weight	25	KN/m^3	
	Thickness	0.125	m	
	Area	13.4475	m^2	
	Weight of Slab	42.02344	KN	
2	FLOOR FINISH			

	FF 1	1.08	KN/m^2	
	FF2	1.08	KN/m^2	
	Weight of Floor Finish	14.5233	KN	
3	BEAM			
	Size of beam	350×550	mm×mm	
	Unit Weight	25	KN/m^3	
	Width	0.35	m	
	Depth	0.55	m	
	Length	5.2	m	
	Self-Weight of Beam	19.3375	KN	
4	LIVE LOAD			
	LL 1	2	KN/m^2	
	LL2	2	KN/m^2	
	Weight of live load	26.895	KN	
	Total Load on Beam=1.5DL+1.2LL	146.1004	KN	
		28.09622	KN/m	

5.2.6 BC Ratio check

Table 28: BC ratio check from Manual Calculation

For Column 1E				
SN	**Notation**	**Value**	**Unit**	Remarks
1	**Moment calculation for column**			
	Fck=	25	N/mm^2	
	Fy=	500	N/mm^2	
	b=	500	mm	
	d=	500	mm	
	d'=	60	mm	
	Upper column			
	Pu=	604.274	kn	From Etabs
	Pt=	0.86	%	From Etabs
	d'/d	0.15		
	Pu/fckbd=	0.09668384		Chart 37 From IS 456 2000

	Pt/fck=	0.0344		Chart 37 From IS 456 2000
	Mu/fckbd2	0.08		Chart 37 From IS 456 2000
	Mu,u=	250000000	Nmm	Chart 37 From IS 456 2000
		250	Kn-m	
	Lower column			
	Pu=	1896.0072	Kn	From Etabs
	Pt=	2.25	%	From Etabs
	d'/d	0.15		
	Pu/fckbd=	0.30336115		Chart 37 From IS 456 2000
	Pt/fck=	0.09		
	Mu/fckbd2	0.15		
	Mu,l=	468750000	Kn-m	
		468.75	Kn-m	
	∑Mc=Mu,u+Mu,l	718.75	Kn-m	
2	**Moment calculation for beam**			
	Left beam Sagging			
	fck	25	N/mm^2	
	fy	500	N/mm^2	
	bw	300	mm	
	D	500	mm	
	d'	60	mm	
	d	440	mm	
	Ast top	1036	mm^2	From Etabs
	Ast bottom	518	mm^2	From Etabs
	Xu=(0.87× fy× Ast)/0.36fckbf	83.4555556	mm	
	Mu=(0.87fyAstd)× (1-(Astfy/bfdfck))	91247083.1	N-mm	
		91.25	Kn-m	
	Right beam hogging			
	fck	25	N/mm2	

	fy	500	N/mm2	
	b	300	mm	
	D	500	mm	
	d'	60	mm	
	d	440	mm	
	Ast top	1136	mm2	
	Ast bottom	568	mm2	
	Xu max= 0.46d	202.4	mm	
	Mu lim= 0.133× fckbd2	193.116	KNm	
	Area of Balanced Section	422.405525		
	Mu	193116000	N-mm	
		193.12	Kn-m	
	∑Mb=	284.37	Kn-m	
3	**Check for BC ratio:**			
	∑Mc/∑Mb=	2.528		> 1.2 OK!!
The concept of Strong Column Weak Beam is established as per clause 4.4.4 of NBC 105:2020				

5.3 MODELLING

5.3.1 Apartment building

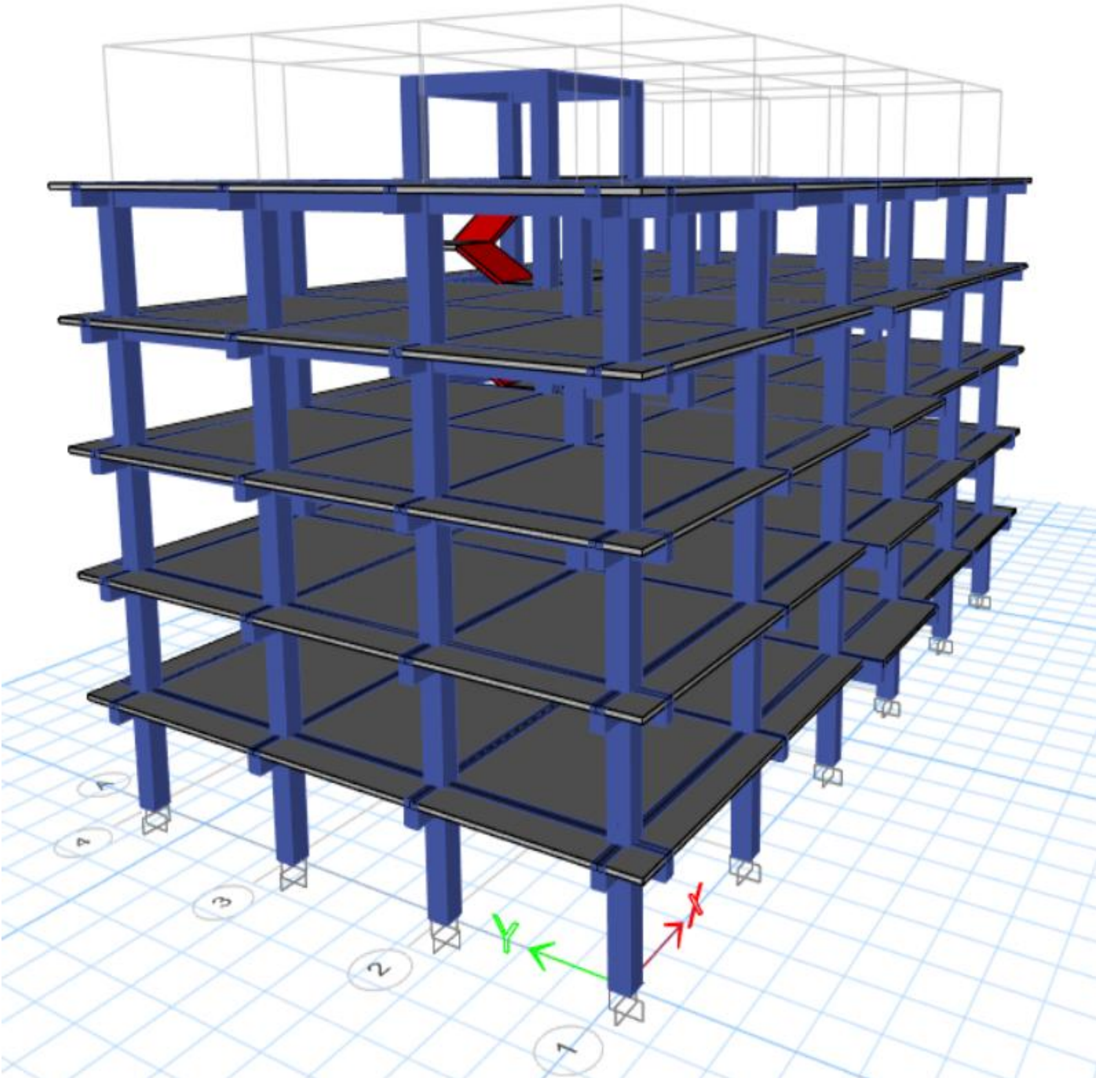

Fig 11: Extruded view of apartment building (Source: ETABS)

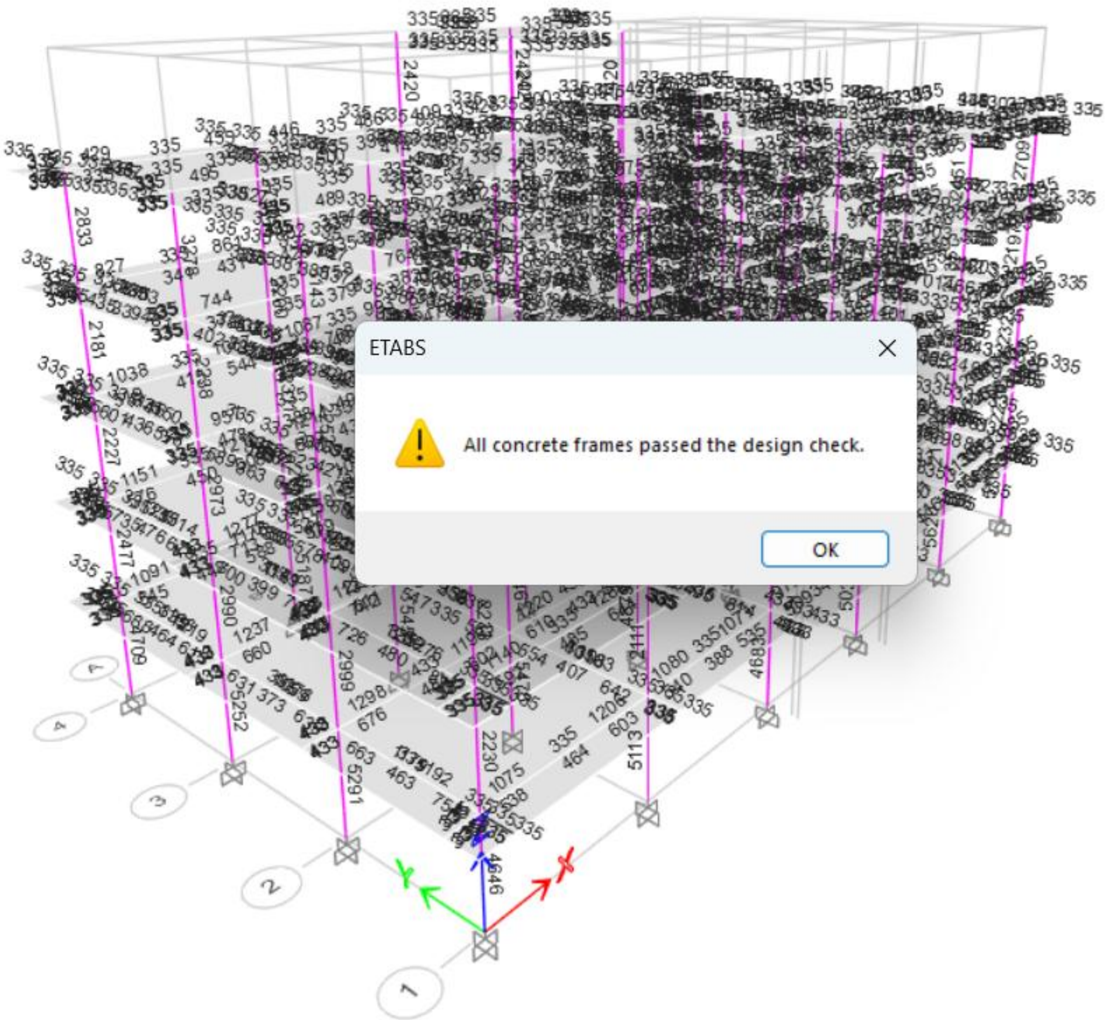

Fig 12: Design Check (Source: ETABS)

Fig 13: Reinforcement Detailing (Source: ETABS)

5.3.2 C shaped building

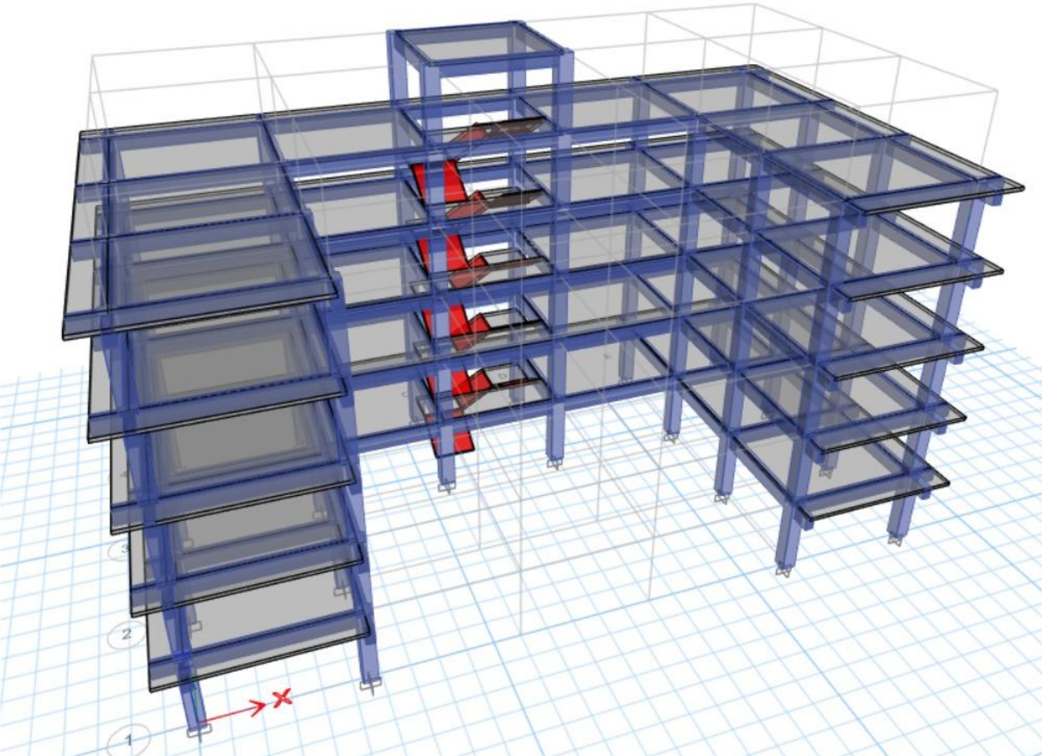

Fig 14: Extruded view of C shaped building (Source: ETABS)

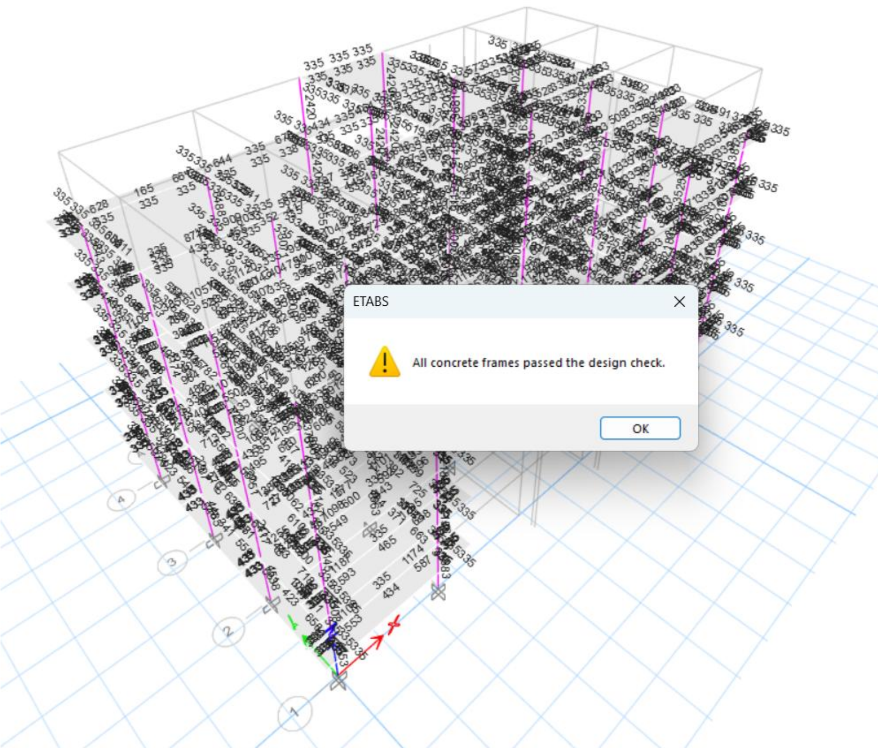

Fig 15: Design Check (Source: ETABS)

5.2.3 E shaped building

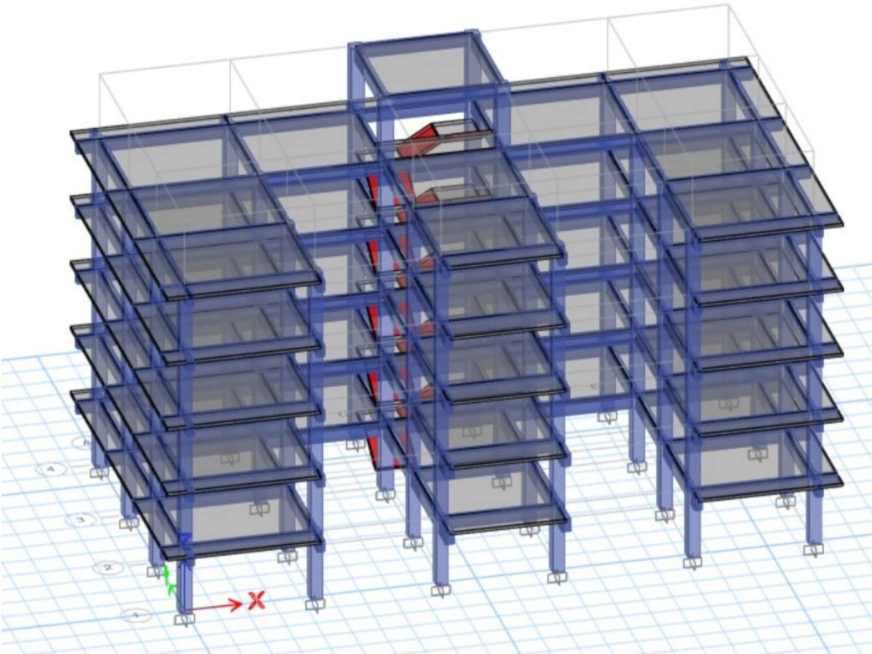

Fig 16: Extruded view of E shaped building (Source: ETABS)

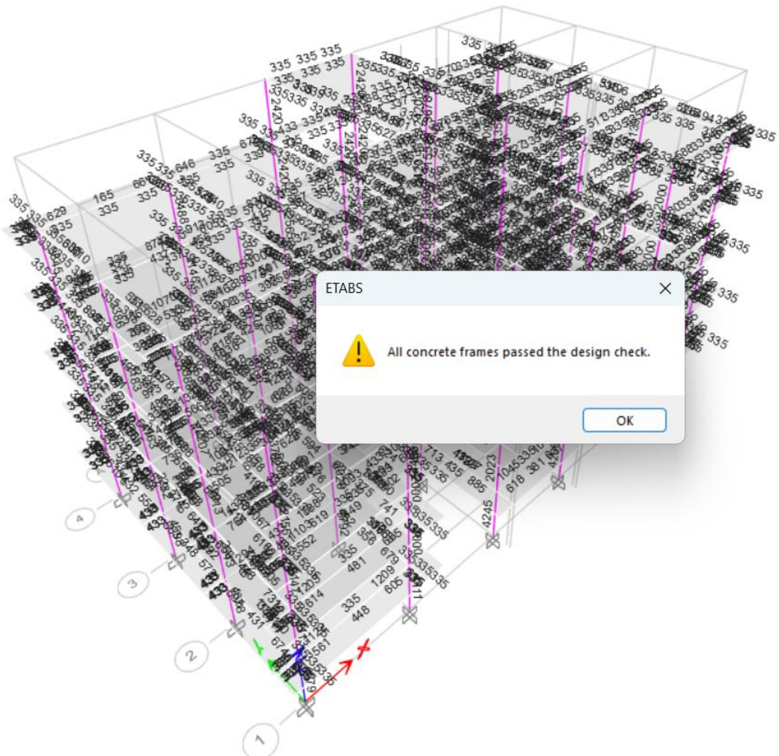

Fig 17: Design Check (Source: ETABS)

5.2.4 L Shaped Building

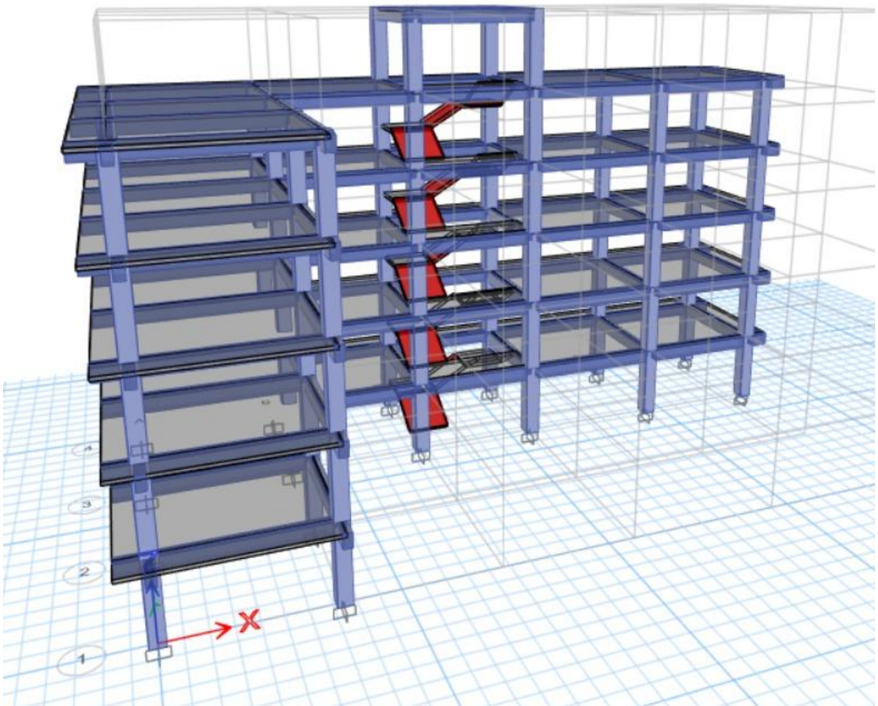

Fig :18 Extruded view of L shaped building (Source: ETABS)

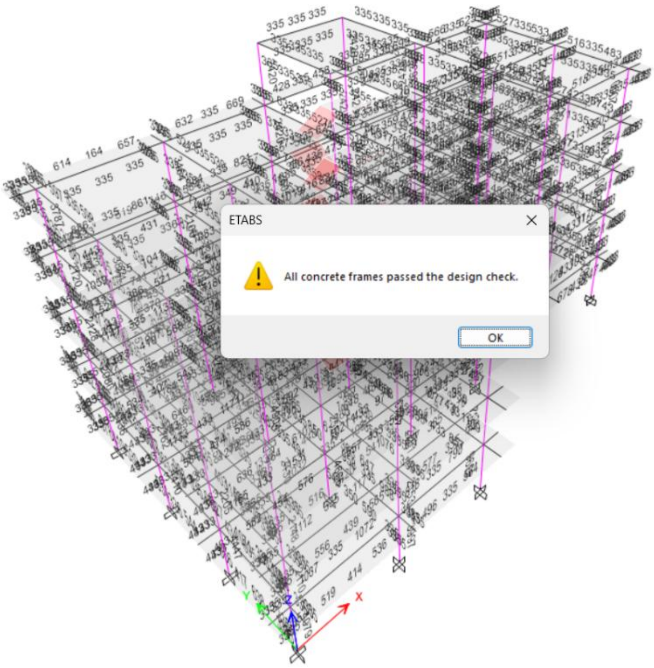

Fig 19: Design check (Source: ETABS)

5.4 Comparison of Base Shear

Table 29: Base Shear for Regular Building

Code	Base Shear Value (KN)
IS	2245.1472
NBC ULS	2842.287
NBC SLS	2716.5223

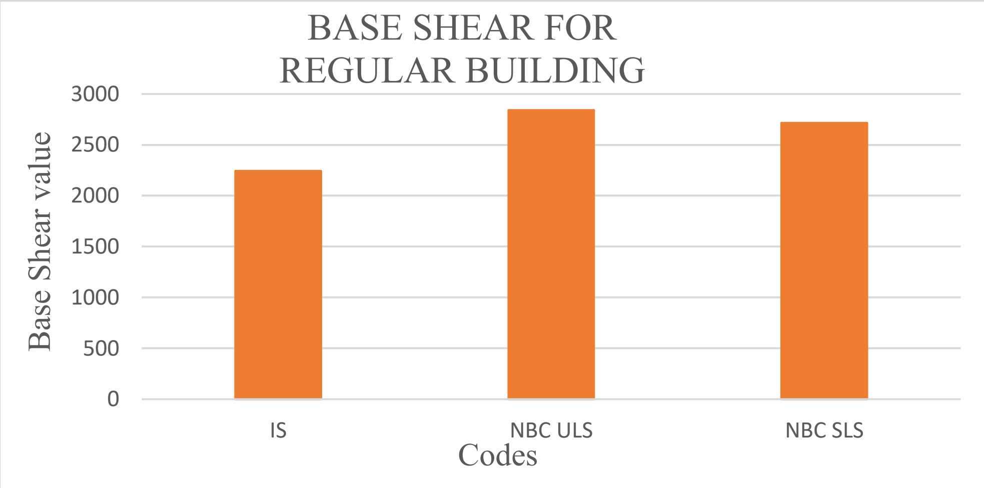

Fig 20: Base Shear Comparison Graph for Regular Building

Table 30: Base Shear for L Shaped Building

Code	Base Shear Value (KN)
IS	1361.9736
NBC ULS	1633.2885
NBC SLS	1561.091

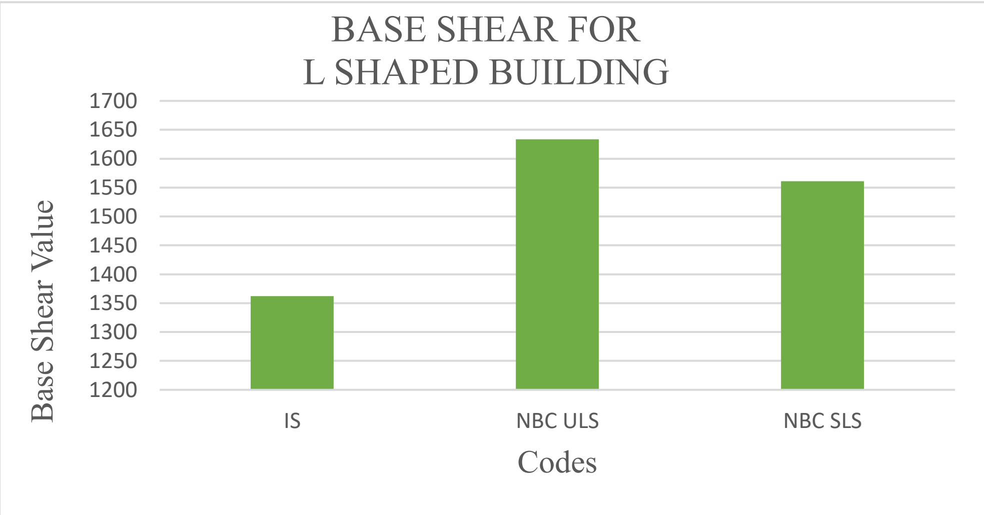

Fig 21: Base Shear Comparison Graph for L Shaped Building

Table 31: Base Shear for C Shaped Building

Code	Base Shear Value (KN)
IS	1712.8258
NBC ULS	2137.3408
NBC SLS	2042.7682

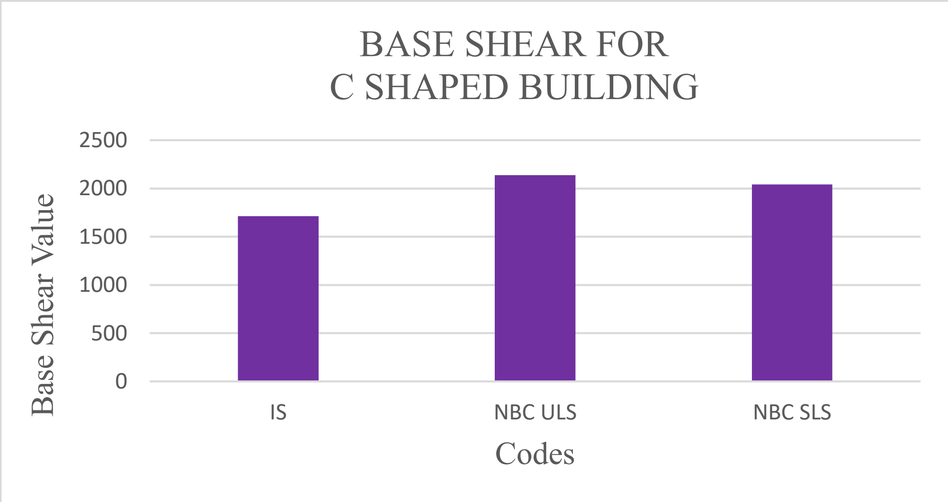

Fig 22: Base Shear Comparison Graph for C Shaped Building

Table 32: Base Shear for E Shaped Building

Code	Base Shear Value (KN)
IS	1927.0275
NBC ULS	2379.6223
NBC SLS	2274.3293

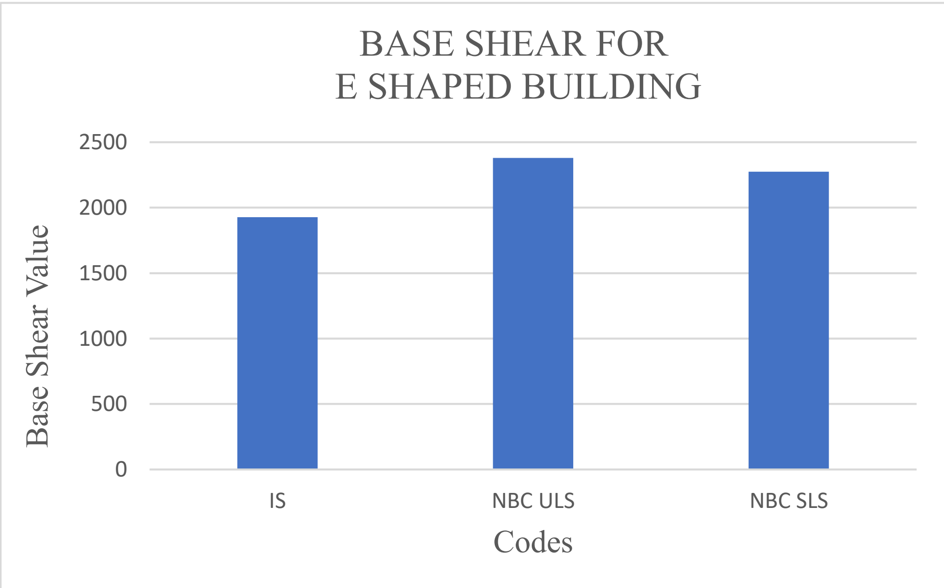

Fig 23: Base Shear Comparison Graph for E Shaped Building

Result: From above graphs, it is evident that the base shear building is higher in NBC than in IS with significant percentage.

Building and Base shear	Remarks
Regular	NBC higher by 26.5%
L Shaped	NBC higher by 19.92%
C Shaped	NBC higher by 24.78%
E Shaped	NBC higher by 23.48%

5.5 Displacement comparison

Table 33: Displacement Comparison for regular building in X direction

X Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	41.338	39.509	37.158
5	38.127	36.44	33.71
4	33.605	32.118	29.166
3	26.165	25.007	22.149
2	16.707	15.968	13.782
1	6.688	6.392	5.403
Base	0	0	0

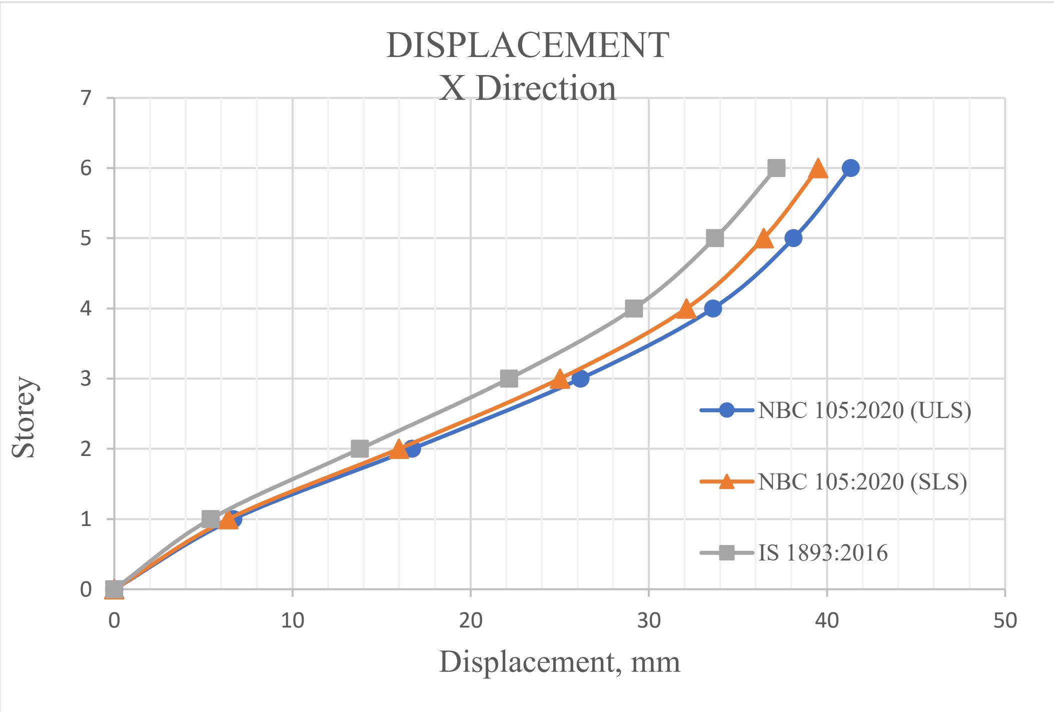

Fig 24: Displacement Comparison Chart for regular building in X-Direction (NBC/IS)

Table 34: Displacement Comparison for regular building in Y direction

Y Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	44.414	42.449	39.686
5	41.047	39.231	36.114
4	36.183	34.582	31.272
3	28.122	26.878	23.72
2	17.787	17	14.622
1	6.922	6.615	5.569
Base	0	0	0

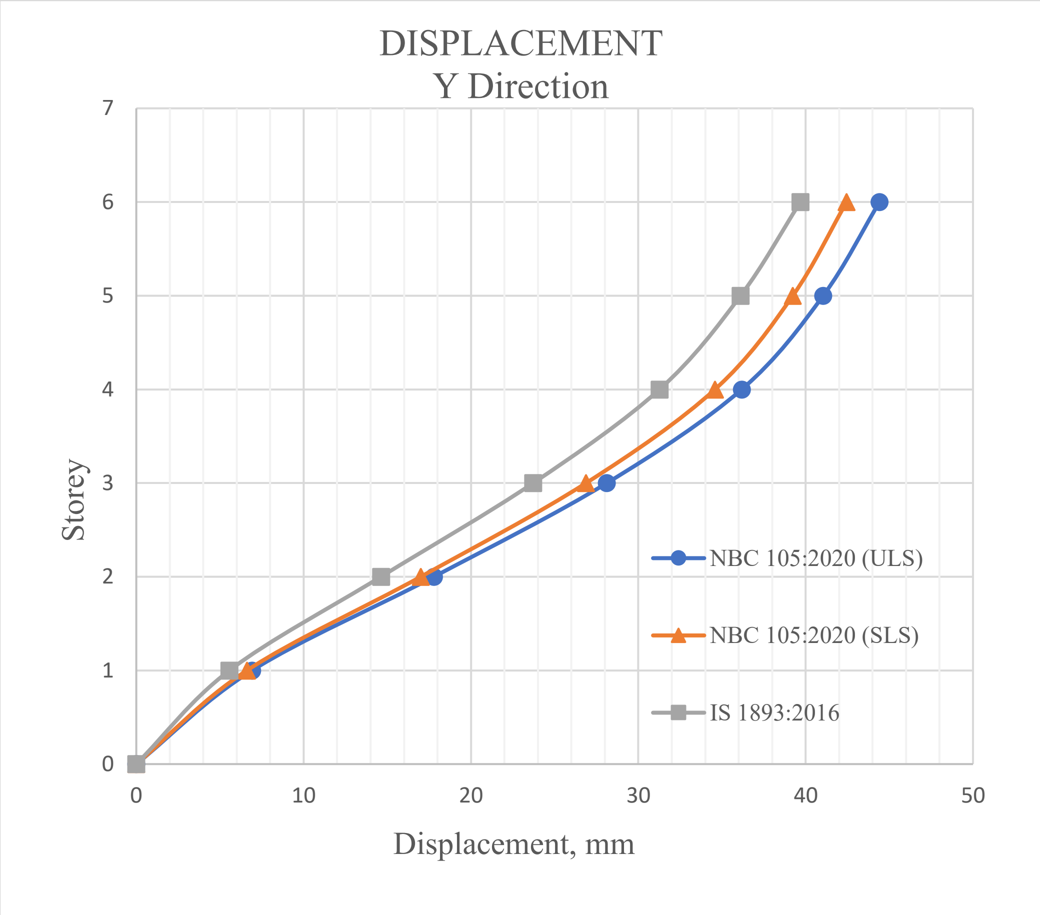

Fig 25: Displacement Comparison Chart for regular building in Y-Direction (NBC/IS)

Table 35: Displacement Comparison for L Shaped Building in X direction

X Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	36.598	34.979	34.889
5	33.52	32.037	31.379
4	29.349	28.05	26.652
3	22.748	21.741	20.037
2	14.471	13.831	12.367
1	5.769	5.513	4.822
Base	0	0	0

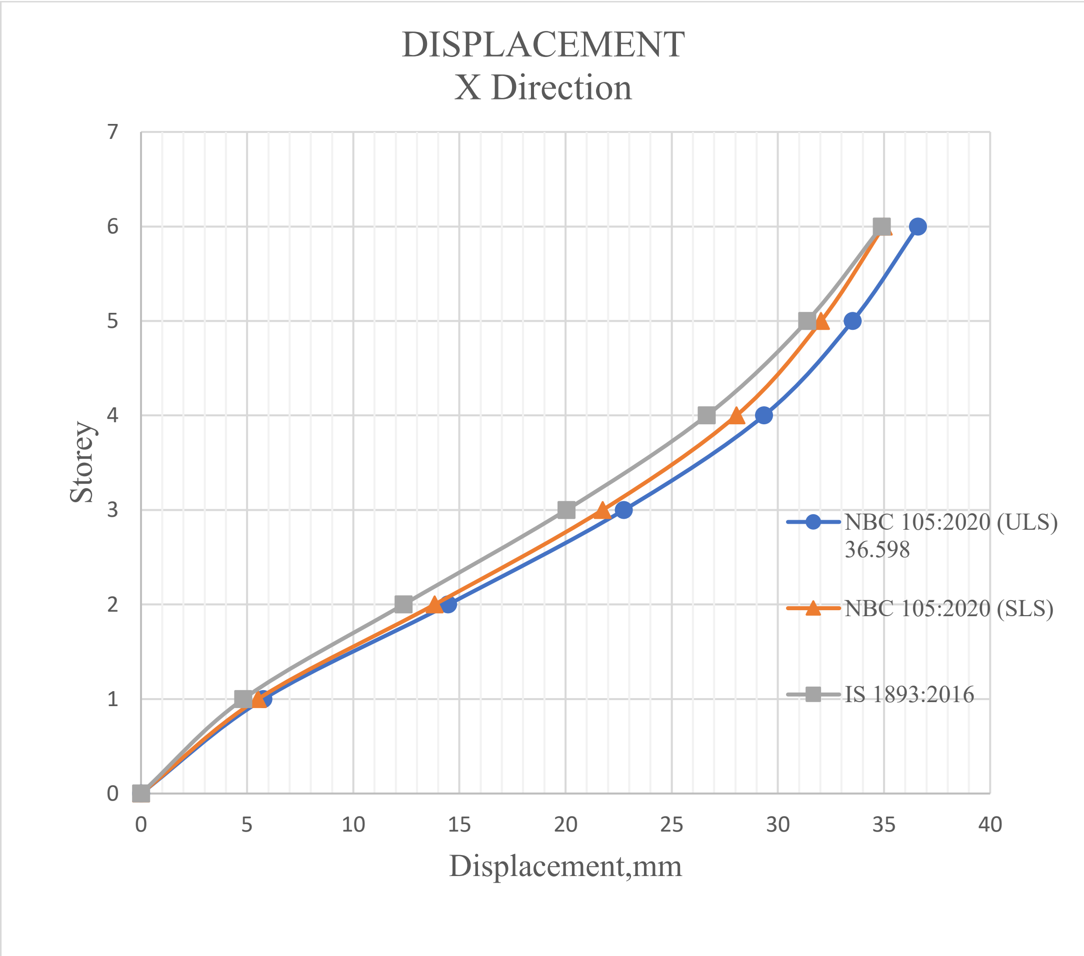

Fig 26: Displacement Comparison Chart for L Shaped Building in X direction (NBC/IS)

Table 36: Displacement Comparison for L Shaped Building in Y direction

Y Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	42.216	40.348	39.573
5	38.462	36.76	35.368
4	33.371	31.894	29.79
3	25.648	24.513	22.233
2	16.032	15.323	13.493
1	6.115	5.845	5.037
Base	0	0	0

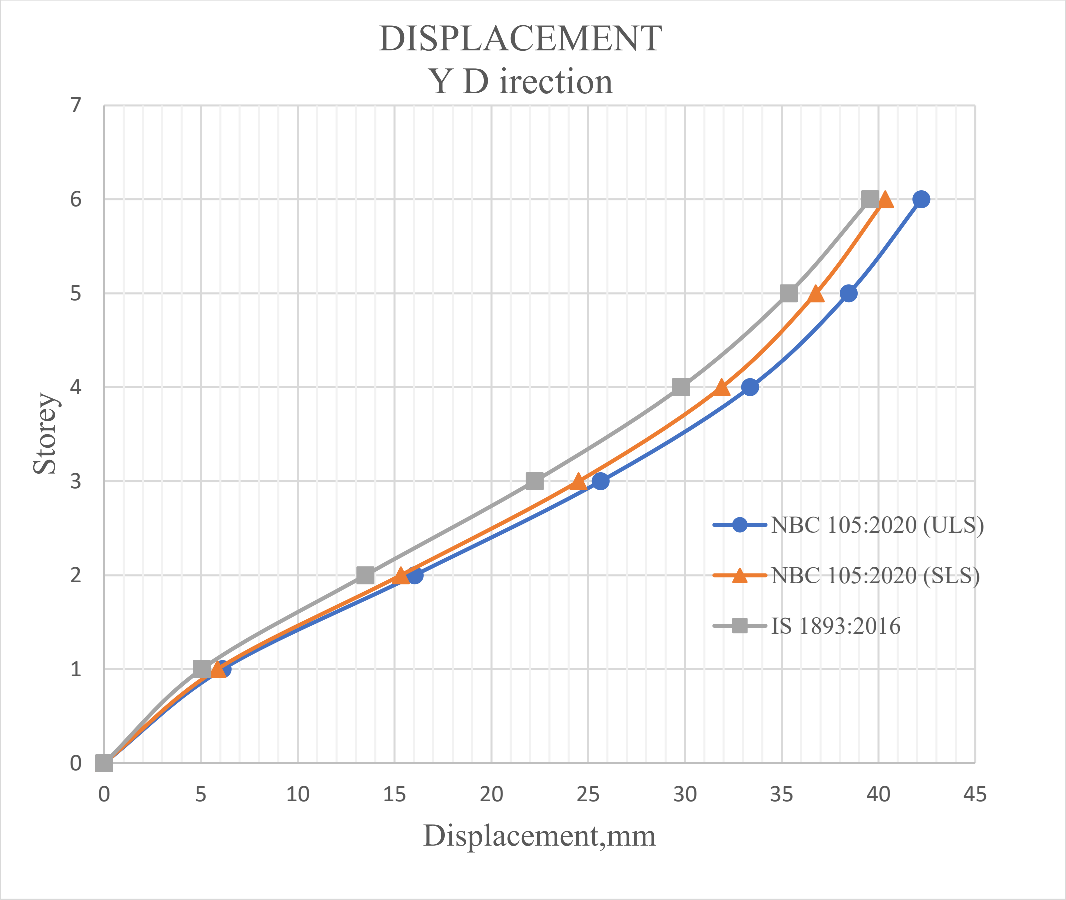

Fig 27: Displacement Comparison Chart for L Shaped Building in Y direction (NBC/IS)

Table 37: Displacement Comparison for C Shaped Building in X direction

X Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	42.217	40.349	37.856
5	38.653	36.943	34.098
4	33.364	31.888	28.881
3	25.468	24.341	21.622
2	15.905	15.201	13.28
1	6.296	6.018	5.0131
Base	0	0	0

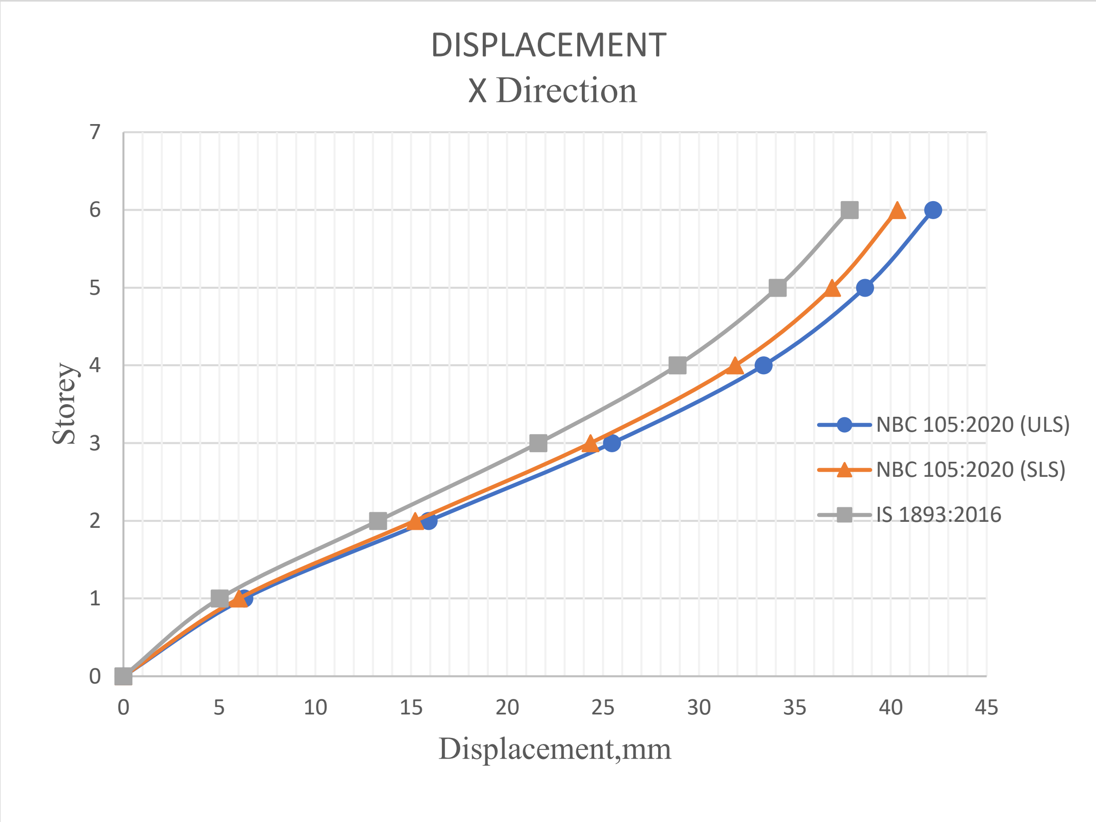

Fig 28: Displacement Comparison Chart for C Shaped Building in X direction (NBC/IS)

Table 38: Displacement Comparison for C Shaped Building in Y direction

Y Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	44.433	42.467	40.168
5	40.435	38.646	36.037
4	34.896	33.352	30.598
3	26.701	25.52	23.012
2	16.664	15.926	14.06
1	6.417	6.133	5.272
Base	0	0	0

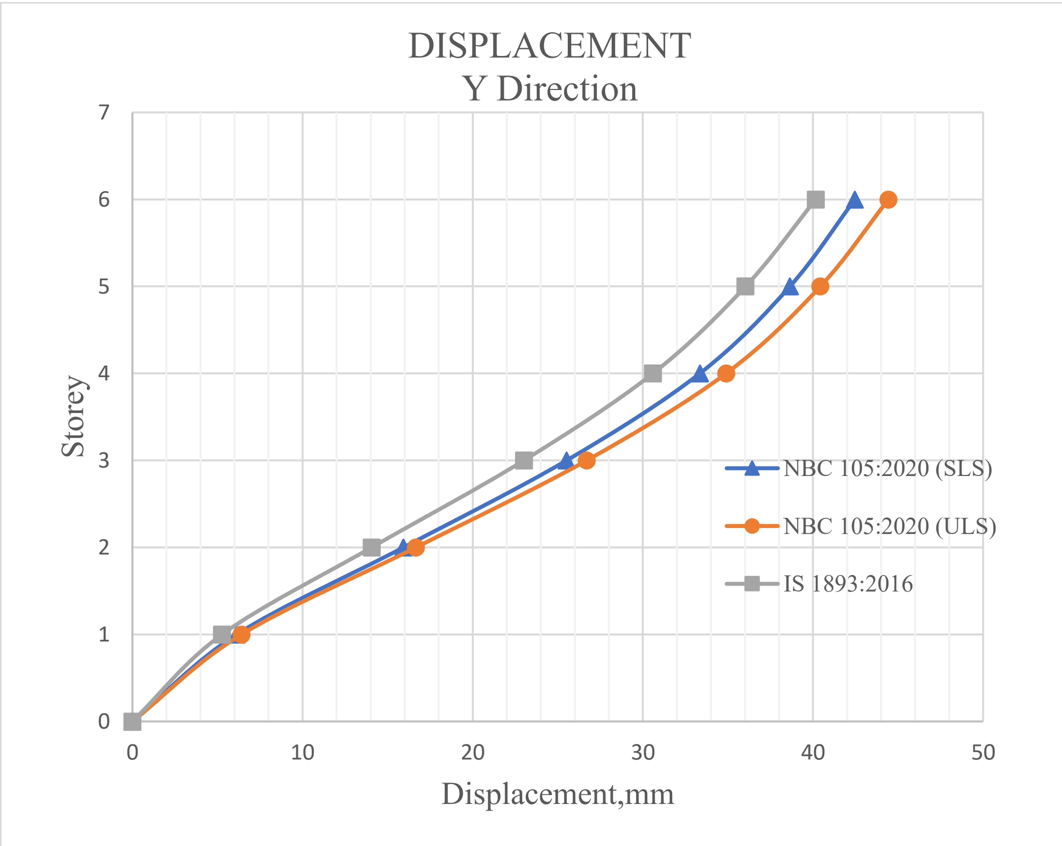

Fig 29: Displacement Comparison Chart for C Shaped Building in Y direction (NBC/IS)

Table 39: Displacement Comparison for E Shaped Building in X direction

X Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	39.891	38.126	36.957
5	36.634	35.013	33.386
4	32.104	30.683	28.661
3	24.874	23.774	21.613
2	15.77	15.072	13.325
1	6.22	5.945	5.146
Base	0	0	0

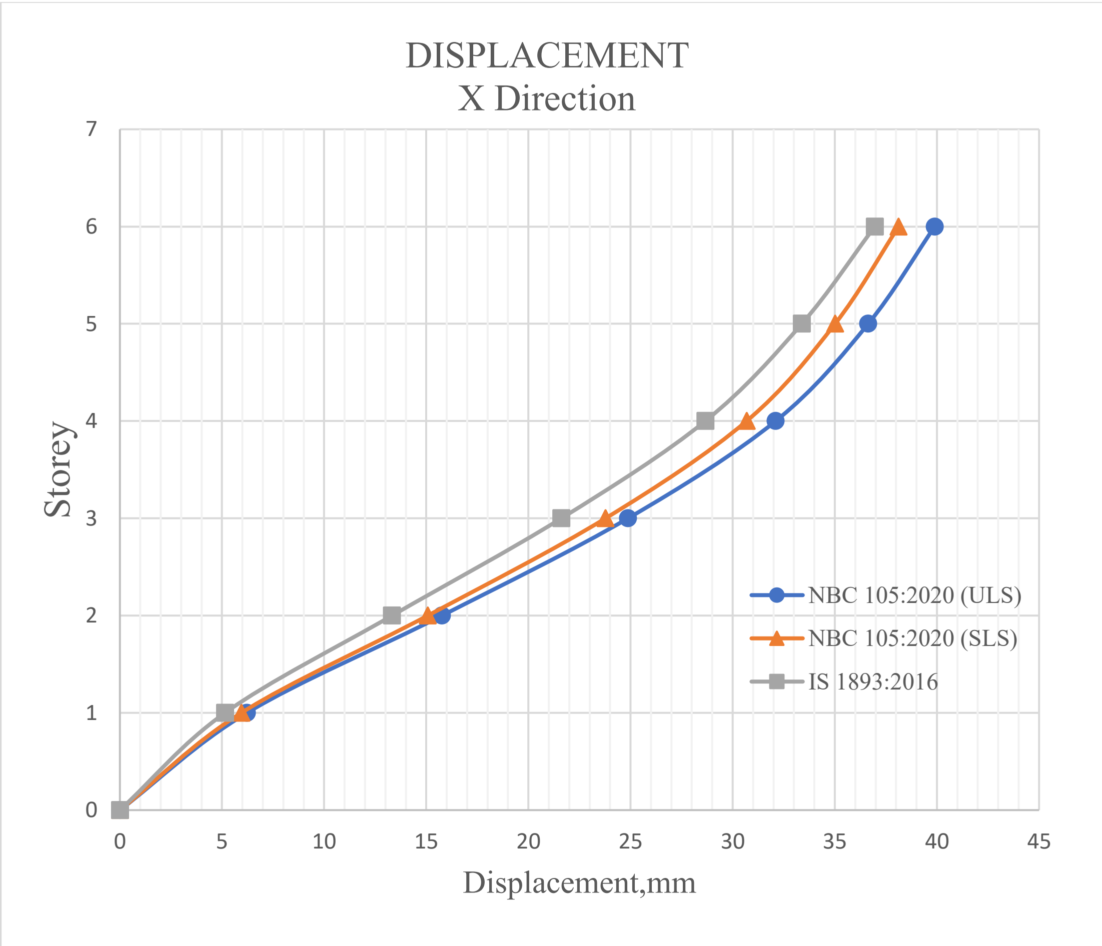

Fig 30: Displacement Comparison Chart for E Shaped Building in X direction (NBC/IS)

Table 40: Displacement Comparison for E Shaped Building in Y direction

Y Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	39.24	37.504	36.234
5	36.07	34.478	32.776
4	31.69	30.288	28.221
3	24.55	23.464	21.3
2	15.46	14.776	13.06
1	5.969	5.705	4.933
0	0	0	0

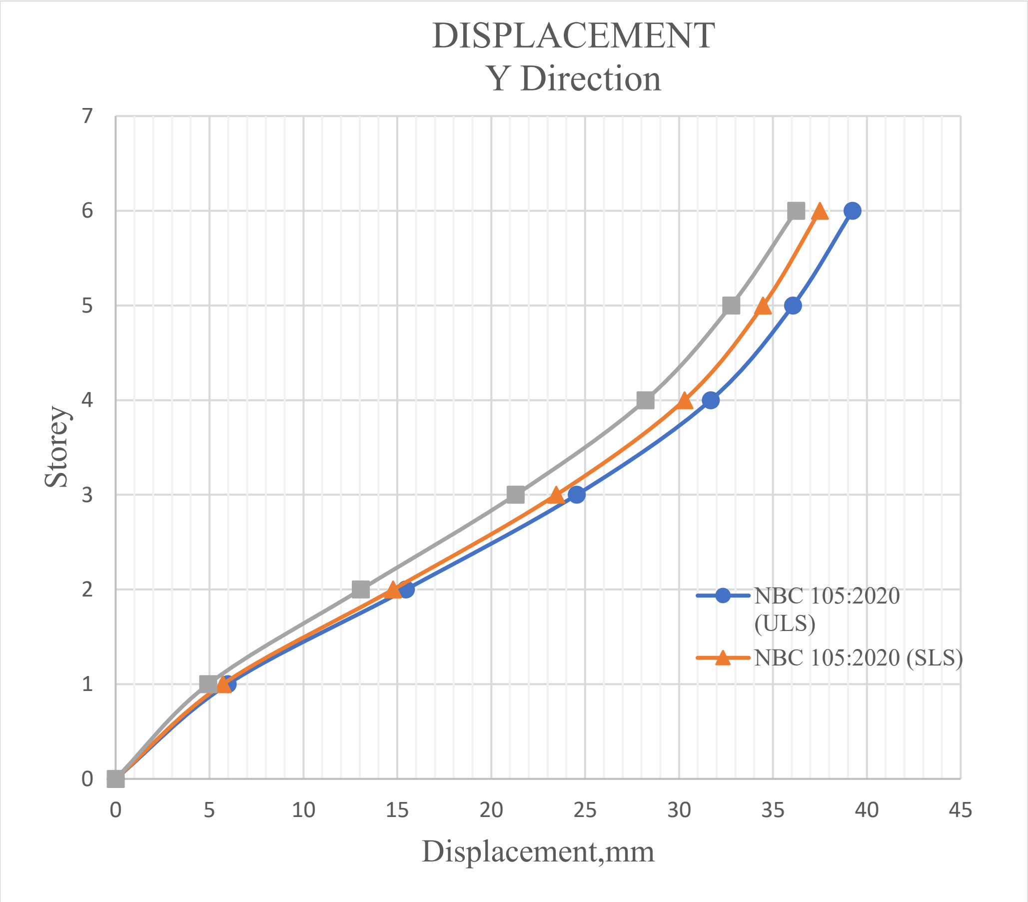

Fig 31: Displacement Comparison Chart for E Shaped Building in Y direction (NBC/IS)

Result: For all buildings, regular and buildings with horizontal irregularities, it is seen that the displacements in both X and Y direction are slightly higher in NBC than in IS.

5.6 Drift Comparison

Table 41: Drift Comparison for regular building in X-direction

X Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	0.001070333	0.001023	0.001149333
5	0.001507333	0.001440667	0.001514667
4	0.00248	0.002370333	0.002339
3	0.003152667	0.003013	0.002789
2	0.003339667	0.003192	0.002793
1	0.002229333	0.002130667	0.001801
Base	0	0	0

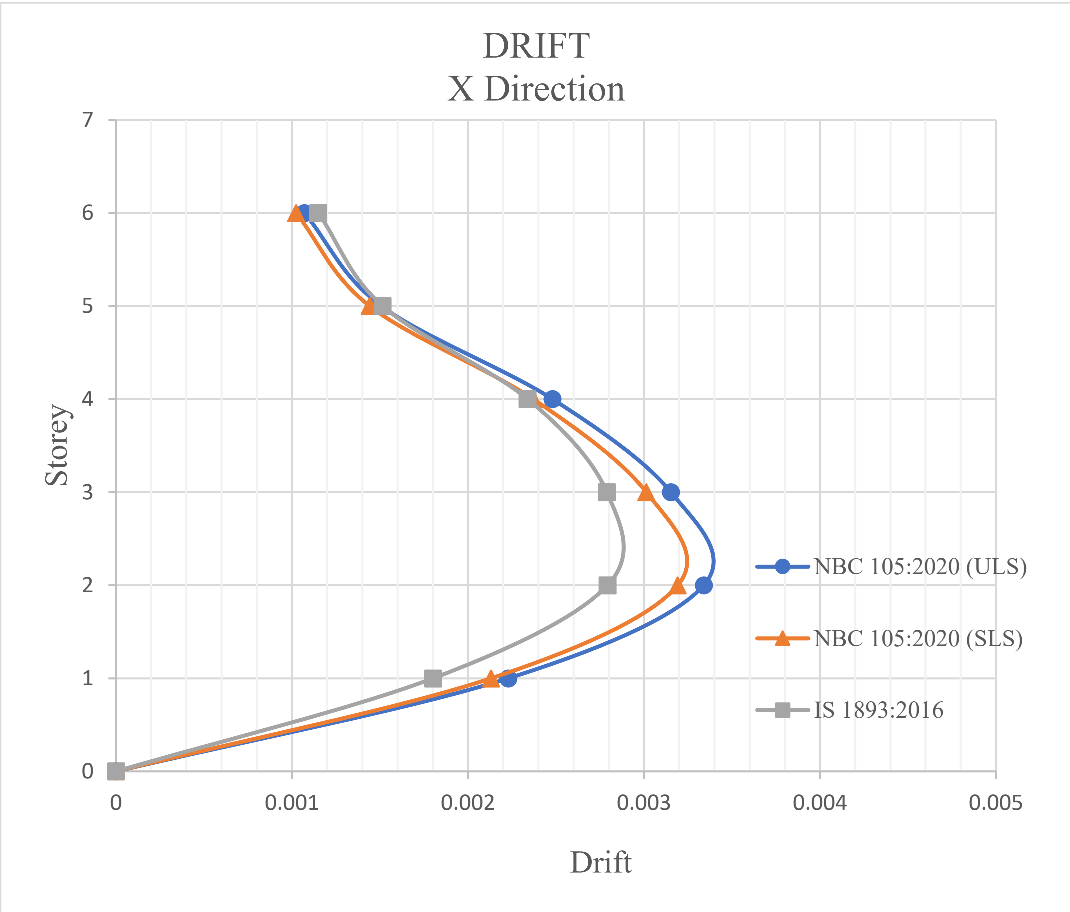

Fig 32: Drift Comparison Chart for regular building in X-Direction (NBC/IS)

Table 42: Drift Comparison for regular building in Y-direction

Y Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	0.001122333	0.001072667	0.001190667
5	0.001621333	0.001549667	0.001614
4	0.002687	0.002568	0.002517333
3	0.003445	0.003292667	0.003032667
2	0.003621667	0.003461667	0.003017667
1	0.002307333	0.002205	0.001856333
0	0	0	0

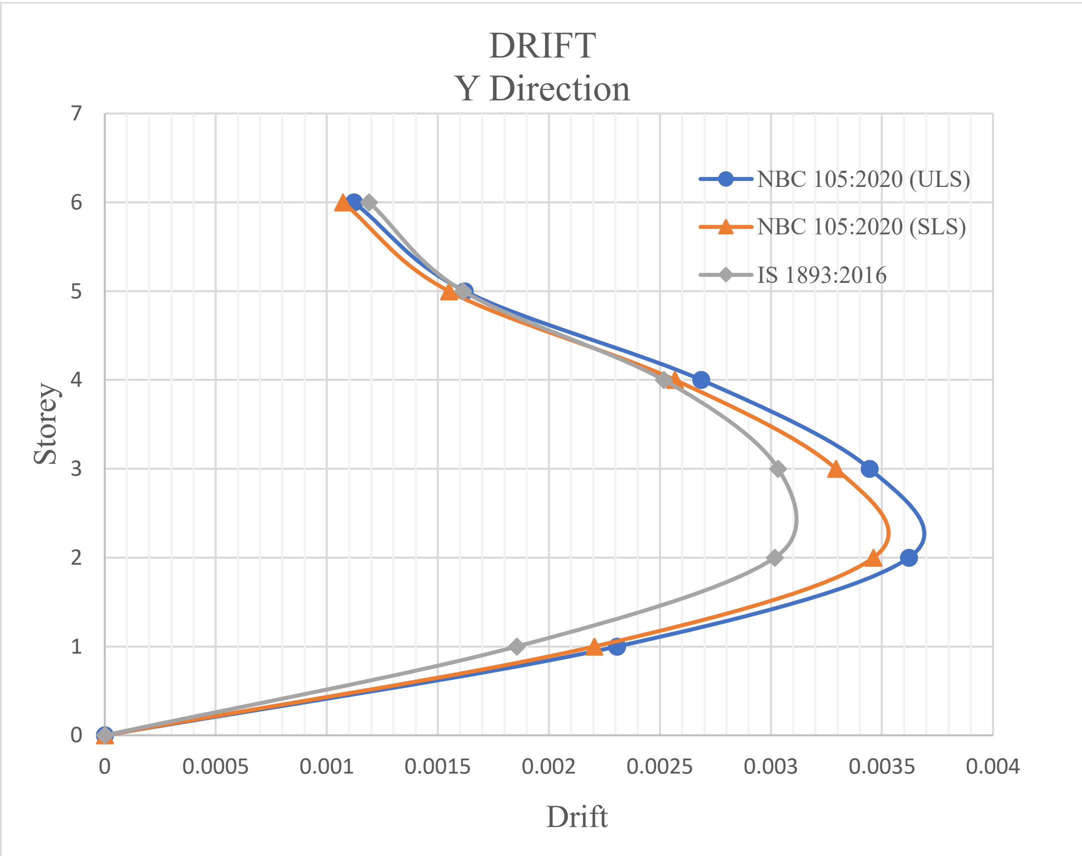

Fig 33: Drift Comparison Chart for regular building in Y-Direction (NBC/IS)

Table 43: Drift Comparison for L Shaped Building in X direction

X Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	0.001026	0.000980667	0.00117
5	0.001390333	0.001329	0.001575667
4	0.002200333	0.002103	0.002205
3	0.002759	0.002636667	0.002556667
2	0.002900667	0.002772667	0.002515
1	0.001923	0.001837667	0.001607333
Base	0	0	0

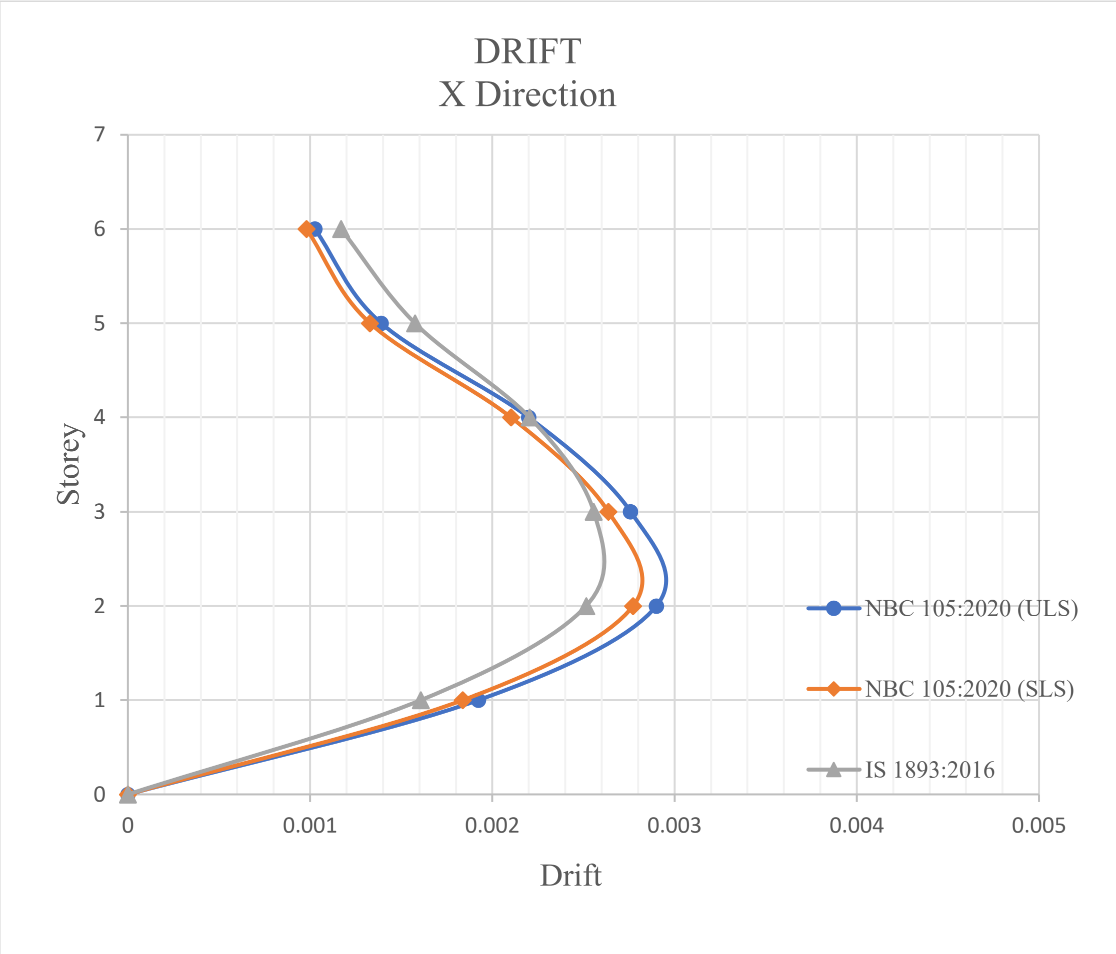

Fig 34: Drift Comparison Chart for L Shaped building in X-Direction (NBC/IS)

Table 44: Drift Comparison for L Shaped Building in Y direction

Y Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	0.001251333	0.001196	0.001401667
5	0.001697	0.001622	0.001859333
4	0.002574333	0.002460333	0.002519
3	0.003205333	0.003063333	0.002913333
2	0.003305667	0.003159333	0.002818667
1	0.002038333	0.001948333	0.001679
0	0	0	0

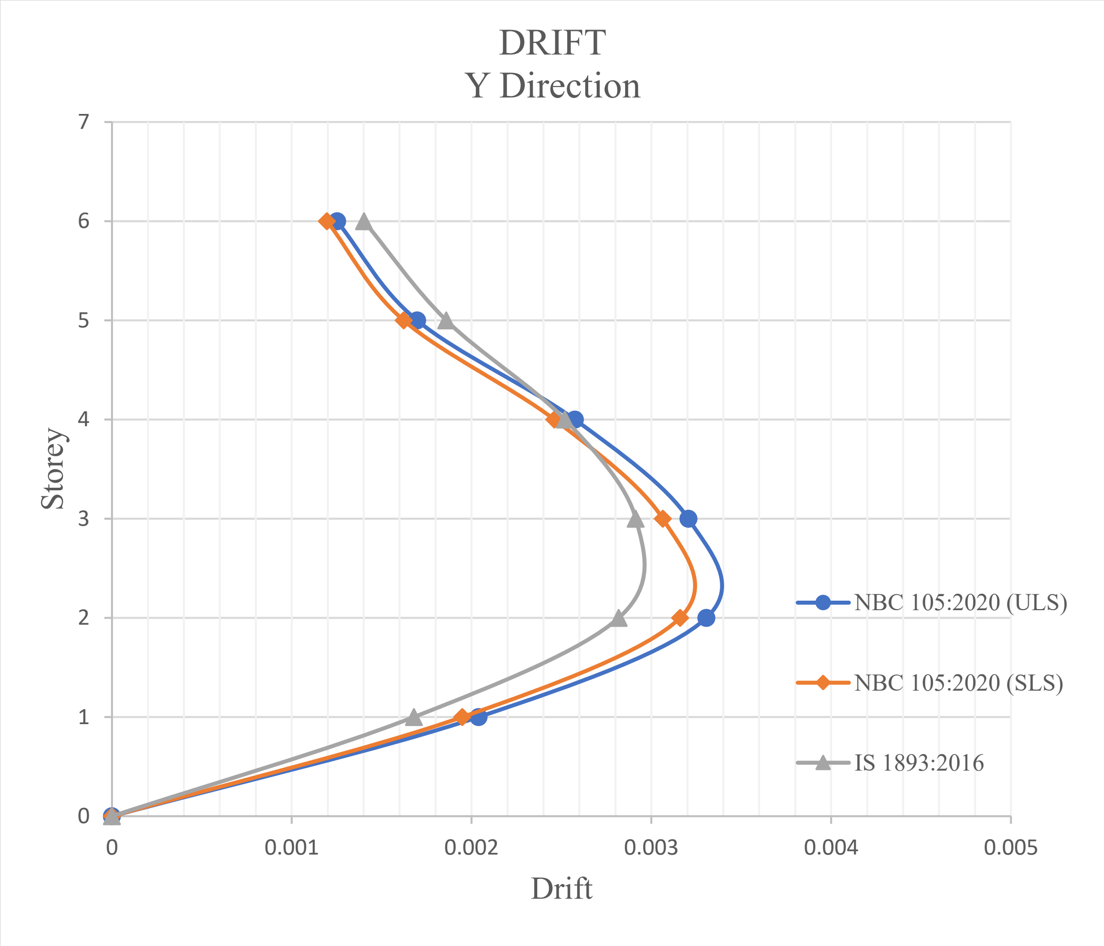

Fig 35: Drift Comparison Chart for L Shaped building in Y-Direction (NBC/IS)

Table 45: Drift Comparison for C Shaped Building in X direction

X Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	0.001188	0.001135333	0.001252667
5	0.001763	0.001685	0.001739
4	0.002632	0.002515667	0.002419667
3	0.003187667	0.003046667	0.002780667
2	0.003203	0.003061	0.002755633
1	0.002098667	0.002006	0.001671033
Base	0	0	0

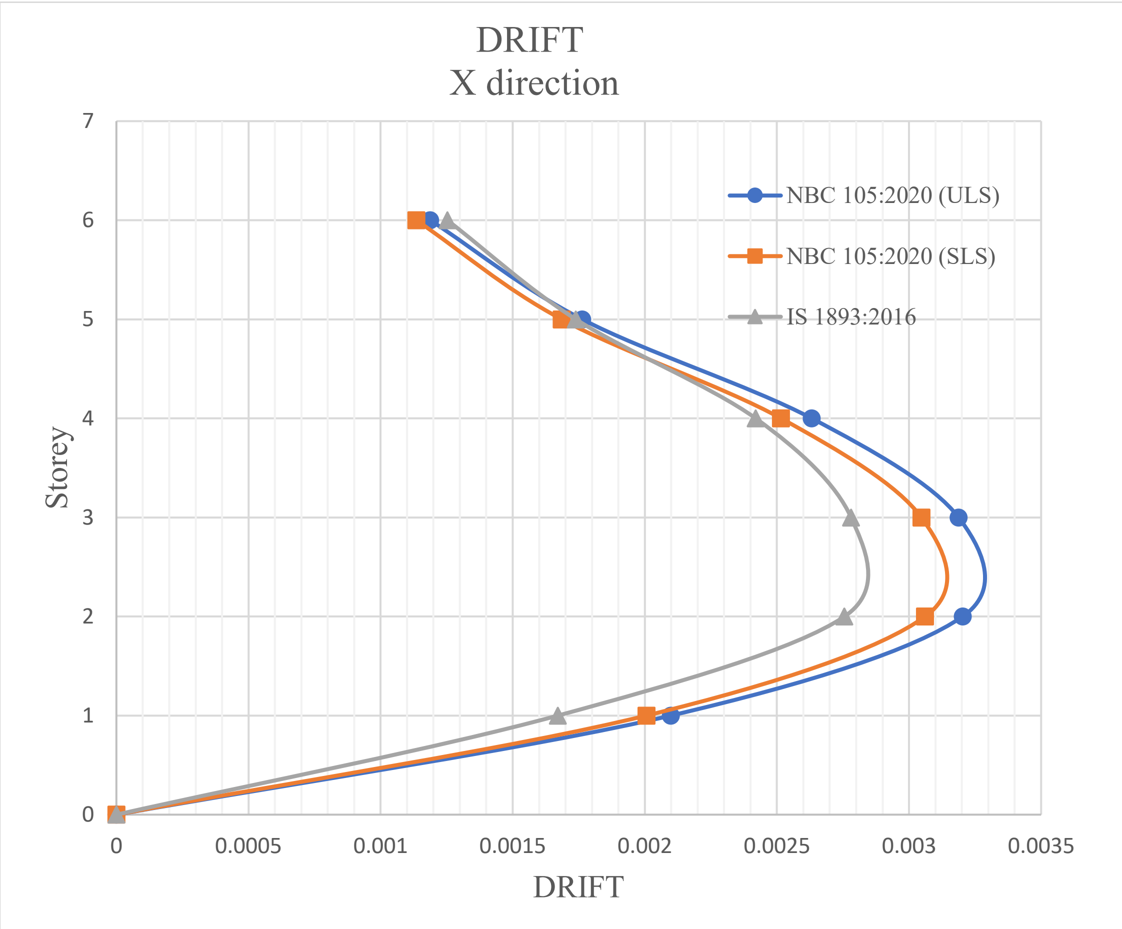

Fig 36: Drift Comparison Chart for C Shaped building in X-Direction (NBC/IS)

Table 46: Drift Comparison for C Shaped Building in Y direction

Y Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	0.001332667	0.001273667	0.001377
5	0.001846333	0.001764667	0.001813
4	0.002731667	0.002610667	0.002528667
3	0.003345667	0.003198	0.002984
2	0.003415667	0.003264333	0.002929333
1	0.002139	0.002044333	0.001757333
0	0	0	0

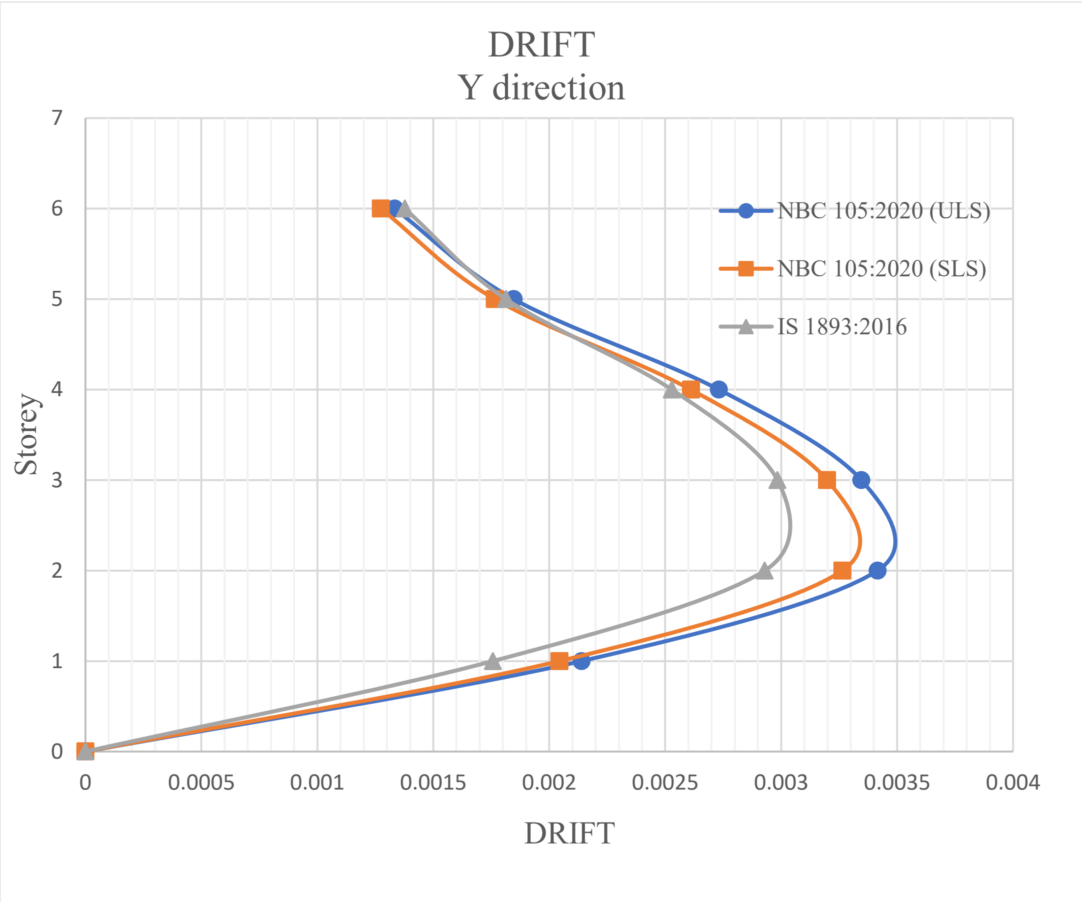

Fig 37: Drift Comparison Chart for C Shaped building in Y-Direction (NBC/IS)

Table 47: Drift Comparison for E Shaped Building in X direction

X Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	0.001085667	0.001037667	0.001190333
5	0.00151	0.001443333	0.001575
4	0.00241	0.002303	0.002349333
3	0.003034667	0.002900667	0.002762667
2	0.003183333	0.003042333	0.002726333
1	0.002073333	0.001981667	0.001715333
Base	0	0	0

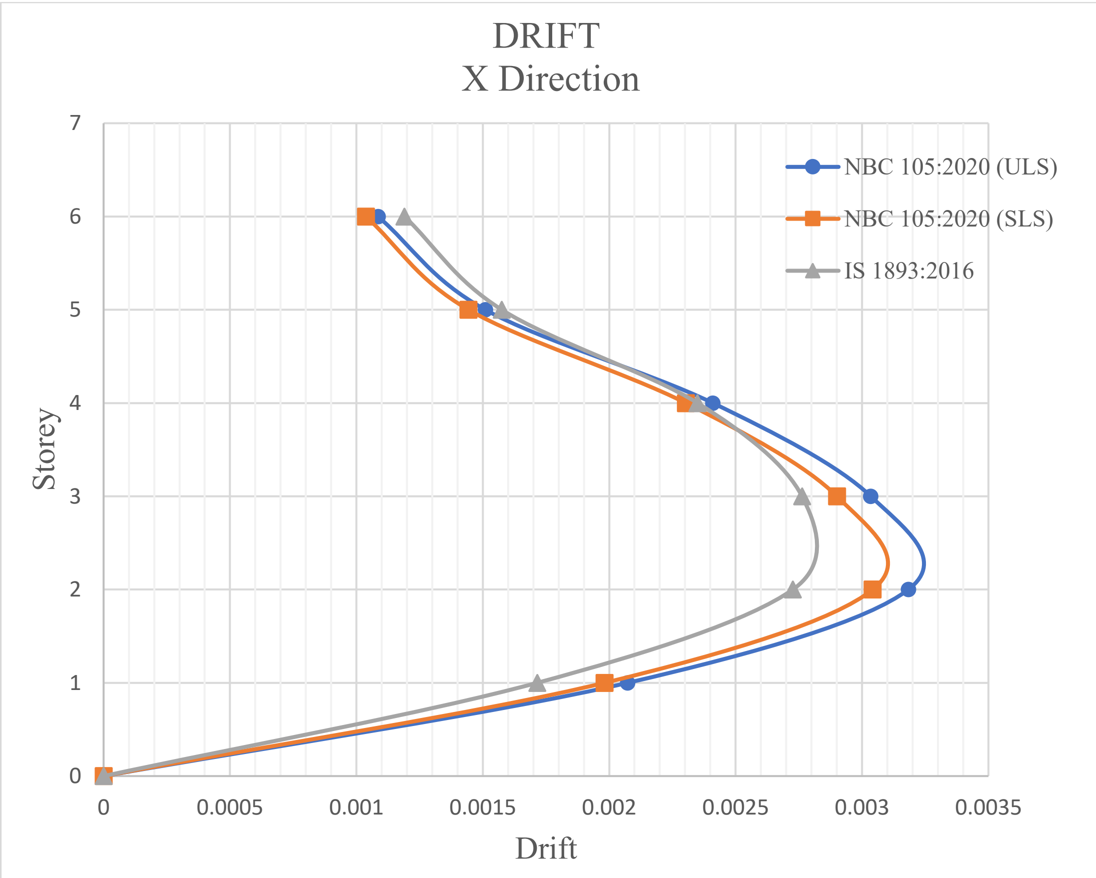

Fig 38: Drift Comparison Chart for E Shaped building in X-Direction (NBC/IS)

Table 48: Drift Comparison for E Shaped Building in Y direction

Y Direction			
Story	NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
6	0.001056667	0.001008667	0.001152667
5	0.00146	0.001396667	0.001518333
4	0.00238	0.002274667	0.002307
3	0.00303	0.002896	0.002746667
2	0.003163667	0.003023667	0.002709
1	0.001989667	0.001901667	0.001644333
0	0	0	0

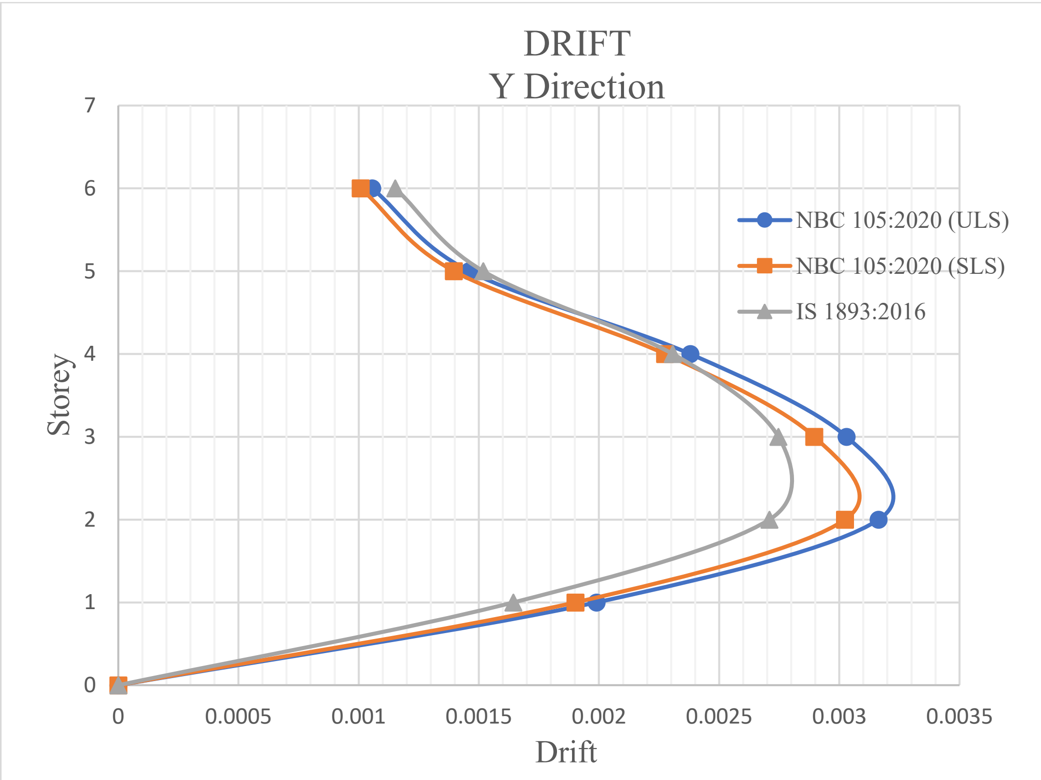

Fig 39: Drift Comparison Chart for E Shaped building in Y-Direction (NBC/IS)

Result: From the above charts, it is seen that NBC gives higher value for maximum drift than the IS in both X and Y directions for both regular and irregular type of buildings.

5.7 Torsion Comparison

Table 49: Torsion Comparison for Regular Building in X direction

X Direction		
NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
1.254	1.254	1.167

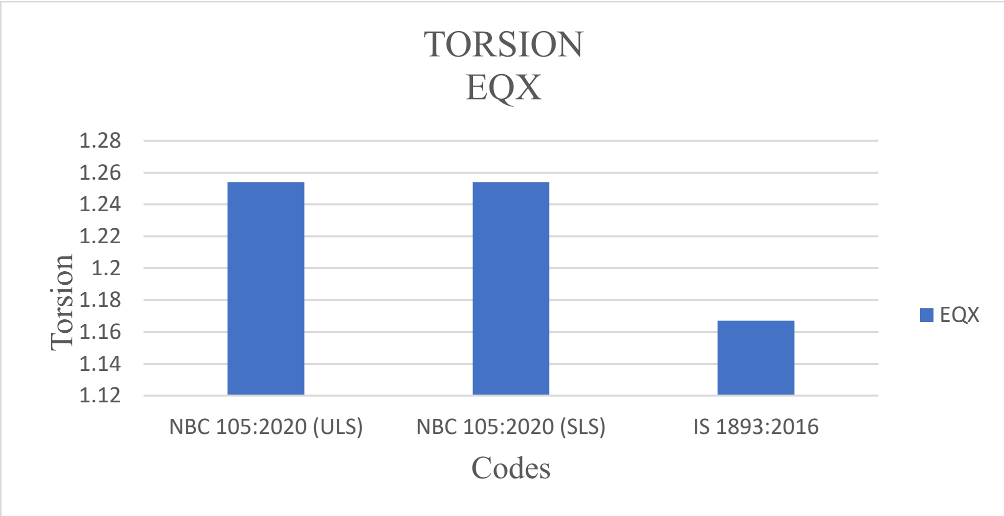

Fig 40: Torsion Comparison Chart for Regular Building in X-Direction (NBC/IS)

Table 50: Torsion Comparison for Regular Building for Y direction

Y Direction		
NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
1.384	1.384	1.217

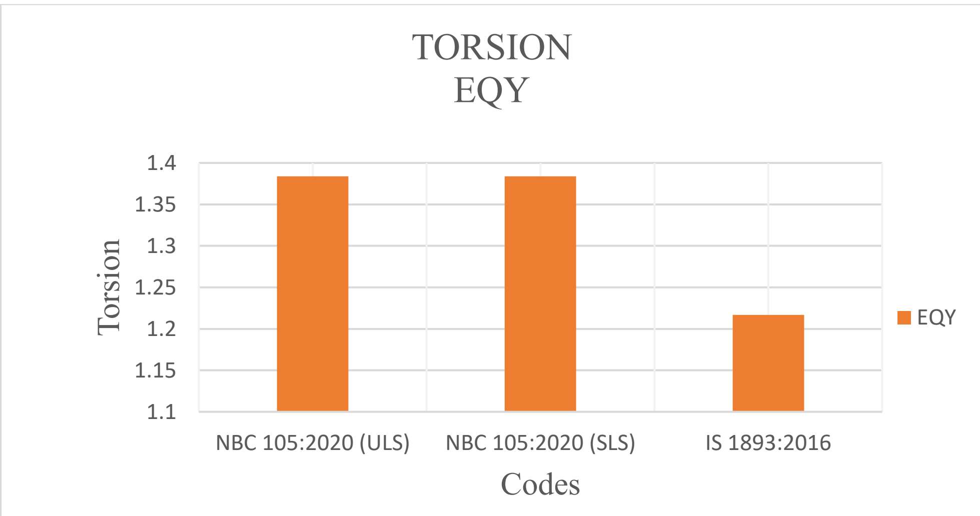

Fig 41: Torsion Comparison Chart for Regular Building in Y-Direction (NBC/IS)

Table 51: Torsion Comparison for L Shaped Building in X direction

X Direction		
NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
1.292	1.292	1.19

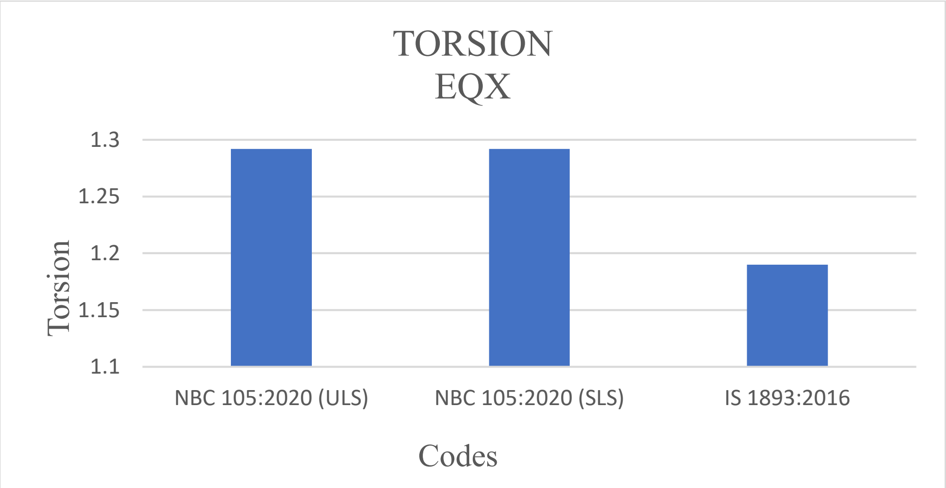

Fig 42: Torsion Comparison Chart for L Shaped Building in X-Direction (NBC/IS)

Table 52: Torsion Comparison for L Shaped Building in Y direction

Y Direction		
NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
1.609	1.609	1.17

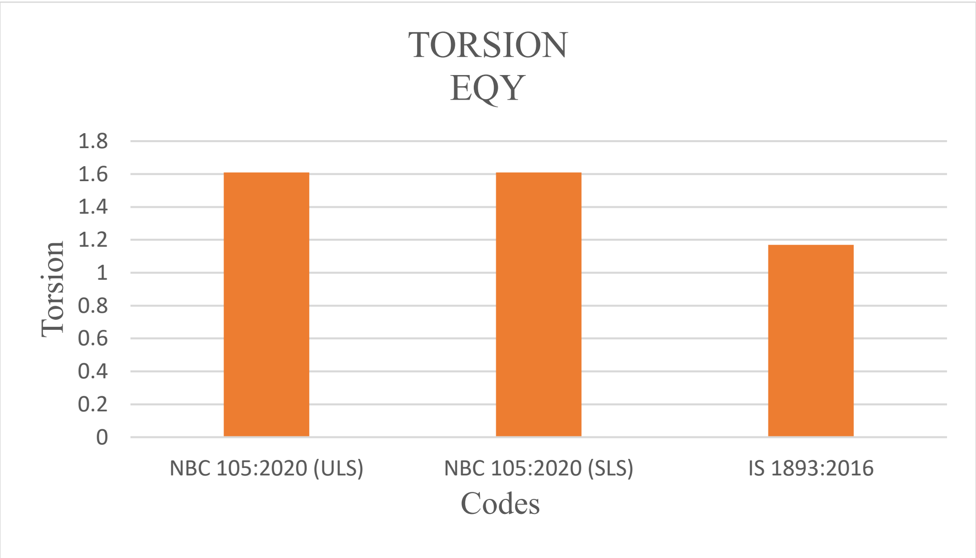

Fig 43: Torsion Comparison Chart for L Shaped Building in Y-Direction (NBC/IS)

Table 53: Torsion Comparison for C Shaped Building in X direction

X Direction		
NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
1.167	1.167	1.188

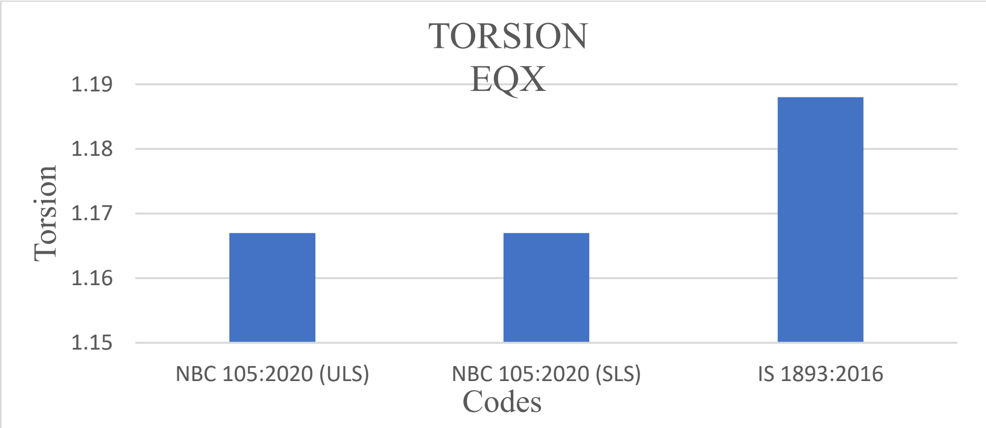

Fig 44: Torsion Comparison Chart for L Shaped Building in X-Direction (NBC/IS)

Table 54: Torsion Comparison for C Shaped Building in Y direction

Y Direction		
NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
1.104	1.104	1.057

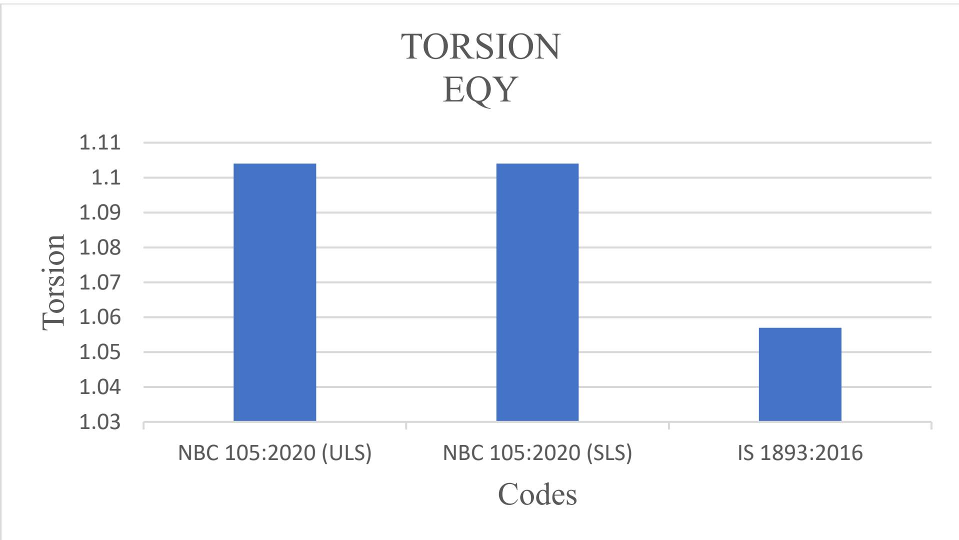

Fig 45: Torsion Comparison Chart for C Shaped Building in Y-Direction (NBC/IS)

Table 55: Torsion Comparison for E Shaped Building in X direction

X Direction		
NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
1.236	1.236	1.136

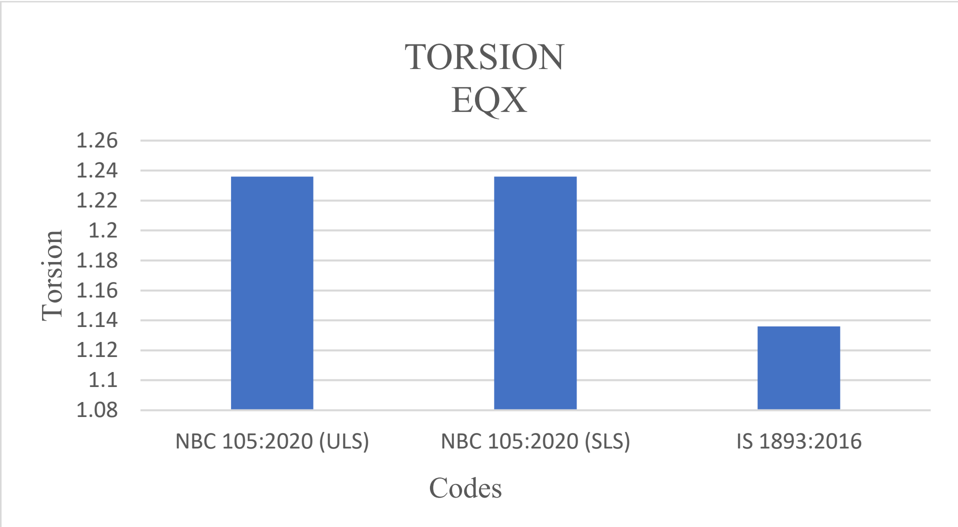

Fig 46: Torsion Comparison Chart for E Shaped Building in X-Direction (NBC/IS)

Table 56: Torsion Comparison for E Shaped Building in Y direction

X Direction		
NBC 105:2020 (ULS)	NBC 105:2020 (SLS)	IS 1893:2016
1.445	1.445	1.096

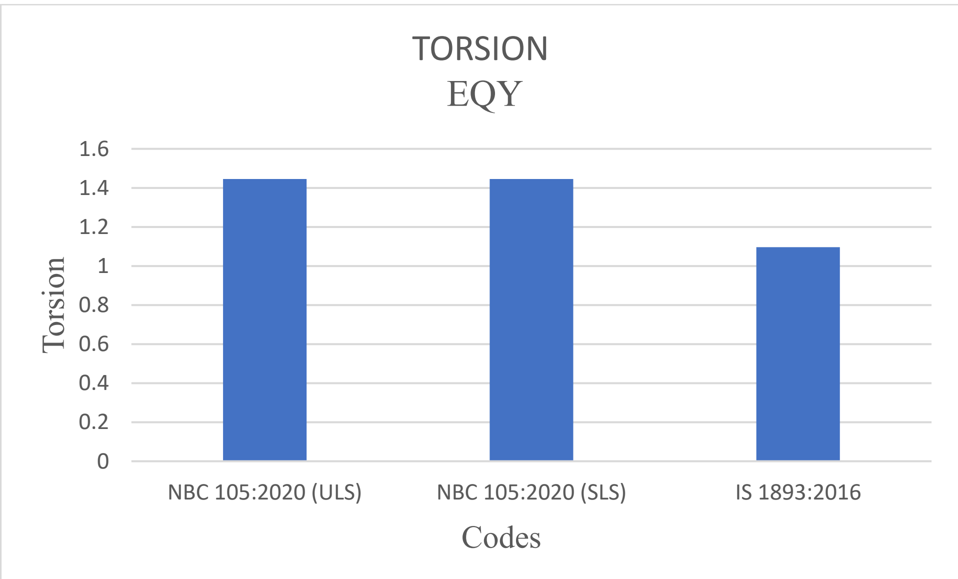

Fig 47: Torsion Comparison Chart for E Shaped Building in Y-Direction (NBC/IS)

Result: The torsion varies for irregularities. From the above charts, it is seen that: For regular building, the torsion is higher in NBC in both X and Y directions. For L shaped building, torsion is higher in NBC in both X and Y directions. For C shaped building, the torsion is higher in IS in X direction and higher in NBC in Y direction. For E shaped building, the torsion is higher in IS in both X and Y directions.

Although IS increases eccentricity uniformly for all shapes, the C-shape's high asymmetry causes amplified torsional response when eccentricity is modified by 1.5. NBC distributes torsion over multiple modes, leading to lower peak torsion in C-shaped buildings despite generally showing higher torsion in other shapes.

Hence, IS 1893:2016 tends to be more conservative for buildings with higher asymmetry, such as C-shaped structures, due to its amplified eccentricity factor (1.5e), which significantly increases torsional response in already irregular configurations.

5.8 Comparison of rebar percentage (in column)

Table 57: Maximum Rebar % Comparison for Regular Building

Regular Building	Rebar %
IS	2.21%
NBC	2.94%

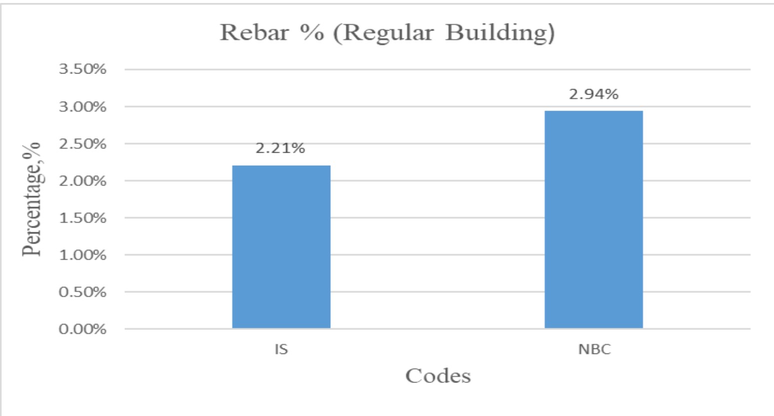

Fig 48: Maximum Rebar % Comparison chart for Regular Building

Result: NBC gave maximum rebar % greater by 28% for regular building.

Table 58: Maximum Rebar % Comparison for L Shaped Building

L Shaped Building	Rebar %
IS	2.03%
NBC	2.85%

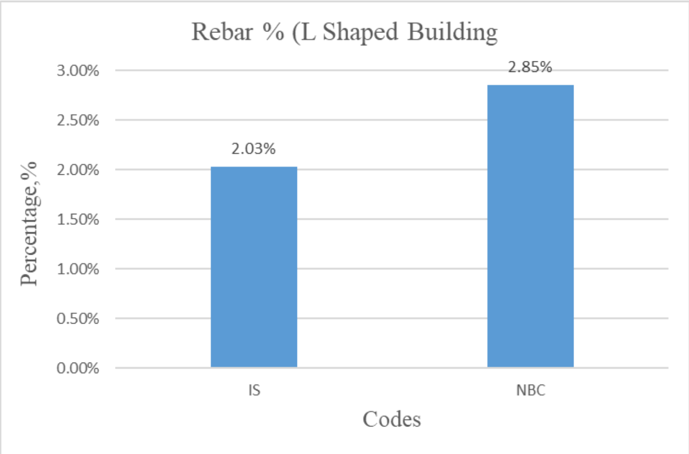

Fig 49: Maximum Rebar % Comparison chart for L shaped Building

Result: NBC gave maximum rebar % greater by 33.06% for L shaped building.

Table 60: Maximum Rebar % Comparison for C Shaped Building

C Shaped Building	Rebar %
IS	2.10%
NBC	2.76%

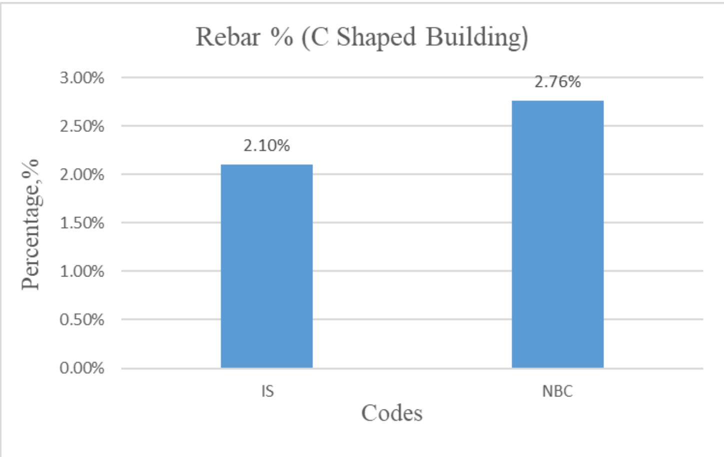

Fig 50: Maximum Rebar % Comparison chart for C shaped Building

Result: NBC gave maximum rebar % greater by 27.16% for C shaped building.

Table 59: Maximum Rebar % Comparison for E Shaped Building

E Shaped Building	Rebar %
IS	1.81%
NBC	2.55%

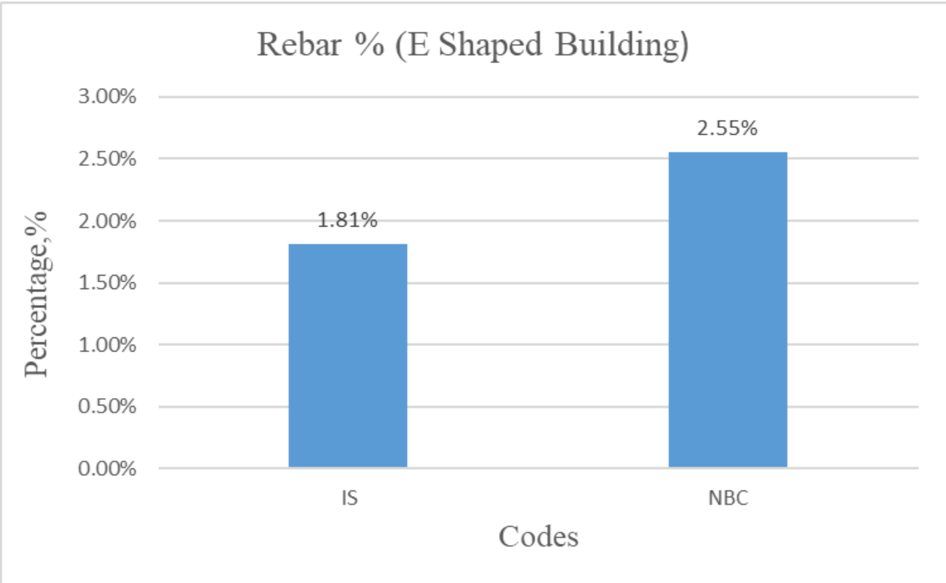

Fig 51: Maximum Rebar % Comparison chart for E shaped Building

Result: NBC gave maximum rebar % greater by 33.94% for E shaped building.

Result: From the above charts, it is evident that the maximum percentage of rebar for column is higher by more than 20% in NBC than in IS for both regular and irregular buildings.

5.9 Comparison of Regular and Irregular Buildings

5.9.1 Displacement Comparison

According to NBC 105:2020

Table 61: Displacement Comparison in X-Direction For Ultimate Limit State(ULS)

X Direction				
Story	Regular ULS	L Shaped ULS	C Shaped ULS	E Shaped ULS
6	41.338	36.598	42.217	39.891
5	38.127	33.52	38.653	36.634
4	33.605	29.349	33.364	32.104
3	26.165	22.748	25.468	24.874
2	16.707	14.471	15.905	15.77
1	6.688	5.769	6.296	6.22
Base	0	0	0	0

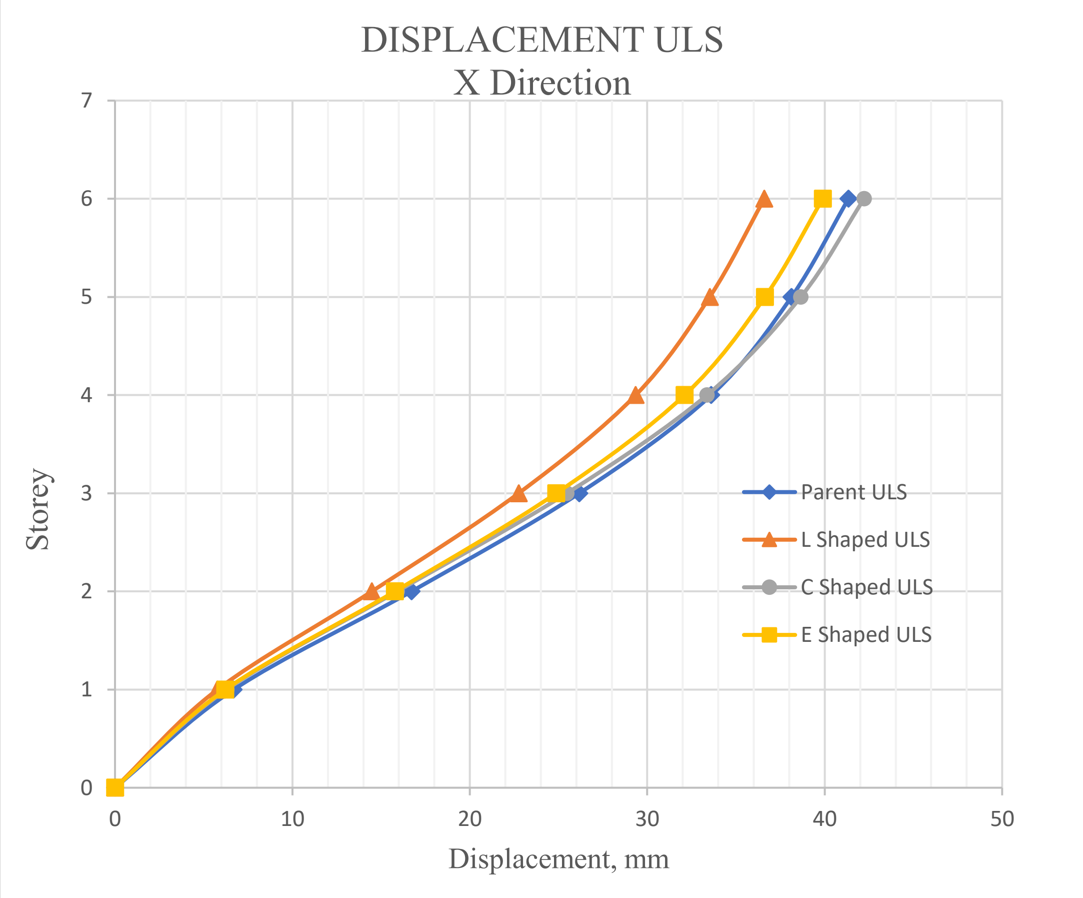

Fig 52: Displacement Comparison graph in X Direction for ULS

Table 62: Displacement Comparison in X-Direction For Serviceability Limit State(SLS)

X Direction				
Story	Regular SLS	L Shaped SLS	C Shaped SLS	E Shaped SLS
6	39.509	34.979	40.349	38.126
5	36.44	32.037	36.943	35.013
4	32.118	28.05	31.888	30.683
3	25.007	21.741	24.341	23.774
2	15.968	13.831	15.201	15.072
1	6.392	5.513	6.018	5.945
Base	0	0	0	0

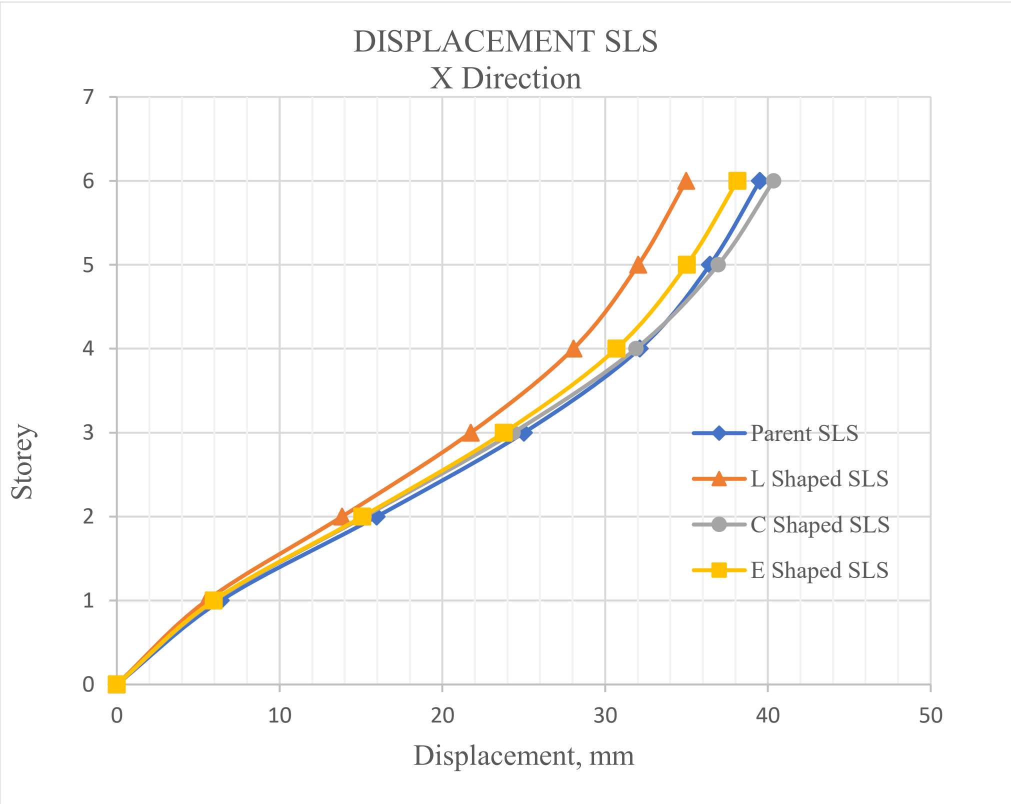

Fig 53: Displacement Comparison graph in X Direction for SLS

Result: From the above charts, the displacement is maximum for C shaped building, then regular building and E shaped building and least for L shaped building as per NBC (SLS) and NBC (ULS) in X direction.

Table 63: Displacement Comparison in Y-Direction For Ultimate Limit State(ULS)

Y Direction				
Story	RegularULS	L Shaped ULS	C Shaped ULS	E Shaped ULS
6	44.414	42.216	44.433	39.24
5	41.047	38.462	40.435	36.07
4	36.183	33.371	34.896	31.69
3	28.122	25.648	26.701	24.55
2	17.787	16.032	16.664	15.46
1	6.922	6.115	6.417	5.969
0	0	0	0	0

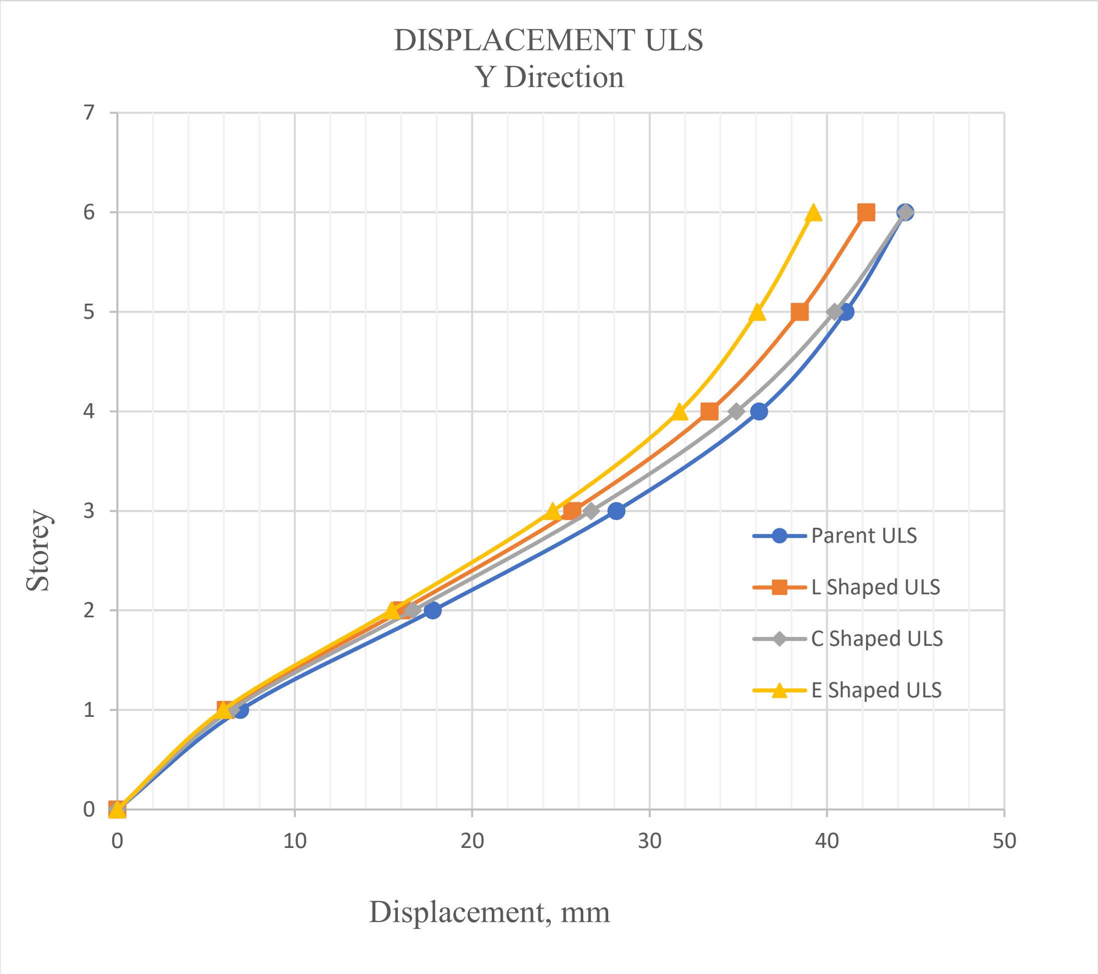

Fig 54: Displacement Comparison graph in Y Direction for ULS

Table 64: Displacement Comparison in Y-Direction For Serviceability Limit State(SLS)

Y Direction				
Story	Regular SLS	L Shaped SLS	C Shaped SLS	E Shaped SLS
6	42.449	40.348	42.467	37.504
5	39.231	36.76	38.646	34.478
4	34.582	31.894	33.352	30.288
3	26.878	24.513	25.52	23.464
2	17	15.323	15.926	14.776
1	6.615	5.845	6.133	5.705
Base	0	0	0	0

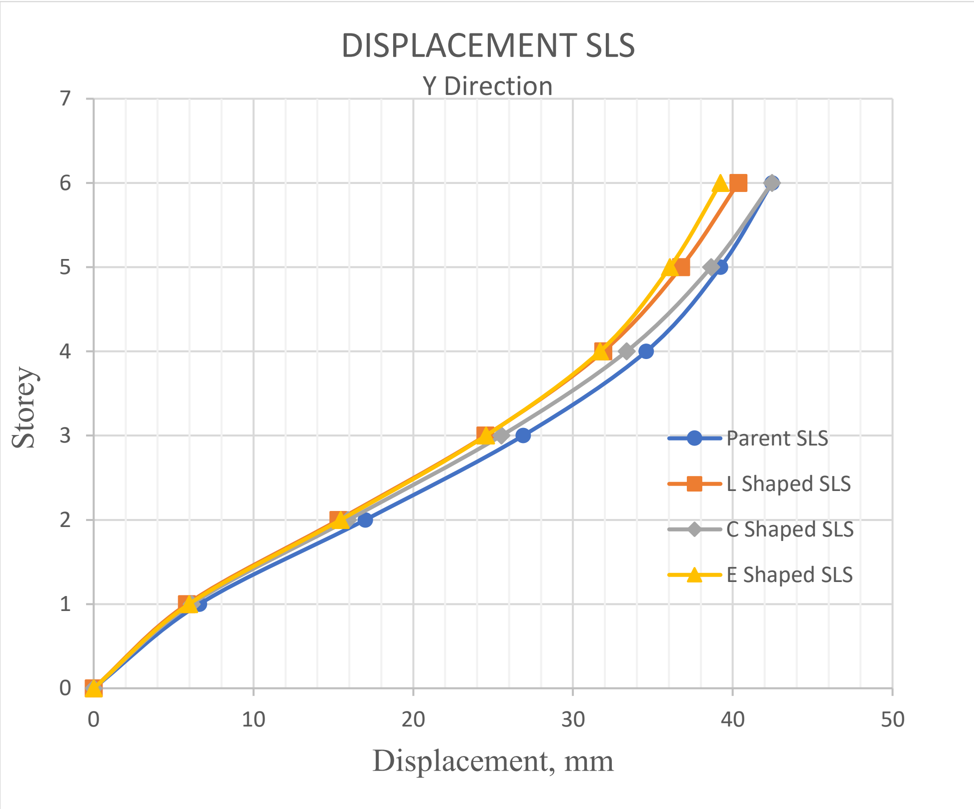

Fig 55: Displacement Comparison graph in Y Direction for SLS

Result: From the above charts, the displacement is maximum for C shaped building, then regular building and L shaped building and least for E shaped building as per NBC (SLS) and NBC (ULS) in Y direction.

According to IS 1893:2016

Table 65: Displacement Comparison in X-Direction

X-Direction				
Story	Regular	L Shaped	C Shaped	E Shaped
6	37.158	34.889	37.856	36.957
5	33.71	31.379	34.098	33.386
4	29.166	26.652	28.881	28.661
3	22.149	20.037	21.622	21.613
2	13.782	12.367	13.28	13.325
1	5.403	4.822	5.0131	5.146
Base	0	0	0	0

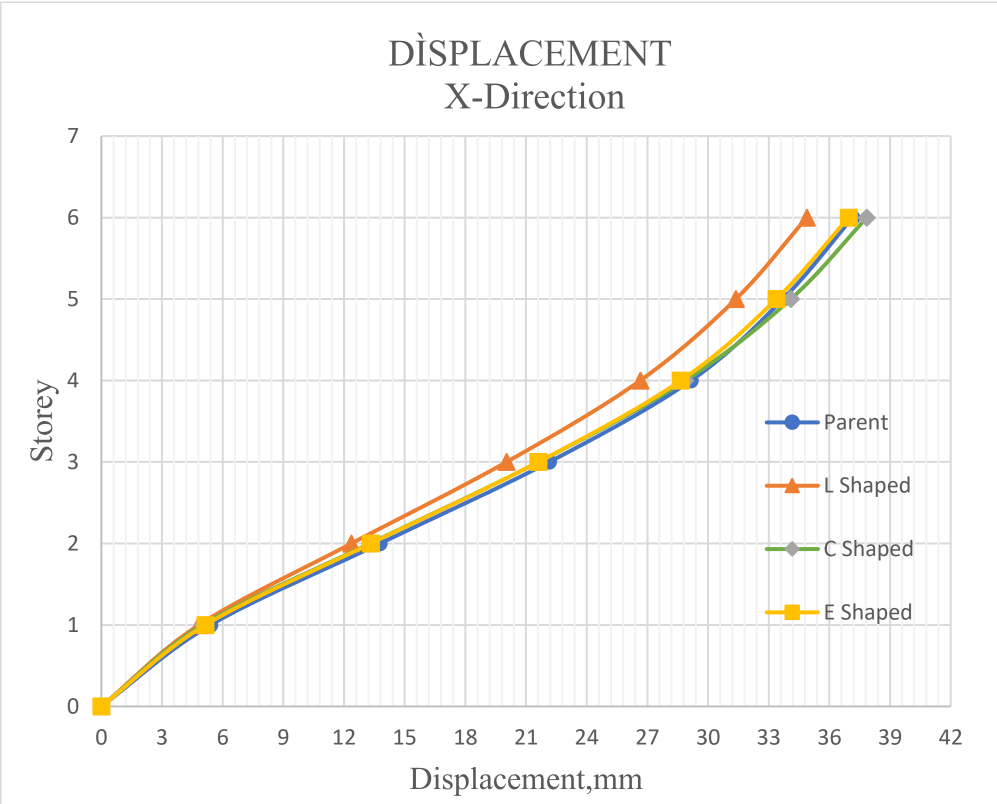

Fig 56: Displacement Comparison graph in X Direction

Result: From the above chart, the displacement is maximum for C shaped building, then regular building and E shaped building and least for L shaped building as per IS in X direction

Table 66: Displacement Comparison in Y-Direction

Y-Direction				
Story	Regular	L Shaped	C Shaped	E Shaped
6	39.686	39.573	40.168	36.234
5	36.114	35.368	36.037	32.776
4	31.272	29.79	30.598	28.221
3	23.72	22.233	23.012	21.3
2	14.622	13.493	14.06	13.06
1	5.569	5.037	5.272	4.933
Base	0	0	0	0

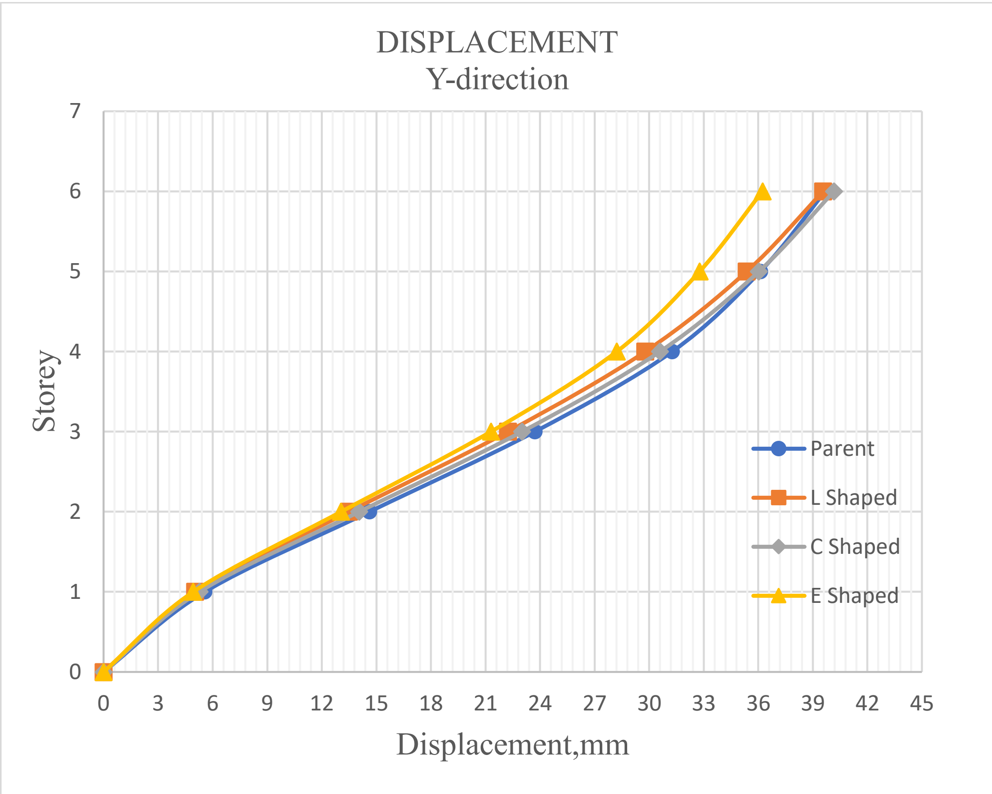

Fig 57: Displacement Comparison graph in Y Direction

Result: From the above chart, the displacement is maximum for C shaped building, then regular building and L shaped building and least for E shaped building as per IS in Y direction.

5.9.2 Drift Comparison

According to NBC 105:2020

Table 67: Drift Comparison in X-Direction For Ultimate Limit State(ULS)

X Direction				
Story	Regular ULS	L Shaped ULS	C Shaped ULS	E Shaped ULS
6	0.001070333	0.001026	0.001188	0.001085667
5	0.001507333	0.001390333	0.001763	0.00151
4	0.00248	0.002200333	0.002632	0.00241
3	0.003152667	0.002759	0.003187667	0.003034667
2	0.003339667	0.002900667	0.003203	0.003183333
1	0.002229333	0.001923	0.002098667	0.002073333
Base	0	0	0	0

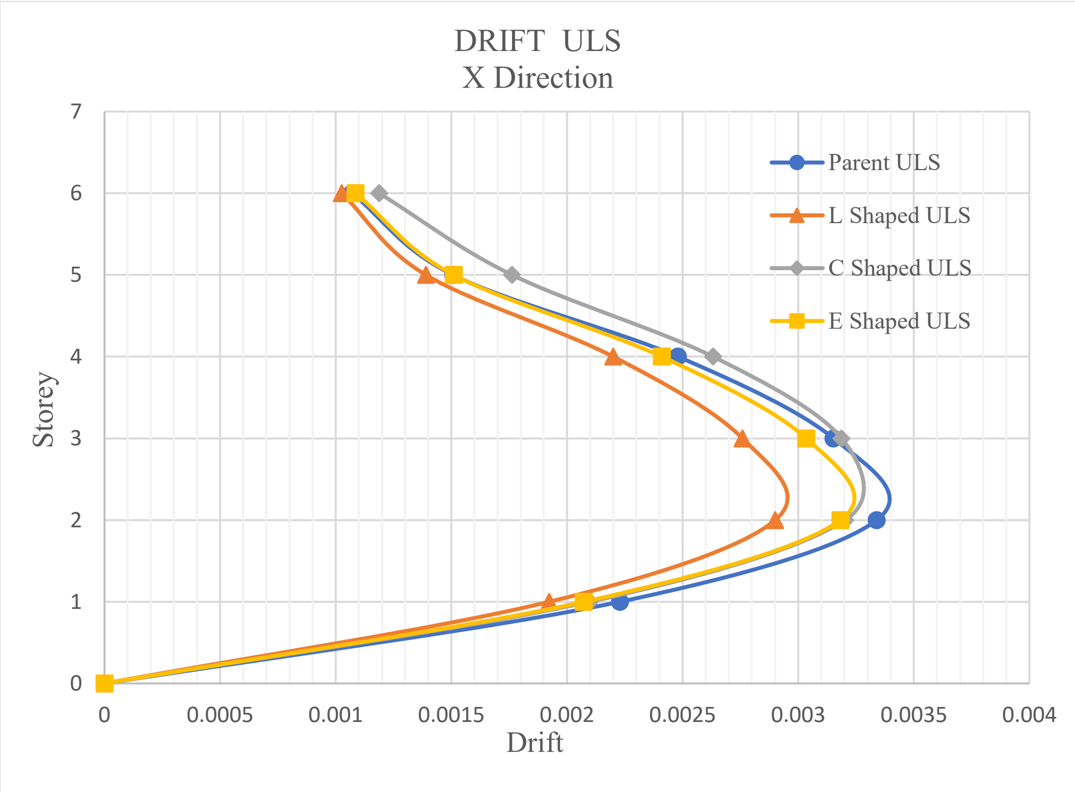

Fig 58: Drift Comparison graph in X Direction for ULS

Table 68: Drift Comparison in X-Direction For Serviceability Limit State(SLS)

X Direction				
Story	Regular SLS	L Shaped SLS	C Shaped SLS	E Shaped SLS
6	0.001023	0.000980667	0.001135333	0.001037667
5	0.001440667	0.001329	0.001685	0.001443333
4	0.002370333	0.002103	0.002515667	0.002303
3	0.003013	0.002636667	0.003046667	0.002900667
2	0.003192	0.002772667	0.003061	0.003042333
1	0.002130667	0.001837667	0.002006	0.001981667
Base	0	0	0	0

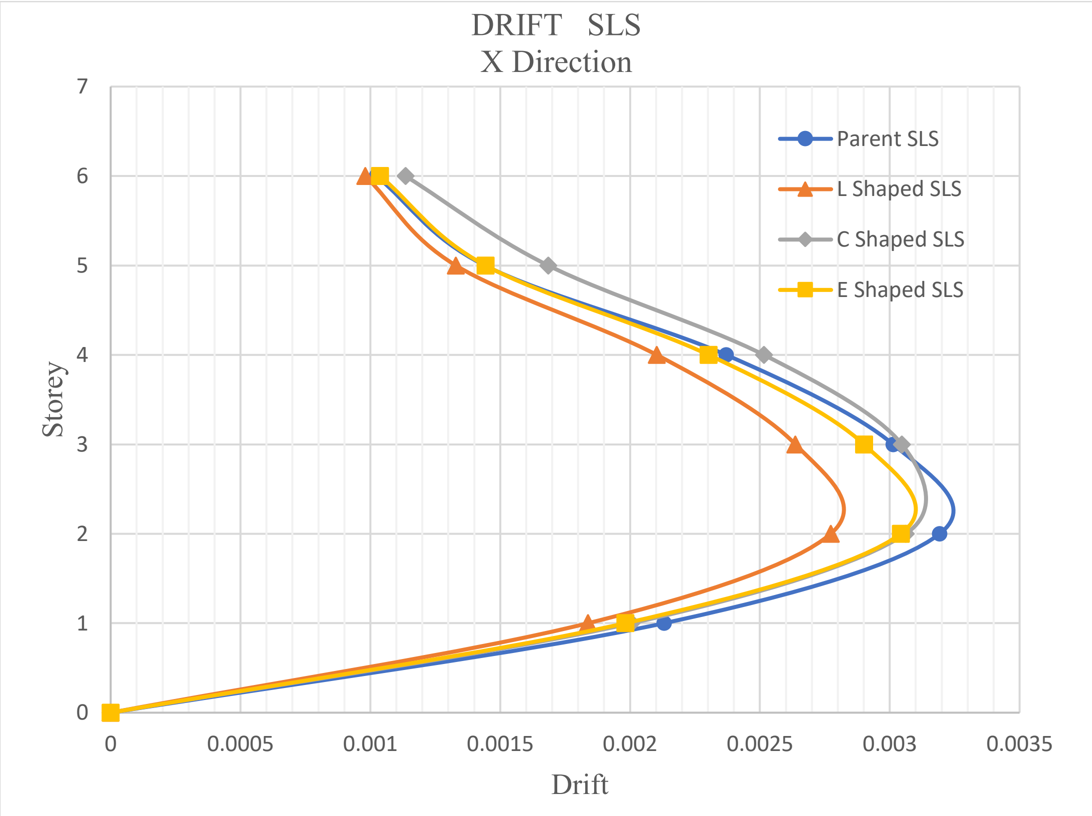

Fig 59: Drift Comparison graph in X Direction for SLS

According to IS 1893:2016

Table 69: Drift Comparison in X-Direction as per IS

X-Direction				
Story	Regular	L Shaped	C Shaped	E Shaped
6	0.00114933	0.00117	0.001252667	0.001190333
5	0.00151467	0.00157567	0.001739	0.001575
4	0.002339	0.002205	0.002419667	0.002349333
3	0.002789	0.00255667	0.002780667	0.002762667
2	0.002793	0.002515	0.002755633	0.002726333
1	0.001801	0.00160733	0.001671033	0.001715333
Base	0	0	0	0

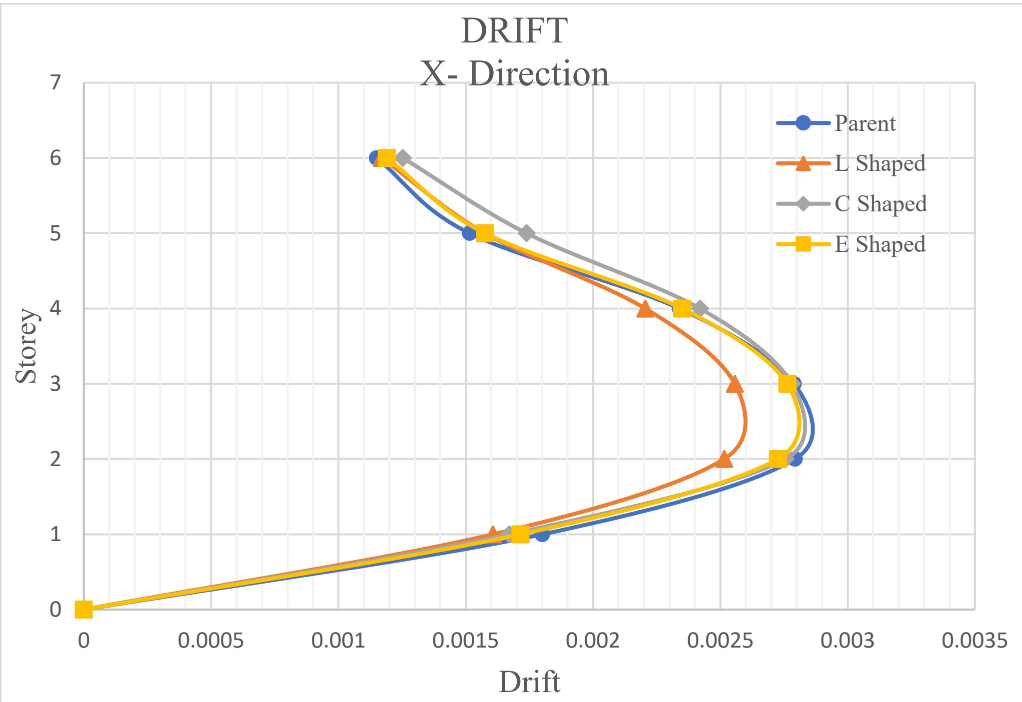

Fig 60: Drift Comparison graph in X Direction

Result: The drift is the maximum in regular building, then in C shaped building and E shaped and least in L shaped building by both NBC and IS in X direction.

Table 70: Drift Comparison in Y-Direction as per IS

Y-Direction				
Story	Regular	L Shaped	C Shaped	E Shaped
6	0.00119067	0.001402	0.001377	0.001153
5	0.001614	0.001859	0.001813	0.001518
4	0.00251733	0.002519	0.002529	0.002307
3	0.00303267	0.002913	0.002984	0.002747
2	0.00301767	0.002819	0.002929	0.002709
1	0.00185633	0.001679	0.001757	0.001644
Base	0	0	0	0

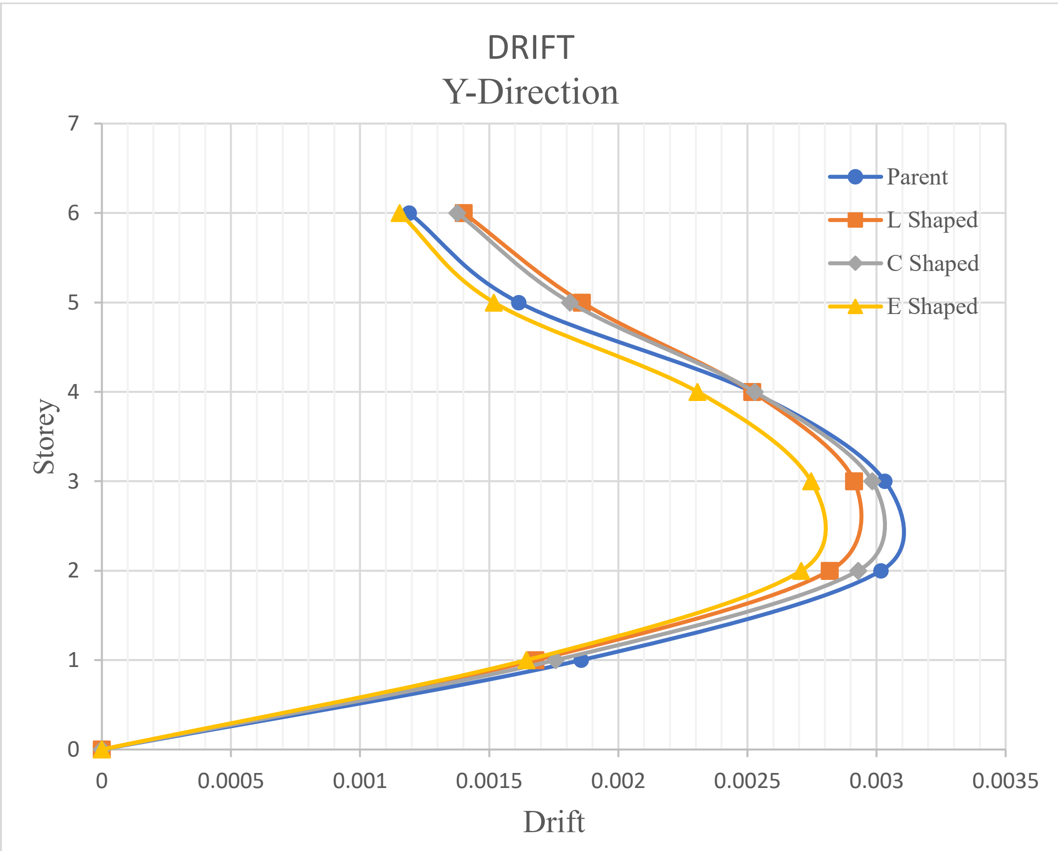

Fig 61: Drift Comparison graph in Y Direction

Table 71: Drift Comparison in Y-Direction as per NBC ULS

Y Direction				
Story	Regular ULS	L Shaped ULS	C Shaped ULS	E Shaped ULS
6	0.001122333	0.001251333	0.001332667	0.001056667
5	0.001621333	0.001697	0.001846333	0.00146
4	0.002687	0.002574333	0.002731667	0.00238
3	0.003445	0.003205333	0.003345667	0.00303
2	0.003621667	0.003305667	0.003415667	0.003163667
1	0.002307333	0.002038333	0.002139	0.001989667
0	0	0	0	0

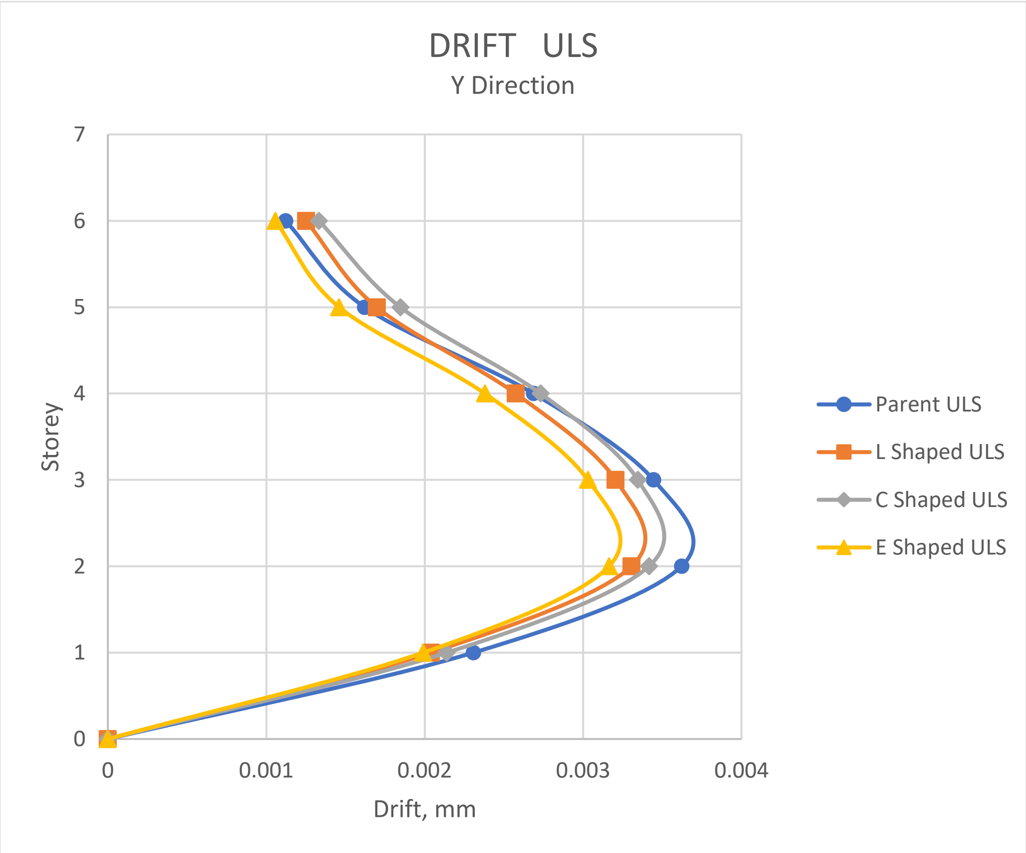

Fig 62: Drift Comparison Chart in Y-Direction as per NBC ULS

Table 72: Drift Comparison in Y-Direction as per NBC SLS

Y Direction				
Story	Regular SLS	L Shaped SLS	C Shaped SLS	E Shaped SLS
6	0.001072667	0.001196	0.001273667	0.001008667
5	0.001549667	0.001622	0.001764667	0.001396667
4	0.002568	0.002460333	0.002610667	0.002274667
3	0.003292667	0.003063333	0.003198	0.002896
2	0.003461667	0.003159333	0.003264333	0.003023667
1	0.002205	0.001948333	0.002044333	0.001901667
Base	0	0	0	0

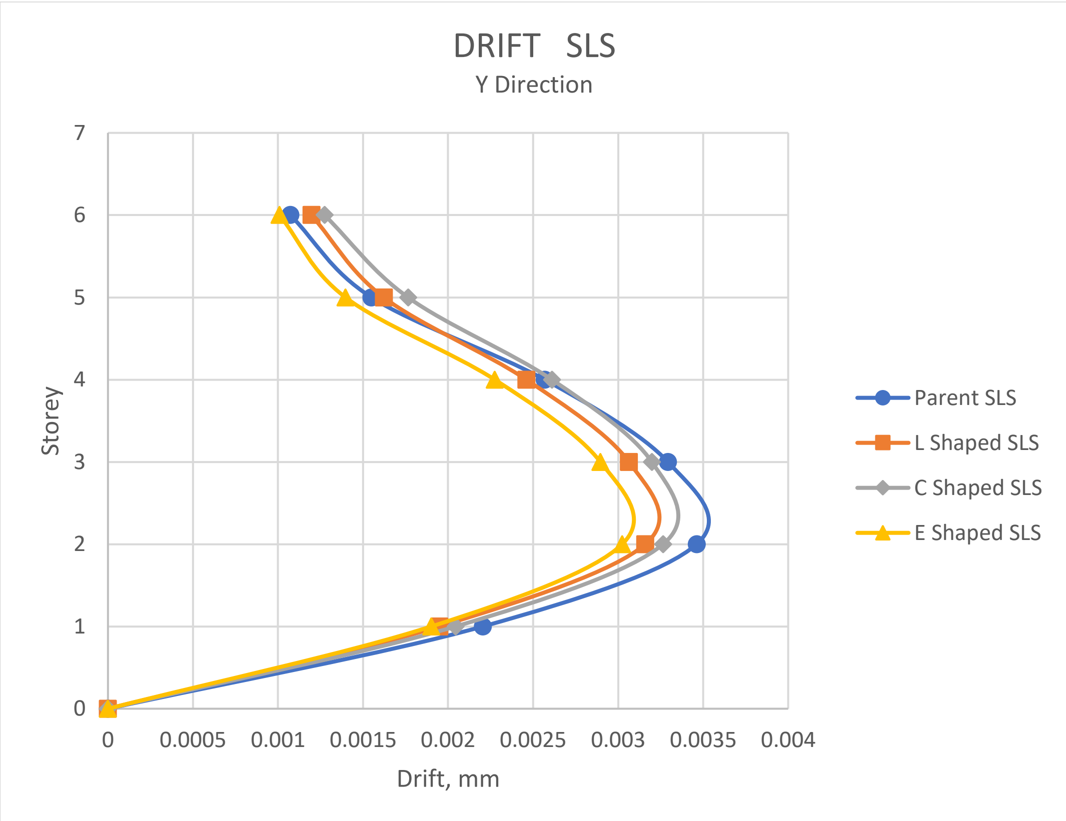

Fig 63: Drift Comparison Chart in Y-Direction as per NBC SLS

Result: The drift is the maximum in regular building, then in C shaped building and L shaped building and least in E shaped building by both NBC and IS in Y direction.

5.9.3 Torsion Comparison

According to NBC 105:2020

Table 73: Torsion Comparison in X direction

X-Direction				
	Regular	L Shaped	C Shaped	E Shaped
ULS	1.254	1.292	1.167	1.236
SLS	1.254	1.292	1.167	1.236

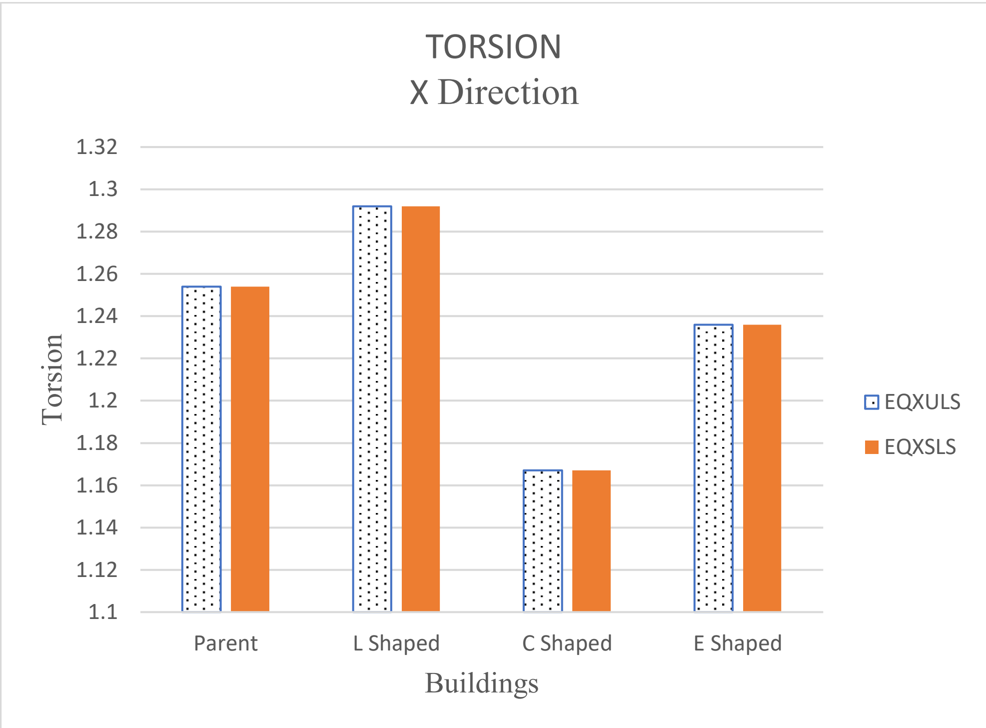

Fig 64: Torsion Comparison Graph in X direction

Result: The torsion is highest for C shaped building, then in E shaped building and the same for regular and E shaped building as per NBC in X direction.

Table 74: Torsion Comparison in Y direction

Y-Direction				
	Regular	L Shaped	C Shaped	E Shaped
ULS	1.384	1.609	1.104	1.445
SLS	1.384	1.609	1.104	1.445

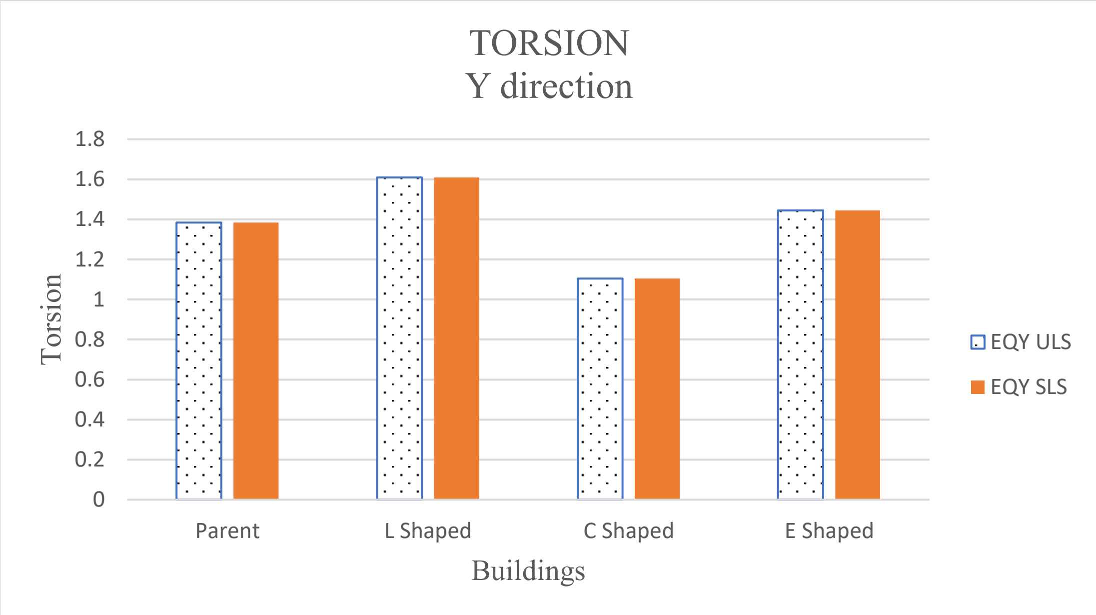

Fig 65: Torsion Comparison Graph in Y direction

Result: The torsion is the lowest for C shaped building, then almost the same for regular, L shaped and E shaped building as per NBC in Y direction as per NBC.

According to IS 1893:2016

Table 75: Torsion Comparison in X direction

X-Direction			
Regular	L Shaped	C Shaped	E Shaped
1.167	1.19	1.188	1.136

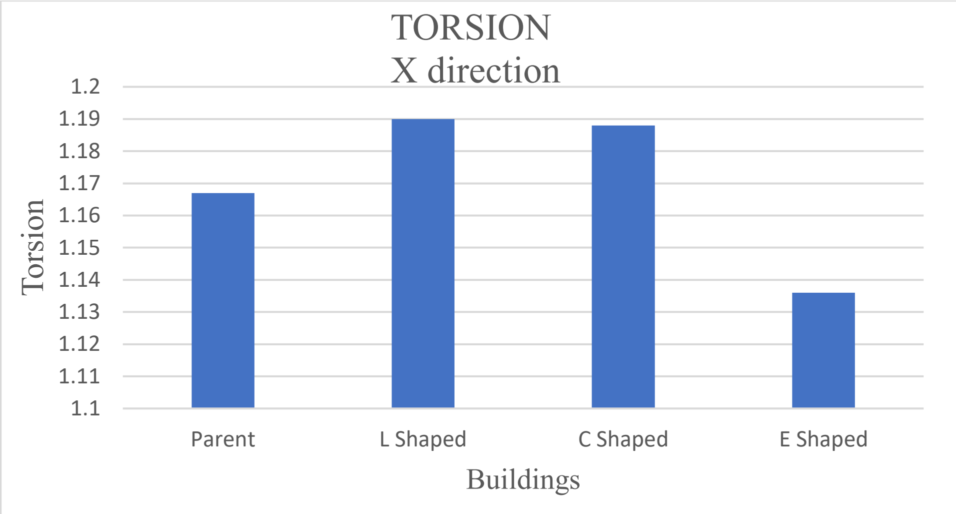

Fig 66: Torsion Comparison Graph in X direction

Table: Torsion Comparison in Y direction

Y-Direction			
Regular	L Shaped	C Shaped	E Shaped
1.217	1.17	1.057	1.096

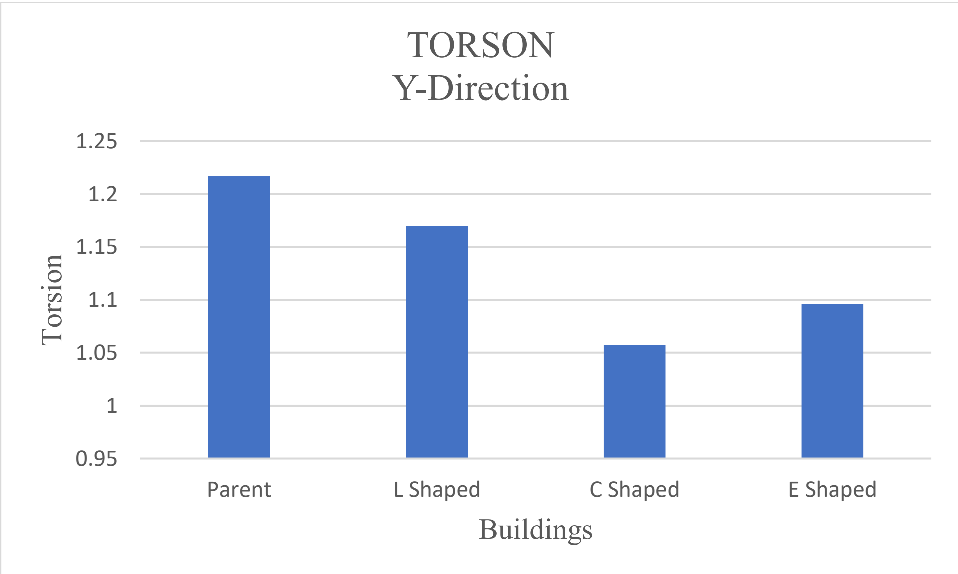

Fig 67: Torsion Comparison Graph in Y direction

Result: The torsion according to IS in X direction is higher in C shaped, then in L shaped, then regular and least in E shaped building.

In Y direction, the torsion is higher in E shaped, then in regular building, then L shaped building and the least in C shaped building.

CHAPTER 6: RESULTS AND DISCUSSION

Discussions:

This project compares the seismic performance of buildings using two codes: NBC 105:2020 and IS 1893:2016. Both codes have different rules for handling horizontal irregularities like L, U and C shaped buildings. These irregular shapes affect how buildings behave during earthquakes. The discussion focusses on how the results differ between two codes in terms of drift, displacement and torsion.

Detailed Comparison and Discussion:

The seismic responses of irregular buildings vary under NBC 105:2020 and IS 1893:2016 due to differences in key parameters such as seismic zoning factor, response reduction factor, importance factor, and base shear calculation method. These differences lead to variations in drift, torsion, and displacement, especially in irregular building shapes.

1. **Seismic Zoning and Design Acceleration:**
    - NBC 105:2020 uses Peak Ground Acceleration (PGA) based on Probabilistic Seismic Hazard Assessment (PSHA), which often gives higher seismic demand in high-risk zones.
    - IS 1893:2016 uses a zone factor (Z) based on historical data, which may lead to lower base shear in some areas compared to NBC.

Effect:
This difference leads to higher base shear, displacement, and drift in NBC-designed buildings.

2. **Response Reduction Factor (R):**
    - NBC assigns R = 4.0 for Special Moment Resisting Frames (SMRF) as ductility factor.
    - IS uses R = 5.0 for the same system.

Effect:

Increasing the Response Reduction Factor (R) reduces the design seismic force, assuming the structure can undergo larger inelastic deformations without collapse.

While R does not directly affect material stiffness or elasticity, a higher R often leads to lighter, more flexible structures with reduced stiffness due to smaller member sizes. It also increases the ductility demand, requiring detailed ductile reinforcement as per seismic codes. Conversely, a lower R increases design forces, leading to stronger and stiffer members, lower ductility demand, and less emphasis on ductile detailing. The elastic properties like Young's modulus remain unchanged, but the structural behaviour shifts between elastic and inelastic depending on R.

3. **Importance Factor (I):**
   - Both codes consider importance, but NBC often assigns a slightly higher I depending on building usage. For our building the I was 1 form both the codes.

Effect:

Higher importance factor increases design base shear and leads to higher demand on structural elements, increasing drift and displacement.

4. **Seismic Weight (W)**
   - NBC 105:2020 considers 30% of live load for seismic weight under the category "live load for other purposes," which applies to our building type.
   - IS 1893:2016 considers 25% of live load for seismic weight when the live load is less than or equal to 3 KN/m^2, which the case for our structure.

Effect:

The higher percentage of live load considered in NBC 105:2020 (30%) compared to IS 1893:2016 (25%) leads to a higher seismic weight when using NBC. Since base shear is directly proportional to seismic weight, this results in greater base shear forces in the NBC analysis. As a result, NBC may predict higher seismic demand, leading to safer structural design compared to IS for buildings with similar occupancy and live load conditions. This difference, though small (only 5%), can influence member sizing, reinforcement, and overall cost.

5. **Torsional Irregularity Criteria:**
   - NBC uses stricter limits for allowable torsion ratio (Dmax/Davg > 1.5) to define irregularity.
   - IS allows a slightly more relaxed criterion.

In the range 1.5-2.0, (a) the building configuration shall be be revised to ensure that the natural period of the fundamental torsional mode of oscillation shall be smaller than those of the first two translational modes along each of the principal plan directions, and then (b) three-dimensional dynamic analysis method shall be adopted and more than 2.0, the building configuration shall be revised.

Effect:

IS flags torsional irregularity more aggressively, while NBC gives a more realistic response based on true building geometry. So, it requires heavier design in IS; NBC allows more efficient design but only if torsion is within limits.

6. Drift Limits:

- NBC enforces inter storey limits more strictly in Performance-Based Design.

Effect:

NBC may result in larger member sizes or stiffer frames to control drift, while IS may allow more flexibility.

7. Eccentricity

- NBC considers a fixed accidental eccentricity of ±0.1b (10% of building width perpendicular to seismic force direction). It doesn't include or amplify the static eccentricity (actual offset between centre of mass and centre of rigidity).
- IS considers design eccentricity= 1.5× static eccentricity ±0.05b. it accounts for both actual torsional irregularity and accidental uncertainty.

Effect:

IS is more conservative, leading to higher torsional moments and safer design, especially for irregular buildings. NBC may underestimate torsion, which could lead to less safe design in buildings with significant asymmetry.

The reasons why these differences matter are stated as:

- Base Shear directly influences the total lateral force; higher base shear causes higher drift and displacement.
- Torsion becomes critical in irregular buildings where mass and stiffness are not symmetrically distributed; stricter torsion checks in NBC mean it catches irregular behaviour earlier.

- Drift affects structural safety and serviceability. NBC tends to produce higher drift values due to higher input forces, demanding better detailing.
- Code conservativeness: NBC is generally more conservative, leading to safer but possibly more expensive designs.

Discussions regarding irregularities:

Torsion is the key factor affecting the structure's stability and strength having irregularities. Because of the asymmetry of the irregular buildings, the difference in the centre of the mass and the centre of stiffness (eccentricity) of the building creates significant torsion.

From our research, it is found that;

Torsion is high in C-shaped and U-shaped buildings primarily due to asymmetry in mass and stiffness, which causes the building to twist (rotate) during seismic shaking. Here's why this happens:

1. **Asymmetric Geometry**
    - C and U shapes have open sides and uneven distribution of structural elements, like columns, shear walls, or beams.
    - This creates an offset between the center of mass (CM) and center of rigidity (CR).
    - When seismic forces act through the CM, but the structure resists them through the CR, a torsional moment is generated.
2. **Uneven Stiffness Distribution**
    - The arms of a C or U shape may be more flexible than the core or closed side.
    - This stiffness imbalance causes one part of the building to displace more than the other, increasing the rotational (torsional) response.
3. **Re-entrant Corners and Stress Concentration**
    - The "notched" parts of the building (re-entrant corners) interrupt the uniform flow of forces.
    - These corners behave like hinges or weak points under earthquake motion, increasing deformation and twisting in those regions.
4. **Lack of Symmetry in Both Axes**
    - U and C shapes are often not symmetric in either X or Y direction.

- This results in torsion in multiple directions, making the building vulnerable to complex displacement patterns.

The following table describes the above points.

Reason	Effect
Center of mass ≠ Center of rigidity	Torsional moments develop due to eccentricity
Uneven stiffness across plan	Unequal displacements cause twisting
Open or discontinuous floor plan	Disrupts load path and adds torsion
Re-entrant corners	Local stress concentration and distortion

Code Comparison on Irregular Shapes:

Provisions of NBC 105:2020:

- Defines re-entrant corners (>15%), torsional irregularity (max/min displacement >1.5), diaphragm discontinuity, and out-of-plane offsets in Clause 5.5.2
- Requires response spectrum analysis for any identified plan irregularities.
- Applies a fixed accidental eccentricity (±0.1b), but does not amplify based on severity.

Provisions of IS 1893:2016:

- Defines similar plan irregularities in Clause 7.1 & Table 5, including torsional irregularity (max/min drift >1.5), re-entrant corners (>15%) .
- Requires dynamic analysis for any significant plan irregularity.
- Amplifies torsion by using eccentricity = 1.5 × static + ±0.05b, and imposes displacement checks (e.g., drift ratio >1.2)

Recommendations for Safety:

C- and U-shaped layouts are the most critical in case of torsion. In these cases:

- IS 1893:2016 offers greater safety with its amplified eccentricity and strict checks making it the more robust code.

- NBC 105:2020, while simpler, may underestimate torsional effects unless designers thoroughly model the irregular geometry and consider additional safety margins.

Design Implication:

These shapes require special attention in dynamic analysis. Codes like IS 1893:2016 account for this with amplified eccentricity and drift ratio checks. NBC 105:2020 requires dynamic analysis but does not amplify torsion, so additional modeling care is essential.

Increasing eccentricity (e.g., from 0.1b to 0.15b or 1.5e as in IS 1893) results in:

- Greater torsional moment
- Higher lateral displacements at edges of the building
- More conservative design (safer)
- Potentially larger member sizes or reinforcements
- Decreasing eccentricity (e.g., using 0.05b or no eccentricity):
- Underestimates torsional effects
- May lead to unsafe design in torsionally irregular buildings

So, increasing or decreasing 10% eccentricity refers to adjusting the assumed offset between the seismic load point and the resisting centre. This directly impacts the torsional forces and deformation patterns of a structure.

Using ±0.1b is a way to simulate realistic irregular behaviours and ensure safety under torsion. More advanced codes like IS 1893:2016 go further by combining real eccentricity with accidental offsets and amplification, especially for irregular buildings.

Conclusion:

From our observation, it is seen that NBC provides more conservative and safer designs as it gives higher base shear but for structures having complex irregularities, IS is more conservative. Since NBC provides safer design, design from NBC seems uneconomical as comparing to IS.

CHAPTER 7: DETAILED DESIGN

7.1 Slab

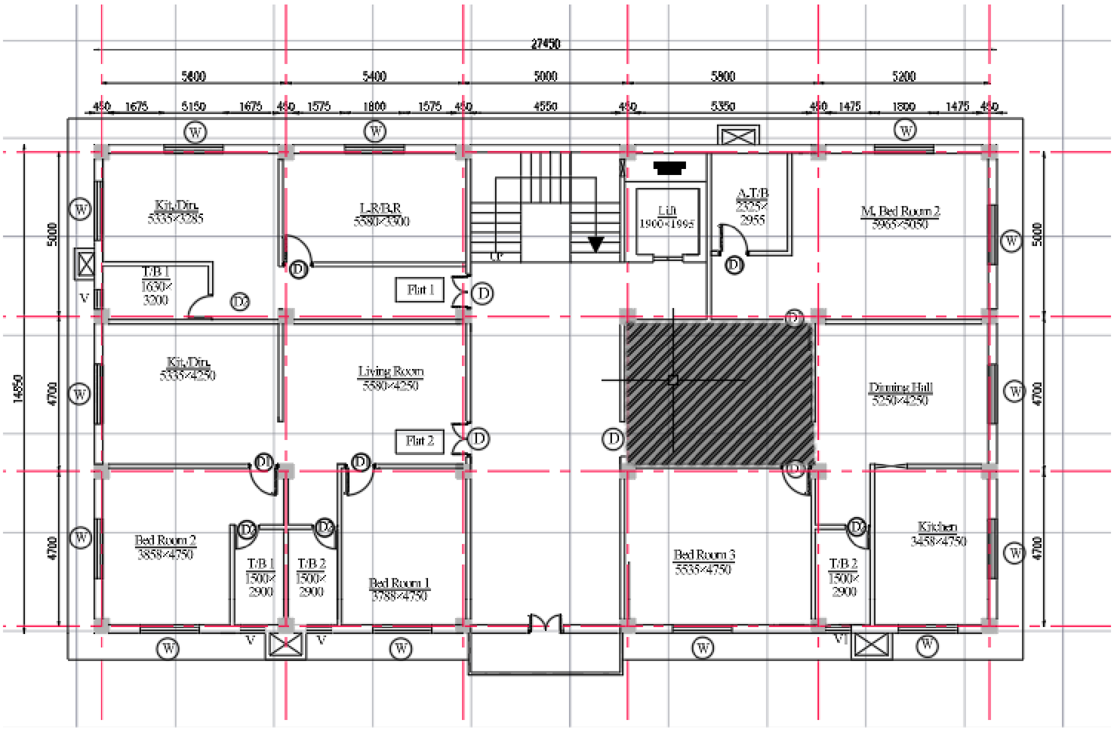

Fig 68: Designed slab (hatched) *Source: Autocad*

Table 77: Detailed Design of Slab

	CALCULATIONS	VALUES	UNITS	REMARKS	INFO.
	Beam size	**350×550**	**mm**		
	Width of Beam	350	mm		
	Depth of Beam	550	mm		
1	**Known Data**				
	Depth of Slab(d)	105	mm		
	Clear Cover	15	mm		
	Effective Cover	20	mm		
	Concrete Grade	25	N/mm^2		
	Steel Grade	500	N/mm^2		
	Clear Long Span	5.35	m		
	Clear Short Span	4.25	m		
	Overall Depth of Slab(D)	125	mm		

	Diameter of the Bar	10	mm		
	Effective Depth in Shorter span	95	mm		
	Effective Depth in Longer span	85	mm		
2	**Effective Span**				
	Ly = Lesser of (5.35+0.085) and (5.35+0.35)	5.435	m	Smaller Among Two	From CL. 22.2.a of Code IS 456:2000
	Lx = Lesser of (4.25+0.095) and (4.25+0.35)	4.345	m	Smaller Among Two	
	L_y/L_x =	1.25	Less than 2	**Hence it is Two Way Slab**	From Annex D-1.11 of IS 456:2000
3	**Load Calculation**				
	Dead Load = $\gamma_{RC} \times b \times D \times 1$	3.125	KN/m^2	γ_{RC} = 25 KN/m^2	
	Floor Finish	1.08	KN/m^2		
	Live Load	2	KN/m^2		
	Total Dead Load (W)	4.205	KN/m^2		
	Total Factored Load (W_u) = 1.2DL+1.5LL	8.046	KN/m^2		
4	**Moment Calculation**				
	αx	0.034		Positive Moment at Mid- Span	From Table 26 of Code IS 456:2000
	αx	0.045		Negative Moment at continous edge	
	αy	0.024		Positive Moment at Mid- Span	
	αy	0.032		Negative Moment at continous edge	
	$M_x^+ = \alpha_x^+ \times w \times L_x^2$	5.1646	KNm	Moment Coefficents are Obtained, case as	
	$M_x^- = \alpha_x^- \times w \times L_x^2$	6.8355	KNm		

	$M_y^+ = \alpha_y^+ \times w \times L_x^2$	3.6456	KNm		
	$M_y^- = \alpha_y^- \times w \times L_x^2$	4.8608	KNm		
	M_{max}	6.8355	KNm		
5	**Check For Balancing Depth**				
	For fe 500				
	$M_{max} =$	$k \times fck \times b \times d^2$		k=0.133,for Fe500	
	$d_{req} =$	76.639965	mm	Less than 95 mm.	
				Hence OK.	
6	**Reinforcement Calculation**				
	$M_{max} = 0.87f_y \times A_{st} \times d \times 1-(f_y \times A_{st})/f_{ck} \times b \times d)$				
	For Shorter Span Direction:				
	Positive Reinforcement Calculation:				
	$M_x^+ =$	5.1646	KN/m		
	$A_{st,x}^+ =$	134.128	mm^2		
$A_{st,x}^+$ should be greater than $A_{st,\ min}$ and less than $A_{st,\ max}$.					
	$A_{st,\ min}$ = 0.12% of b × D	150	mm^2	Hence not ok, so adopt Ast=174mm^2	
	Area of single bar, A_{st1}	78.5398	mm^2	Provide 10mm dia bars	
	$Spacing_x^+ = (A_{st1}) \times b / A_{st,x}^+$	523.5987	mm		From Clause 26.3.3 of Code IS 456:2000
	Spacing should not be greater than:	300	mm	Hence Not OK.	
		3d	mm	Hence Not OK.	
	Hence, Provide $Spacing_x^+ =$	300	mm		

	Then, $A_{st}^{+}{}_{provided}$ =	261.7993	mm^2	Hence provide 10mm bars @300mm spacing.	
	Negative Reinforcement Calculation:				
	M_x^- =	6.8355	KN/m		
	$A_{st,x}^-$ =	182.32	mm^2		
$A_{st,x}^-$ should be greater than $A_{st,\ min}$ and less than $A_{st,\ max}$.					
	$A_{st,\ min}$ = 0.12% of b × D	150	mm^2	Hence OK.	
	Area of single steel, A_{st1}	78.5398	mm^2		
	$Spacing_x^-=(A_{st1})\times b/A_{st,x}^-$	430.7799	mm		
	Spacing should not be greater than:	300	mm	Hence Not OK.	
		3d	mm	Hence Not OK.	From Clause 26.3.3 of Code IS 456:2000
	Hence, Provide $Spacing_x^-$ =	300	mm		
	Then, $A_{st}^{-}{}_{provided}$ =	261.7993	mm^2	Hence provide 10mm bars @300mm spacing.	
	For Longer Span Direction:				
	Positive Reinforcement Calculation:				
	M_y^+ =	3.6456	KN/m		
	$A_{st,y}^+$ =	92.57	mm^2		
$A_{st,y}^+$ should be greater than $A_{st,\ min}$ and less than $A_{st,\ max}$.					
	$A_{st,\ min}$ = 0.12% of b×D	150	mm^2	Hence not ok, so adopt Ast=174mm^2	
	Area of single steel, A_{st1}	78.5398	mm^2		
	$Spacing_y^+=(A_{st1})\times b/A_{st,y}^+$	523.5987	mm		

	Spacing should not be greater than:	300	mm	Hence Not OK.	
		3d	mm	Hence Not OK.	From Clause 26.3.3 of Code IS 456:2000
	Hence, Provide $Spacing_y^+$ =	300	mm		
	Then, $A_{st}^{+}{}_{provided}$ =	261.7993	mm^2	Hence provide 10mm bars @300mm spacing.	
	Negative Reinforcement Calculation:				
	M_y^- =	4.8608	KN/m		
	$A_{st,y}^-$ =	125.65	mm^2		
$A_{st,y}^-$ should be greater than $A_{st,\ min}$ and less than $A_{st,\ max}$.					
	$A_{st,\ min}$ = 0.12% of b×D	150	mm^2	Hence Not OK.	
	Area of single steel, A_{st1}	78.5398	mm^2		
	$Spacing_y^-=(A_{st1})\times b/A_{st,y}^-$	523.5987	mm		
	Spacing should not be greater than:	300	mm	Hence Not OK.	
		3d	mm	Hence Not OK.	From Clause 26.3.3 of Code IS 456:2000
	Hence, Provide $Spacing_y^-$ =	300	mm		
	Then, $A_{st}^{-}{}_{provided}$ =	261.7993	mm^2	Hence provide 10mm bars @300mm spacing.	
	Hence Provide mm bar @ 300mm Spacing at Mid Span of X-Axis.				
	Hence Provide mm bar @300 mm Spacing at Support of X-Axis.				
	Hence Provide mm bar @ 300 mm Spacing at Mid Span of Y-Axis.				
	Hence Provide mm bar @ 300 mm Spacing at Support of Y-Axis.				
7	**Check For Shear**				

	Maximum Shear Force (Vmax)= Wu ×(Lx/2)	17.479935	KN		
	Shear Stress(τ_{max}) = Vmax /(b×d)	0.1839993	N/mm^2		
	For τ_c				
	Pt = 100Ast/b×d	0.2755782			
	Hence,τ_c=	0.373	N/mm^2	from interpolation	Table 19, Clause 40.4 of code 456:2000
	k=	1.3			Clause 40.2.1.1of code 465:2020
	k×τ_c=	0.4849	N/mm^2		
	Maximum Shear Stress,$\tau_{c(max)}$=	3.1	N/mm^2		Table 20, clause 40.4
	Hence, τ_c<k×τ_c<$\tau_{c(max)}$			**Safe in Shear. OK**	
8	**Check For Deflection**				
	α	26			From Clause 26.2.1 of Code IS 456:2000
	β	1		Since L_x<10m.	
	λ	1			
	δ	1			
	Now, For γ:				From Figure 4 of Code IS 456:2000
	(A_{st}/bd) × 100	0.2756	%		
	F_s =0.58×f_y ×($A_{st,req}$ /$A_{st,prov}$)	201.9593	N/mm^2		
	Modification Factor, (γ)	1.9031		From, Figure 4	
	αβγδλ=	49.4806			

	$(L/d)_{provided}$	45.7368			From Clause 23.2.1 of Code IS 456:2000
	For Deflection it must Satisfy this experssion: $(L/d)_{provided} < \alpha\beta\lambda\delta\gamma$				
	Pass in deflection				
9	**Check for the Development Length**				
	Design bond stress, τ_{bd}	1.4	N/mm^2		From Clause 26.2.1.1 of Code IS 456:2000
	$L_d=0.87f_y\varphi/(4\tau_{bd})$	776.78571	mm		From Clause 26.2 of Code IS 456:2000
	$M_1 = 0.87f_y \times A_{st} \times d(1 - (f_y \times A_{st})/(f_{ck} \times b \times d)$	10.22	KNm.		
	Take L_o =largest of $(d,12\varphi)$	120	mm	L_o = Anchor Length	
	$1.3(M_1/v)+L_o$ =	900.16	mm	1.3(M1/v)+Lo > Ld	From Clause 26.2.3.3 of Code IS 456:2000
	Hence, Development Length is OK.				

7.2 BEAM

Table 78: Detailed Design of Flanged Beam

DESIGN OF FLANGED BEAM					
Beam ID		**3EF**			
	CALCULATIONS	**VALUES**	**UNITS**	**REMARKS**	**INFO.**
Beam Size		**350 × 550**	**mm**		
1	**Known Data**				
	Depth of Beam (D) =	550	mm		

	Width of Beam (b_w) =	350	mm	Which is greater than 200 mm, Hence Ok.	From clause 4.1.1 (b) of NBC 105:2020
	Thickness of Flange (D_f) =	125	mm		
	Clear Span of Beam (L) =	5200	mm		
	Grade of Concrete (f_{ck}) =	25	N/mm^2		
	Grade of Steel (f_y) =	500	N/mm^2		
	Effective Cover(d') =	50	mm		
	Effective Depth (d) =	500	mm		
2	**Effective Width of the flange beam**				
	Effective Length (l_{eff}) =	5200	mm		From clause 23.1.2 (a) of IS code 456:2000
	$D = l_{eff}/12$ =	433.33333	mm	Is less than 550 Hence OK.	
	Distance between two point of contra flexure (l_o) = $0.7 \times l_{eff}$	3640	mm	For Continuous Beam	From clause 23.1.2 (a) of IS code 456:2000
	$b_f = (L_o/6) + b_w + 6D_f$	1706.6667	mm	i.e. For T-Beam	
3	**Check for Axial Stress**				
	Facotred Axial Stress	0	KN/m^2		From clause 4.1 of NBC 105:2020
	$0.08 \times f_{ck}$ =	2		Which is greater than Axial Stress	
	Hence Design as a Flexure Member.				
4	**Check for Size in Beam**				
	B/D =	0.6363636		Which is greater than 0.3, Hence OK.	From clause 4.1.1 (a) of NBC 105:2020
	L/D =	9.4545455		Which is greater than 4, Hence OK.	From clause 4.1.1 (c) of NBC 105:2020
	Check for the Limiting Longitudinal Reinforcement				
	Minimum Reinforcement, = $0.24\times (f_{ck}^{0.5}) / f_y$	0.0024	%		From clause 4.1.1

	$A_{st,min}$ =	420	mm^2		(b) of NBC 105:2020
	Maximmum Reinforcement = 0.025 ×b × d	4375	mm^2		From clause 4.1.1 (c) of NBC 105:2020
	For Left Support at Hogging				
6	Moment ($M_{u,Hogging,left}$)	231.2855	KNm	From	ETABS
	Assuming Depth of Neutral Axis (x_u)				
	$M_u = 0.87f_y \times A_{st} \times d \times (1-(f_yA_{st})/f_{ck}b_f \times d)$				From ANNEX G 1.1 of IS code 456:2000
	$A_{st, top}$ =	1091.8542	mm^2	Ast,max>Ast ,top>Ast,min , Hence OK.	
	$A_{st, top,required}$ =	1091.8542	mm^2		
	Depth of Neutral Axis (x_u) = (0.87 × f_y × A_{st})/(0.36 × f_{ck} × b_f	30.9217	mm	Less than Depth of Flange	
	Hence, our assumption that Neutral Axis lies in Flange is true.				
	Calculation of the limiting value of depth of neutral axis ($x_{u,max}$)				
	(xu,max/d)=0.46	230	mm	For Fe 500	
	Comparison between x_u and $x_{u,max}$				
	Since xu (i.e.30.9216525173798 mm) < xu,max(230 mm)				
	Hence, The Section is under reinforced - section. Therefore, design Singly reinforced beam.				
	Hence ,Area for Compression reinforcement (Asc) is not required at bottom.				
	But Asc must be at least 50 % of Ast, hence				From clause 4.1.2 (d) of NBC 105:2020
	Asc= 50% of Ast	545.92711	mm^2		
	For Mid Span at Sagging				
	Moment ($M_{u,sagging, mid}$)	102.8596	KNm	From Sap	
	Assuming Depth of Neutral Axis (x_u)				
	$M_u = 0.87f_y \times A_{st} \times d \times (1-(f_yA_{st})/f_{ck}b_f \times d)$				From ANNEX G 1.1 of IS code 456:2000
	$A_{st, bottom}$ =	478.52093	mm^2	Ast,max>Ast ,top>Ast,min , Hence OK.	
	$A_{st, bottom, required}$ =	478.52093	mm^2		

	Depth of Neutral Axis (x_u) = (0.87 × f_y × A_{st})/(0.36 × f_{ck} × b_f	13.5519	mm	Less than Depth of Flange	
	Hence, our assumption that Neutral Axis lies in Flange is true.				
	Calculation of the limiting value of depth of neutral axis ($x_{u,max}$)				
	(xu,max/d)=0.46	230	mm	For Fe 500	
	Comparison between x_u and $x_{u,max}$				
	Since xu (i.e.13.551862339067 mm) < xu,max(230 mm)				
	Hence, The Section is under reinforced - section. Therefore, design Singly reinforced beam.				
	Hence, Area for Compression reinforcement (Asc) is not required at top.				
	But Asc must be at least 50 % of Ast, hence				From clause 4.1.2 (d) of NBC 105:2020
	Asc= 50% of Ast	239.26047	mm^2		
	For Right Support at Hogging				
	Moment ($M_{u,sagging, right}$)	237.5409	KNm	From Envelope Etabs	
	Assuming Depth of Neutral Axis (x_u)				
	M_u = 0.87f_y × A_{st} × d × (1-($f_y A_{st}$)/$f_{ck} b_f$ ×d)				From ANNEX G 1.1 of IS code 456:2000
	$A_{st, top}$ =	1122.204	mm^2	Ast,max>Ast,top>Ast,min, Hence OK.	
	$A_{st, top,required}$ =	1122.204	mm^2		
	Depth of Neutral Axis (x_u) = (0.87 × f_y × A_{st})/(0.36 × f_{ck} × b_f	31.7812	mm	Less than Depth of Flange	
	Hence, our assumption that Neutral Axis lies in Flange is true.				
	Calculation of the limiting value of depth of neutral axis ($x_{u,max}$)				
	(xu,max/d)=0.46	230	mm	For Fe 500	
	Comparison between x_u and $x_{u,max}$				
	Since xu (i.e.31.7812 mm) < xu,max(230 mm)				
	Hence, The Section is under reinforced - section. Therefore, design Singly reinforced beam.				
	Hence, Area for Compression reinforcement (Asc) is not required at bottom.				
	But Asc must be at least 50 % of Ast, hence				From Clause 4.1.2 (d) of NBC 105:2020
	Asc= 50% of Ast	561.10198	mm^2		

	For left Support at Sagging				
	Moment ($M_{u,sagging, left}$)	137.9928	KNm	From Envelope Etabs	
	Assuming Depth of Neutral Axis (x_u)				
	$M_u = 0.87f_y \times A_{st} \times d \times (1-(f_yA_{st})/f_{ck}b_f \times d)$				From ANNEX G 1.1 of IS code 456:2000
	$A_{st, bottom} =$	644.50243	mm^2	Ast,max>Ast ,top>Ast,min , Hence OK.	
	$A_{st, bottom, required} =$	644.50243	mm^2		
	Depth of Neutral Axis (x_u) = $(0.87 \times f_y \times A_{st})/(0.36 \times f_{ck} \times b_f$	18.2525	mm	Less than Depth of Flange	
	Hence, our assumption that Neutral Axis lies in Flange is true.				
	Calculation of the limiting value of depth of neutral axis ($x_{u,max}$)				
	(xu,max/d)=0.46	230	mm	For Fe 500	
	Comparison between x_u and $x_{u,max}$				
	Since xu (i.e.18.2525101668494 mm) < xu,max(230 mm)				
	Hence, The Section is under reinforced - section. Therefore, design Singly reinforced beam.				
	Hence, Area for Compression reinforcement (Asc) is not required at top.				
	But Asc must be at least 50 % of Ast, hence				From clause 4.1.2 (d) of NBC 105:2020
	Asc= 50% of Ast	322.25121	mm^2		
	For Right Support at Sagging				
	Moment ($M_{u,sagging, right}$)	159.8001	KNm	From Envelpe Etabs	
	Assuming Depth of Neutral Axis (x_u)				
	$M_u = 0.87f_y \times A_{st} \times d \times (1-(f_yA_{st})/f_{ck}b_f \times d)$				From ANNEX G 1.1 of IS code 456:2000
	$A_{st, bottom} =$	748.20088	mm^2	Ast,max>Ast ,top>Ast,min , Hence OK.	
	$A_{st, bottom, required} =$	748.20088	mm^2		
	Depth of Neutral Axis (x_u) = $(0.87 \times f_y \times A_{st})/(0.36 \times f_{ck} \times b_f$	21.1893	mm	Less than Depth of Flange	
	Hence, our assumption that Neutral Axis lies in Flange is true.				
	Calculation of the limiting value of depth of neutral axis ($x_{u,max}$)				
	(xu,max/d)=0.46	230	mm	For Fe 500	
	Comparison between x_u and $x_{u,max}$				

	Since xu (i.e.21.1892827138528 mm) < xu,max(230 mm)				
	Hence, The Section is under reinforced - section. Therefore, design Singly reinforced beam.				
	Hence, Area for Compression reinforcement (Asc) is not required at top.				
	But Asc must be at least 50 % of Ast, hence				From clause 4.1.2 (d) of NBC 105:2020
	Asc= 50% of Ast	374.10044	mm^2		
	For Hogging at top				
		Left	Mid	Right	
	M_u (KN) =	231.2855	0	237.5409	
	At Top, A_{st} (mm^2) =	1091.8542	0	1122.203962	
	At Bottom A_{sc} (mm^2) =	545.92711	0	561.1019809	
	For Sagging at mid				
		Left	Mid	Right	
	M_u (KN)=	0	102.9	0	
	At Bottom, A_{st} (mm^2) =	0	478.5	0	
	At Top A_{sc} (mm^2) =	0	239.3	0	
	For Sagging at bottom				
		Left	Mid	Right	
	M_u (KN)=	137.9928	0	159.8001	
	At Bottom, A_{st} (mm^2) =	644.50243	0	748.2008793	
	At Top A_{sc} (mm^2) =	322.25121	0	374.1004396	
	Reinforcement due to Flexure				
	Top	Left	Mid	Right	
	Diameter of Rebar (mm)	20	16	20	
	Ast of 1 bar (mm2)	314.15927	201.1	314.159265	
	Number of Rebars (nos)	4	3	4	
	Ast , Provided	1256.6371	603.2	1256.63706	
	Bottom	Left	Mid	Right	
	Diameter of Rebar (mm)	16	16	16	
	Ast of 1 bar (mm2)	201.06193	201.1	201.0619296	
	Number of Rebars (nos)	4	3	4	
	Ast , Provided	804.24772	603.2	804.2477184	
	Design for Shear				
	For Left End				

	Tensile steel provided at left end = (100 × Ast)/(b×d)	0.7180783			From Tabe 19 of IS code 456:2000
	τ_c =	0.560	N/mm^2		
	Design of shear strength of concrete = $\tau_c \times b \times d$	97962.386	N		
		97.962386	KN		
	For Mid Span				
	Tensile steel provided at left end = (100 × Ast)/(b×d)	0.3446776			From Tabe 19 of IS code 456:2000
	τ_c =	0.4092323	N/mm^2		
	Design of shear strength of concrete = $\tau_c \times b \times d$	71615.661	N		
		71.615661	KN		
	For Right End				
	Tensile steel provided at left end = (100 × Ast)/(b×d)	0.7180783			From Tabe 19 of IS code 456:2000
	τ_c =	0.4092323	N/mm^2		
	Design of shear strength of concrete = $\tau_c \times b \times d$	71615.661	N		
		71.615661	KN		
	Check for Shear Force				From clause 4.1.3 (e) of NBC 105:2020
	Shear Force ($V_{u,a}$) =	194.3025	KN	Envelope	From Etabs
	Shear Force ($V_{u,b}$) =	195.4898	KN	Envelope	From Etabs
	Design Shear Force At A=	194.3025	KN		
	Design Shear Force At B=	195.4898	KN		
	Design for stirrups				
	For the left end				
	Design for shear Force (V_u)	195.4898	KN	Maximum of Shear Froce	

	Capacity of the concrete (V_{uc})	97.962386	KN		
	Required capacity of the stirrup at the left end $V_{us} = V_u - V_{uc}$	97.527414	KN		
	Since V_{us} is greater than stirrup is necessary.				
	So, Provide stirrup of,				
	Diameter of stirrups =	8	mm		
	No of Stirrups =	2	No.s		
	A_{sv} =	100.53096	mm^2		
	For Spacing,				
	$S_v = (0.87 \times f_y \times A_{sv} \times d)/V_{us}$	224.19835	mm		
	Maximum Spacing	312.36407	mm		From Clause 26.5.1.6 of IS 456:2000
		375	mm		From Clause 26.5.1.5 of IS 456:2000
	Provide spacing =	200	mm		
	Therefore, provide 10 mm ,2 Legged stirrups @ 200mm c/c spacing				
	Check For Deflection				
	α	26		For Continuous Beam	From Clause 26.2.1 of Code IS 456:2000
	β	1		Since L_x<10m.	
	λ	0.8		For Flenged Beam	
	For δ			For Compression	
	(A_{sc}/bd) × 100%	0.4595701	%	Beam	
	δ =	1.15		From, Figure 5	
	Now, For γ:			For Tension	
	(A_{st}/bd)	0.7180783	%		

	$F_s = 0.58\times f_y (A_{st,req} / A_{st,prov})$	251.97229	N/mm^2		
	Modification Factor, (γ)	1.1		From, Figure 4	
	$(L/d)_{provided}$	9.4545455			
	$\alpha\beta\lambda\delta\gamma$	26.312			
	For Deflection it must Satisfy this experssion: $(L/d)_{provided} < \alpha\beta\lambda\delta\gamma$				
	Deflection Control is satisfactory.				
10	**Check For the Development Length**				
	$L_d = 0.87 f_y \varphi/(4\tau_{bd})$	970.98214	mm		From Clause 26.2 of Code IS 456:2000
	$M_1 = 0.87\times f_y \times A_{st} \times d (1 - (f_y A_{st})/(f_{ck}\times b\times d)$	234.0657	KNm.		From ANNEX G - 1.1 b) of Code IS 456:2000
	Take L_o = largest(d,12φ)	500	mm	L_o = Anchor Length	
	$1.3(M_1/v)+L_o$ =	2216.0396	mm	1.3(M1/v) +Lo > Ld	From Clause 26.2.3.3 of Code IS 456:2000
	Development Length is OK.				

7.3 Column

Table 79: Detailed Design of Column`

COLUMN ID: B3

	CALCULATIONS	VALUES	UNIT	REMARKS	INFO
	Column B3 Size	500×500	**mm**		
	depth of beam	550	mm		
1	**Known Data**				
	Width along x - direction = (b)	500	mm		
	Width along y - direction = (d)	500	mm		
	Clear Cover	40	mm		
	Length	3000	mm		
	Unsupported length of column(L)	2450			

	Grade of Concrete (fck)	25	MPA		
	Grade of Steel (fy)	500	MPA		
	Percentage of Steel (Pt)	2.5	%		
	Effective Width of Column along x – direction				
	Effective Width of Column along y – direction				
	Axial Force (Pu)	1802.6	KN		
	Moment about x - Direction	408.9	KNm		
	Moment about y - Direction	38.87	KNm		
2	**Check For Axial stress**				
	Pu/b×d	7.2104	N/mm^2	>0.08 × fck=2	From Clause 4.2, NBC 105:2020
	Hence, it is designed as Compression Member.				
3	**Calculation of Effective Length**				
	Effective Length (Leff)	1592.5	mm	Recommended value of effective length.	Table 28 of code IS 456:2000
4	**Check for Slenderness Ratio**				
	Slenderness Ratio For x - axis, $\lambda x = \frac{Leff}{b}$	3.185		Is less than 12.	Clause 25.1.2 of IS 456:2000
	Slenderness Ratio For y - axis, $\lambda y = \frac{Leff}{D}$	3.185		Is less than 12.	Clause 25.1.2 of IS 456:2000
	Hence, it is a Short Column.				
5	**Check for Eccentricity**				
	Minimum Eccentricity, emin,x = (L/500)+(b/30)	21.566667		Is Greater than 20 mm.	From Clause 25.4 of IS 456:2020

	Minimum Eccentricity, emin,y = (L/500)+(D/30)	21.566667		Is Greater than 20 mm.	From Clause 25.4 of IS 456:2020
	Permissible Eccentricity along x-direction = 0.05 × b	25		Greater than e,min,x, Hence OK.	From Clause 39.3 of IS 456:2000
	Permissible Eccentricity along y-direction = 0.05 ×D	25		Greater than e,min,y, Hence OK.	
6	**Calculation of Moment Due to Eccentricity**				
	Mx,emin = Pu × emin,x	38.876073	KNm	Mux greater than Mx,emin.	
	My,emin = Pu × emin,y	38.876073	KNm	Muy greater than My,emin	
	Mux	408.9	KNm		
	Muy	38.87	KNm		
7	**Calculation of Moment Capacity**				
	$((M_{ux}/M_{ux1})^{\alpha n}+(M_{ux}/M_{ux1})^{\alpha n}) \leq 1$			From Clause 39.6 of IS456:2000	
	Moment Capacity along Larger Side:				
	Assume, Reinforcement to be distributed equally on two sides			From Chart 37 of IS 456:2000	
	$\frac{d'}{D}$	0.15		assuming	
	p/fck	0.1		p= 2.19	
	Pu/fck×b×d	0.288416			
	Now,				
	Mux1/fck×b×D2	0.15			
	Mux1	468750000	N/mm		
		468.75	KNm		

	Moment Capacity along Shorter Side:				
	Assume, Reinforcement to be distributed equally on four sides			From Chart 37 of IS 456:2000	
	$\frac{d'}{D}$	0.15		assuming	
	p/fck	0.1		p= 2.19	
	Pu/fck×b×d	0.288416			
	Now,				
	Muy1/fck×b×D^2	0.15			
	Mux1	468750000	N/m		
		468.75	KNm		
	Puz = 0.45 × fck × Ac + 0.75× fy × Asc	4687500	N	The value of αn varies from 1 to 2 for the value of Pu/Puz = 0.2 to 0.8	From Clause 39.6 of IS 456:2000
		4687.5	KN		
	pu/puz	0.3845547			
	αn =	1.3		on interpolation	
	Therefore,				
	$((M_{ux}/M_{ux1})^{\alpha n}+(M_{ux}/M_{ux1})^{\alpha n}) \leq 1$	0.8765845		As, it is less than 1, Hence OK.	
	The design is safe with 2.5 % steel.				
8	**Calculation and Check of Area and Number of Longitudinal Reinforcement:**				
	Area of Steel (As)= 2.5% of bD	6250	mm^2		
	Minimum Reinforcement = 0.8% of bD =	2000	mm^2	As greater than 2000. Hence OK	from clause 26.5.3.1.a of IS 456:2000

	Maximum Reinforcement = 4% of bD =	10000	mm^2	As less than 10000. Hence OK	from clause 26.5.3.1.a of IS 456:2000
Provide, 25 mm diameter longitudinal bars.					
	Area of each bar = $\pi \times \frac{\emptyset^2}{4}$	490.85938	mm^2		
	Numbers of Bars required =	12.732771	No.s		
	Number of Bars Provided =	14	No.s	Greater than 4.	From clause 26.5.3.1.c of IS 456:2000
	Area of Bar provided =	6518.6125	mm^2		
Between 1600 to 6400. Hence Ok.					
9	**Check for spacing of longitudinal bars**				
	Provide, 4 bars along each x and y - direction.				
	Along x-direction and Y-direction				
	S= $\frac{500-2*40-4*25}{3}$	106.67	mm	>75 mm. OK	From Clause 26.5.3.2.b IS 456:2000
10	**Design of transverse reinforcement**				
	Provide, 8 mm diameter lateral ties.				
	Diameter of Lateral ties= Greater (6mm or $\frac{d}{4}$ mm)	10	mm	>6 mm or 20/4=5 mm	From clause 26.5.3.2.c.2 of IS 456:2000
	Spacing between Bars = minimum (b, 16d, 300)	300	mm	<300 mm	From clause 26.5.3.2

					.c.2 of IS 456:2000
11	**Check of Column According to NBC 105:2020**				
A	**Dimensional Limits**				
	Min^{m} Dimension of Column	500		>20db=400 or 300 mm	From Clause 4.2.1 of NBC 105:2020
	Width/Depth	1		>0.45(OK)	
B	**Longitudinal Reinforcement**				
	No of bars provided	14	No.s	>8(OK)	
	Min^{m} longitudinal steel	3.2593063	%	>1%, OK	From Clause 4.2.2 of NBC 105:2020
	Max^{m} longitudinal steel	2.607445	%	<4%, OK	
	Minimum diameter of bar	20	mm	>12 mm, OK	
C	**Transverse Reinforcement**				
	Spacing of longitudinal bars	103.3333	mm	<300 mm, cross tie not required	From Clause 4.2.3 of NBC 105:2020
	Hook length provided=	65	mm	>6 ´ 8=48 mm or 65 mm	
	Minimum diameter of hoop	8	mm	8 mm, OK	
	Max^{m} spacing of hoop	200	mm	475/2 = 237.5 mm, OK	

7.4 STAIRCASE

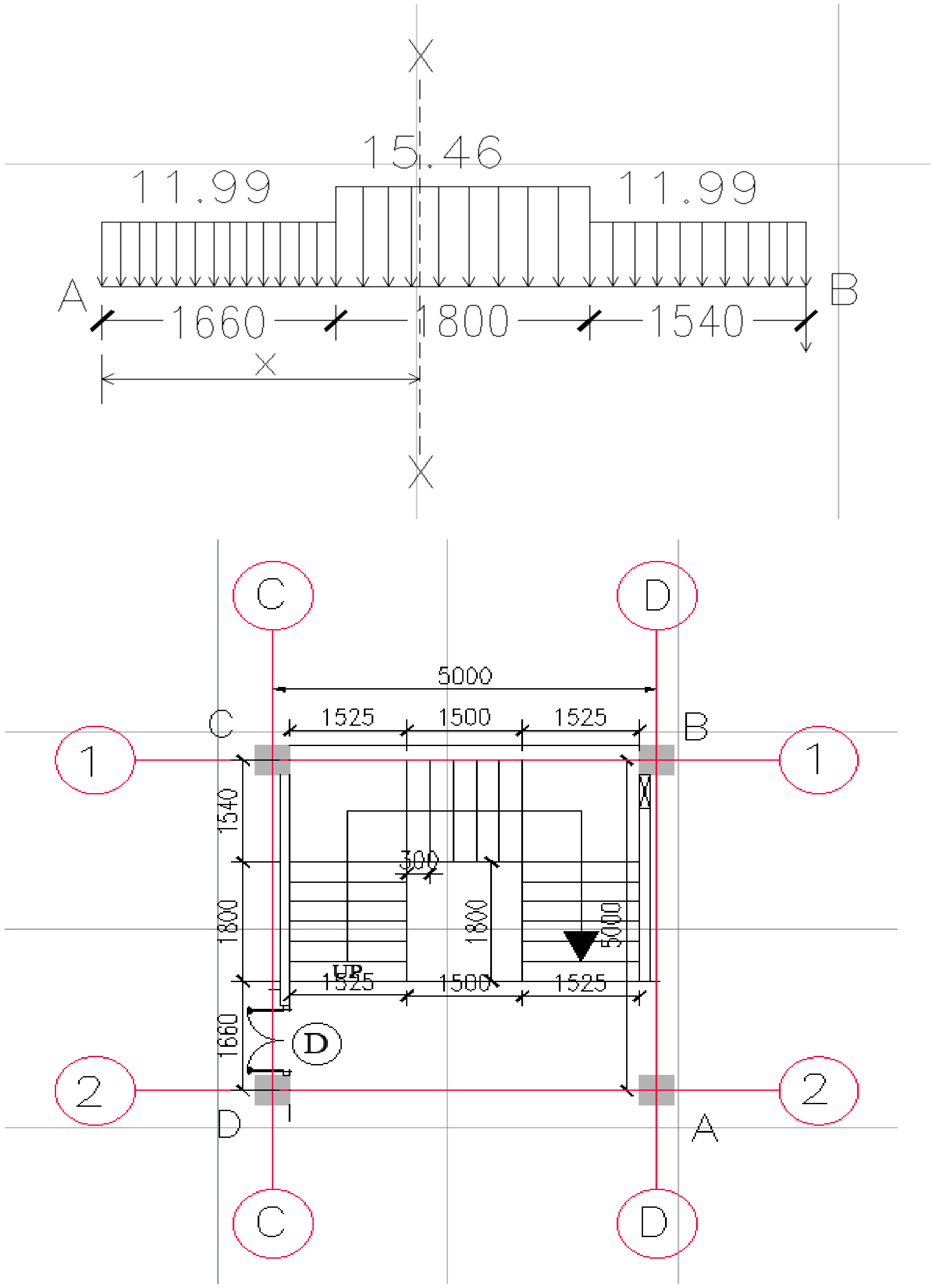

Fig 69: Staircase section *Source: Autocad*

Table 80: Detailed design of Third quarter turn staircase

Notation	Value	Unit	Description	Remarks

l_{eff}	5000.00	Mm	effective span	
R	150.00	Mm	Riser (maximum 190)	From NBC 206:2024 Table 4
T	300.00	Mm	Thread (minimum 250)	From NBC 206:2024 Table 4
D	200.00	Mm	total depth	
Cc	25.00	mm	Clear Cover	25 mm minimum for moderate exposure condition according to is 456:2000 Table 16
$\phi1$	16.00	Mm	diamerer of main reinforcement bar	
$\phi2$	10.00	Mm	diameter of distribution bar	
d	167.00	Mm	effective depth	
f_y	500.00	Mpa	grade of steel reinforcement	
f_{ck}	25.00	Mpa	grade of concrete	
Calculation of load				
	Loads on Landing		**Loads on Going**	
Self-Weight of waist slab = 25× d/1000	5.00	KN/ m^2	5.59	KN/m^2
Self-Weight of Step = 25× 0.5× R/1000	0.00	KN/ m^2	1.88	KN/m^2
Live Load	3.00	KN/ m^2	3.00	KN/m^2
Floor Finish	1.24	KN/ m^2	1.24	KN/m^2
Total Load	9.24	KN/ m^2	11.71	KN/m^2
Factored Load	13.86	KN/ m^2	17.56	KN/m^2
Landing 1st Width	1.66	M		
Going Width	1.80	M		

Landing 2nd width	1.54	M		
Total	5.00	M		
Tota Force ∑ F	75.89	KNm /m		
Taking moment at Support A				
∑ M_A	0.00			
R_B	38.03	KNm /m		
R_A	37.86	KNm /m		
Taking moment at refernce axix X-X at distance x from Support A				
$M_{X\text{-}X} = R_A \times$ x-11.99× 1.66× (x-1.66/2)-15.46× (x-1.66/2)(x-1.66/2)× 0.5				
Again differentiating this moment with respect to x and equating to zero to get maximum moment				
$\frac{d\,M_{X-X}}{d\,x} = R_A - 11.99\times 1.66 - 2\times 15.56\times (x-1.66)\times 0.5 = 0$				
$x = \frac{R_A - 11.99 * 1.66}{15.46} + 1.66$				
x	2.51	M		
M_{MAX}	50.03	KNm /m		
	50033356.32	Nm/m		
Check For depth				
$d_{required} = \sqrt{\frac{M_{MAX} * 10^6}{k\, f_{ck} b}}$	122.67	Mm	which is less than d provided ie 167mm ,OK!!	
Calculation of area of steel Ast				
(A_{st})required	735.98	mm^2		
Calculation of minimum reinforcement				
(A_{st})min=0.12%bD	240.00	mm^2	which is less than Ast req, OK!	
Calculation of Main Bars				
Provide Diameter of main bar	16.00	Mm		
Spacing (s)=$\frac{1000 * \frac{\pi}{4} * \phi^2}{(A_{st})required}$	273.19	Mm		

Provide (s) =	200.00	Mm	which is less than 300 mm and 3d=375 mm, hence ok.	
(A_{st})provided	1005.31	mm^2		from clause 26.3.3 b) 1) of IS456:2000
Calculation of Distribution bar				
Provide diameter of Distribution bar	10.00	Mm		
Spacing (s)=$\frac{1000*\frac{\pi}{4}*\phi^2}{(A_{st})min}$	327.25	Mm		
Provide (s) =	300.00	Mm	which is less than 450mm and 5d = 625mm, hence ok.	
(A_{st})provided	261.80	mm^2		from clause 26.3.3 b) 2) of IS456:2000
Provide Main Bar of 16 mm diameter @ 200 mm c/c spacing				
Provide Distribution Bar of 10 mm diameter @ 300 mm c/c spacing				
Check For Shear				
Shear Force (V_{max}) = Max(R_A, R_B)	38.03	KN		
$Shear\ Stress(\tau_c)=\frac{V_{max}}{b\ d}$	0.23	N/mm²		
$P_t = \frac{100\ *\ A_{st}}{b\ d}$				
Percentage of Steel (P_t)	0.60	%		
Shear Strenght of Concrete	0.49	N/mm²	which is than τ_c , then ok	Table 19 IS 456:2000
Check for Deflection				
α	26		For Simply Supported	From Clause 26.2.1 of Code IS 456:2000
β	1		Since l_{eff}<10m.	
λ	1		For no Flanged Beam	

δ	1		For no compression	
Now, For γ:			For Tension	
$P_t = \frac{100 * A_{st}}{b\ d}$	0.503	%		
$F_s = 0.58 \times f_y \times (A_{st,req} / A_{st,prov})$	212.3058235	N/mm^2		
Modification Factor, (γ)	1.3		From Figure 4	
$(L/d)_{provided}$	29.94011976			
$\alpha\beta\lambda\delta\gamma$	33.8			
For Deflection it must Satisfy this expression: $(L/d)_{provided} < \alpha\beta\lambda\delta\gamma$				
Safe in Deflection.				
Check for the Development Length				
$L_d = 0.87 f_y \varphi / (4\tau_{bd})$	776.7857143	Mm	τ_{bd}=1.4 for M25 from clause 26.2.1.1	From Clause 26.2 of Code IS 456:2000
$M_1 = 0.87 \times f_y \times A_{st} \times d \times (1 - (f_y \times A_{st})/(f_{ck} \times b \times d)$	69.81783504	KNm.		From ANNEX G - 1.1 b) of Code IS 456:2000
Take L_o = largest(d,12ϕ)	192	Mm	L_o = Anchor Length	
$1.3(M_1/v) + L_o$ =	2636.433334	Mm	1.3(M1/v)+Lo > Ld	From Clause 26.2.3.3 of Code IS 456:2000
Development Length is OK.				

7.5 FOOTING

Table 81: Detailed Design of Isolated Footing (Footing F1,30)

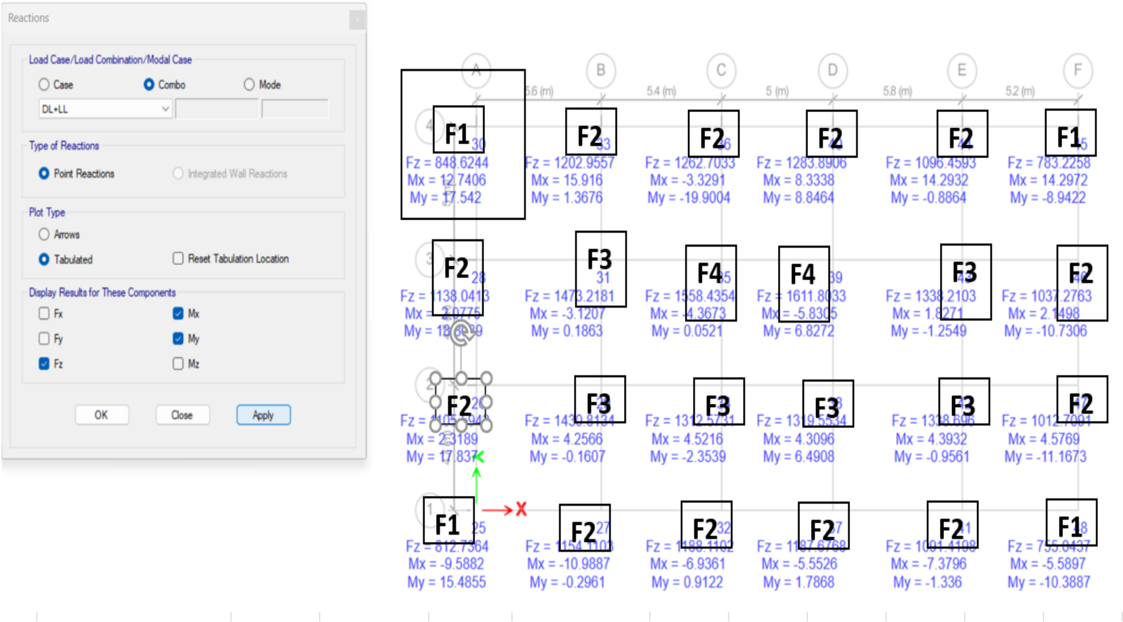

Fig 70: Base Reaction in footing *Source: ETABS*

	Footing ID		F1(30)		
S.No.	**CALCULATIONS**	**VALUES**	**UNIT**	**REMARS**	**INFORMATION**
	Column Size	500 ×500	mm ×mm		
	Length of Column(l)	500	mm		
	Width of Column (b)	500	mm		
1	**Known Data**				
	Type of Footing	Isolated Footing			
	Service Load (P)	848.6244	KN		From Etabs
	Grade of Concrete (f_{ck})	25	N/mm^2		
	Grade of Steel (f_y)	500	N/mm^2		
	Effective Cover	50	mm		
	Safe Bearing Capacity of Soil (q)	150	KN/m^2		Assume
	Moment in X-DIR (M_x)	12.7406	KNm		From Etabs
	Moment in Y-DIR (M_y)	17.542	KNm		From Etabs
	Diameter of Column Bar	25	mm		
	Depth of Foundation (D_f)	1.7	m		

	Unit Weight of Soil (γ_s)	18.5	KN/m^3		
2	**Size of Footing**				
	Total Service load (P_u) = Service load + Self Weight of Footing				
	$P_u = P + \gamma_s \times D_f \times \frac{P}{q}$				
	P_u	1026.5526	KN		
	$Area\ of\ Footing\ required\ (A_{required}) = \frac{P_u}{q}$				
	$A_{required}$	6.8436843	m^2		
	Providig Square Footing				
	Length of Footing (L) = Breadth of Footing (B) =				
	$L = B = (A_{required})^{\frac{1}{2}}$				
	L=B=	2.6160436	m		
	Take L=B=	2.8	m		
		2800	mm		
	Area of Footing Provided ($A_{provided}$) = L ×B				
	$A_{provided}$	7.84	m^2		
3	**Calculation of Net Earth Pressure Acting Upward and Eccentricity.**				
	Net upward pressure (w) $= \frac{P_u}{A}$				
		130.93784	KN/m^2		
	Size of Footing is ok				
	Factored Soil pressure	196.40676	KN/m^2		
	w	0.1964068	N/mm^2		
	$Eccentricity\ about\ X - DIR\ (e_X) = \frac{M_X}{P}$				
	e_x	12.411054			
	$Eccentricity\ about\ Y - DIR\ (e_Y) = \frac{M_Y}{P}$				
	e_y	17.088261	mm		
4	**Calculation of Maximum Bending Moment**				
	$Moment\ at\ Y - Y\ face\ (M_{Y-Y}) = w \times B \times \left\{\left(\frac{L}{2}\right) - \left(\frac{l}{2}\right) + e_X\right\}^2$				$\times \frac{1}{2}$

		371.53858	KNm		
	$Moment\ at\ X-X face\ (M_{X-X}) = w \times L \times \left\{\left(\frac{B}{2}\right) - \left(\frac{b}{2}\right) + e_Y\right\}^2 * \frac{1}{2}$				
	M_{X-X}	374.53453	KNm		
	M_u = Greater moment of M_{X-X} and M_{Y-Y}				
	M_u	374.53453	KNm		
5	**Calculation of Depth of Footing**				
	Mmax = 0.133 $\times f_{ck} \times b \times d^2$				
	$d_{required}$	200.57236	mm		
	Adopt d	650	mm		
	Cover	50	mm	assume	
	Total Depth (D)	700	mm		
6	**Check for one way shear**				
	$Shear\ Force\ at\ Y-Y\ face\ (V_Y) = w \times B \times \left(\frac{L}{2} - \frac{l}{2} + e_Y - d\right)$				
	V_Y	284.36696	KN		
	$Shear\ Force\ at\ Y-Y\ face\ (V_Y) = w \times L \times \left(\frac{B}{2} - \frac{b}{2} + e_X - d\right)$				
	V_X	281.79478	KN		
	Critical One way Shear(V_u) = Maximum of V_X and V_Y				
	V_u	284.36696	KN		
	$Shear\ Stess\ \tau_V = \frac{V_u}{B * d}$	0.1562456	N/mm^2		
	$For\ Safe\ in\ Shear\ \tau_V \leq \tau_c'$				
	where, $\tau_c' = k_s \tau_c$				
	Let us assume P_t = 0.15%				
	For M25 concrete (τ_c)	0.29	N/mm^2	By Interpolation Using Table 19 of IS 456:2000	
	$k_s=(0.5+\beta_c) \leq 1$ where βc is the ratio of short side to long side of column	1		For square column	Cl 31.6.3.1 of IS 456:2000
	τ_c'	0.29	N/mm^2		

	Safe in one way Shear				
7	*Check for two way Shear (Punching Shear)*				
	V_u = w × (L ×B-(l+d)(b+d))	1280.081	KN		
	b_o = 2 × (l+d+b+d)	4600	mm		
	$\tau_V = \dfrac{V_u}{b_o \times d}$	0.4281207	N/mm^2		
	k_s	1			
	$\tau_c' = k_s \times 0.25 \times \sqrt{f_{ck}}$	1.25	N/mm^2		
	Safe in two way Shear				
8	**Calculation of Reinforcement**				
	$A_{st,min}$=0.12%	2184	mm^2		
	Bending Moment (Mu)	374.53453	KN/m		
	A_{st1}	1345.1602	mm^2		
	Assumed Ast=0.15% ×B ×d	2730	mm^2	$A_{st,min}$=0.12%	
	Ast Required ($A_{st,req}$)	2730	mm^2		
	Provide diameter of main bars (ϕ)	16	mm		
	Provide diameter of main bars (ϕ)	16	mm		
	Area of Single main Bar	201.06176	mm^2		
	Area of Single Distribution Bar	201.06176	mm^2		
	Spacing in X-X direction for main bars				
	$S = \dfrac{L \times \frac{\pi}{4} \times \phi^2}{A_{st,req}}$	206.21719	mm		
	Provide (s)	200	mm		
	$A_{st,prov}$	2814.8646	mm^2		
	Ast Provided is ok				
	Number of Bars (n)	15			
	Provide (n)	18	nos		
	Provide 16 mm diameter of main bars @ 200mm c/c spacing in both direction				
9	**Check for Development Length**				
	τ_{bd}=1.6 ×1.4	2.24			
	$L_{d,required} = \dfrac{\phi \times 0.87 \times f_y}{4 \times \tau_{bd}}$	776.78571	mm		

					Cl 26.2 of IS 456:2000
	$L_{d,provided}$	800	mm		
	$L_{d,available} = \frac{L}{2} - \frac{l}{2} - e_X - cover$				
		1087.5889	mm		
	Since available space is greater than Ld provided so ok				
10	**Check for Bearing Capacity**				
	$Bearing\ Stress\ on\ column\ (\sigma_{br}) = \frac{P_u}{A_c}$				
		5.0917464	N/mm^2		
	Allowable Bearing capacity =0.45 $\times$ f_{ck}	11.25	N/mm^2		
	Hence ok				

Table 82: Detailed Design of Isolated Footing (Footing F2,40)

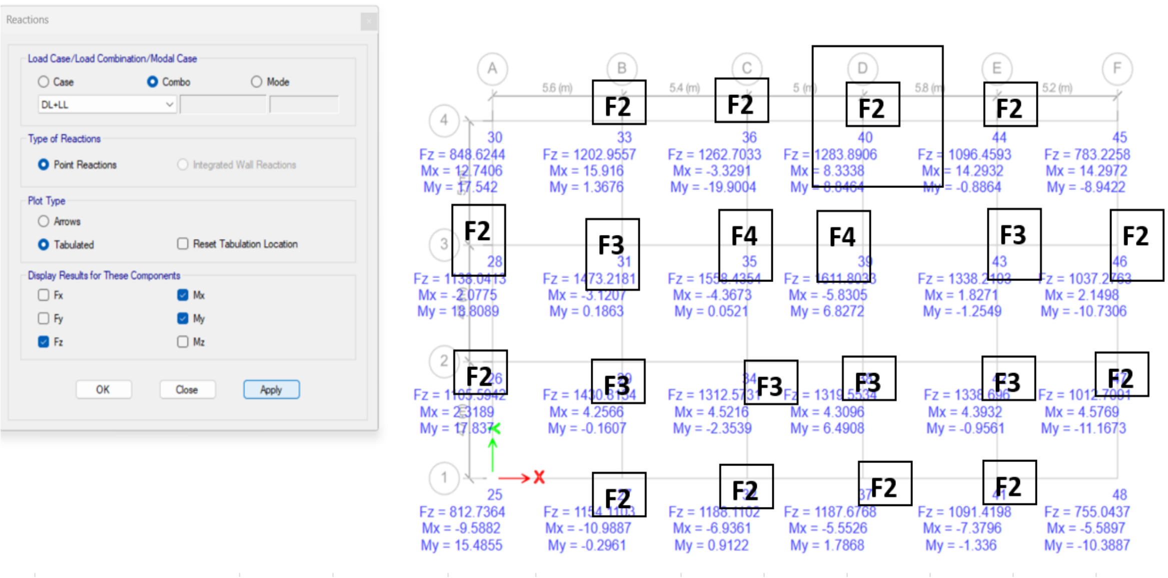

Fig 71: Base Reaction in footing *Source: ETABS*

	Footing ID		**40**		
S.No.	**CALCULATIONS**	**VALUES**	**UNIT**	**REMARKS**	**INFORMATION**
	Column Size	550*550	mm*mm		
	Length of Column(l)	550	mm		

	Width of Column (b)	550	mm		
1	**Known Data**				
	Type of Footing	Isolated Footing			
	Service Load (P)	1283.8906	KN		From Etabs
	Grade of Concrete (f_{ck})	25	N/mm^2		
	Grade of Steel (f_y)	500	N/mm^2		
	Effective Cover	50	mm		
	Safe Bearing Capacity of Soil (q)	150	KN/m^2		Assume
	Moment in X-DIR (M_x)	3.3291	KNm		From Etabs
	Moment in Y-DIR (M_y)	19.9004	KNm		From Etabs
	Diameter of Column Bar	25	mm		
	Depth of Foundation (D_f)	1.7	m		
	Unit Weight of Soil (γ_s)	18.5	KN/m^3		
2	**Size of Footing**				
	Total Service load (P_u) = Service load + Self Weight of Footing				
	$P_u = P + \gamma_s * D_f * \frac{P}{q}$				
	P_u	1553.0797	KN		
	$Area\ of\ Footing\ required\ (A_{required}) = \frac{P_u}{q}$				
	$A_{required}$	10.353864	m^2		
	Providig Square Footing				
	Length of Footing (L) = Breadth of Footing (B) =				
	$L = B = (A_{required})^{\frac{1}{2}}$				
	L=B=	3.2177421	m		
	Take L=B=	3.3	m		
		3300	mm		
	Area of Footing Provided ($A_{provided}$) = L*B				

	$A_{provided}$	10.89	m^2		
3	**Calculation of Net Earth Pressure Acting Upward and Eccentricity.**				
	Net upward pressure (w) $= \frac{P_u}{A}$				
		142.6152 1	KN/m^2		
	Size of Footing is ok				
	Factored Soil pressure	213.9228 2	KN/m^2		
	w	0.213922 8	N/mm^2		
	$Eccentricity\ about\ X-DIR\ (e_X) = \frac{M_X}{P}$				
	e_x	2.143547 5			
	$Eccentricity\ about\ Y-DIR\ (e_Y) = \frac{M_Y}{P}$				
	e_y	12.81350 9	mm		
4	**Calculation of Maximum Bending Moment**				
	$Moment\ at\ Y-Y\ face\ (M_{Y-Y}) = w * B * \left\{\left(\frac{L}{2}\right)-\left(\frac{l}{2}\right)+e_X\right\}^2 * \frac{1}{2}$				
	M_{Y-Y}	669.4212 3	KNm		
	$Moment\ at\ X-Xface\ (M_{X-X}) = w * L * \left\{\left(\frac{B}{2}\right)-\left(\frac{b}{2}\right)+e_Y\right\}^2 * \frac{1}{2}$				
	M_{X-X}	679.8346 2	KNm		
	M_u = Greater moment of M_{X-X} and M_{Y-Y}				
	M_u	679.8346 2	KNm		
5	Calculation of Depth of Footing				
	Mmax = $0.133*f_{ck}*b*d^2$				
	$d_{required}$	248.9137 2	mm		
	Adopt d	650	mm		
	Cover	50	mm	assume	
	Total Depth (D)	700	mm		
6	Check for one way shear				

	$$Shear\ Force\ at\ Y-Y\ face\ (V_Y) = w*B*\left(\frac{L}{2}-\frac{l}{2}+e_Y-d\right)$$				
	V_Y	520.85598	KN		
	$$Shear\ Force\ at\ Y-Y\ face\ (V_Y) = w*L*\left(\frac{B}{2}-\frac{b}{2}+e_X-d\right)$$				
	V_X	513.32357	KN		
	Critical One way Shear(V_u) = Maximum of V_X and V_Y				
	V_u	520.85598	KN		
	$Shear\ Stess\ \tau_V = \frac{V_u}{B*d}$	0.2428233	N/mm^2		
	$For\ Safe\ in\ Shear\ \tau_V \leq \tau_c'$				
	where, $\tau_c' = k_s\tau_c$				
	Let us assume P_t = 0.15%				
	For M25 concrete (τ_c)	0.29	N/mm^2	By Interpolation Using Table 19 of IS 456:2000	
	$k_s=(0.5+\beta_c)\leq 1$ where βc is the ratio of short side to long side of column	1		For square column	Cl 31.6.3.1 of IS 456:2000
	τ_c'	0.29	N/mm^2		
	Safe in one way Shear				
7	**Check for two way Shear (Punching Shear)**				
	V_u = w*(L*B-(l+d)(b+d))	2021.5706	KN		
	b_o = 2*(l+d+b+d)	4800	mm		
	$\tau_V = \frac{V_u}{b_o*d}$	0.6479393	N/mm^2		
	k_s	1			
	$\tau_c' = k_s*0.25*\sqrt{f_{ck}}$	1.25	N/mm^2		
	Safe in two way Shear				
8	**Calculation of Reinforcement**				
	$A_{st,min}$=0.12%	2574	mm^2		
	Bending Moment (Mu)	679.83462	KN/m		

	A_{st1}	2462.0897	mm^2		
	Assumed Ast=0.15%*B*d	3217.5	mm^2	$A_{st,min}$=0.12%	
	Ast Required ($A_{st,req}$)	3217.5	mm^2		
	Provide diameter of main bars (ϕ)	16	mm		
	Provide diameter of main bars (ϕ)	16	mm		
	Area of Single main Bar	201.06176	mm^2		
	Area of Single Distribution Bar	201.06176	mm^2		
	Spacing in X-X direction for main bars				
	$S = \frac{L * \frac{\pi}{4} * \phi^2}{A_{st,req}}$	206.21719	mm		
	Provide (s)	200	mm		
	$A_{st,prov}$	3317.519	mm^2		
	Ast Provided is ok				
	Number of Bars (n)	17.5			
	Provide (n)	18	nos		
	Provide 16 mm diameter of main bars @ 200mm c/c spacing in both direction				
9	**Check for Development Length**				
	τ_{bd}=1.6*1.4	2.24			Cl 26.2 of IS 456:2000
	$L_{d,required} = \frac{\phi * 0.87 * f_y}{4 * \tau_{bd}}$	776.78571	mm		
	$L_{d,provided}$	800	mm		
	$L_{d,available} = \frac{L}{2} - \frac{l}{2} - e_X - cover$				
		1322.8565	mm		
	Since available space is greater than Ld provided so ok				
10	**Check for Bearing Capacity**				
	$Bearing\ Stress\ on\ column\ (\sigma_{br}) = \frac{P_u}{A_c}$				

		6.3663997	N/mm^2		
	Allowable Bearing capacity =0.45 * f_{ck}	11.25	N/mm^2		
	Hence ok				

Table 83: Detailed Design of Isolated Footing (Footing F3,31)

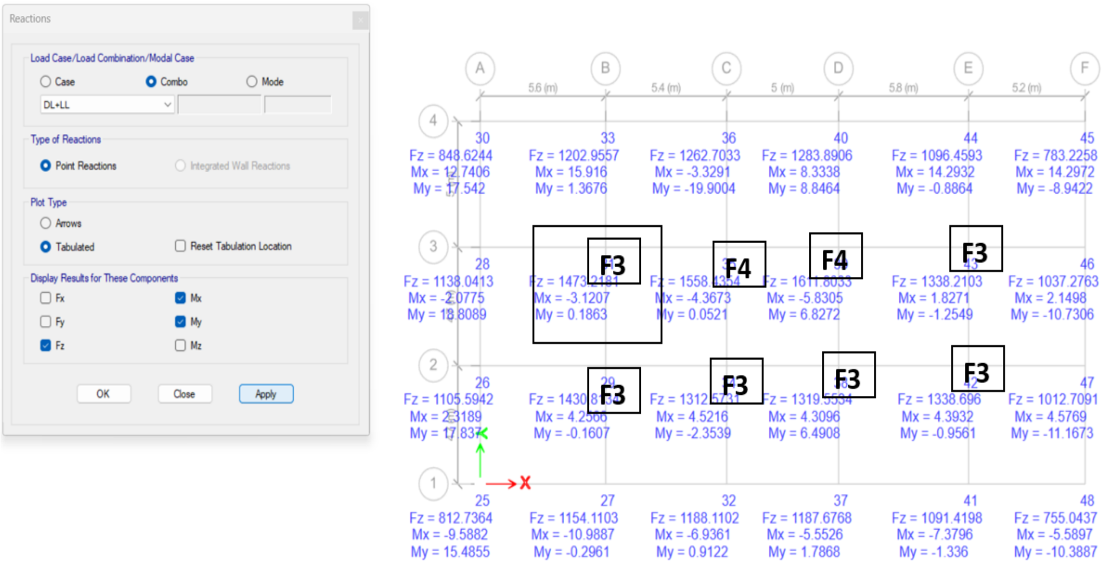

Fig 72: Base Reaction in footing *Source: ETABS*

	Footing ID		F3,31		
S. No.	**CALCULATIONS**	**VALUES**	**UNIT**	**REMARKS**	**INFORMATION**
	Column Size	500*500	mm*mm		
	Length of Column(l)	500	mm		
	Width of Column (b)	500	mm		
1	**Known Data**				
	Type of Footing	Isolated Footing			
	Service Load (P)	1473.2181	KN		From Etabs
	Grade of Concrete (f_{ck})	25	N/mm^2		
	Grade of Steel (f_y)	500	N/mm^2		
	Effective Cover	50	mm		
	Safe Bearing Capacity of Soil (q)	150	KN/m^2		Assume
	Moment in X-DIR (M_x)	3.1207	KN m		From Etabs
	Moment in Y-DIR (M_y)	0.1863	KN m		From Etabs

	Diameter of Column Bar	25	mm		
	Depth of Foundation (D_f)	1.7	m		
	Unit Weight of Soil (γ_s)	18.5	KN/m^3		
2	**Size of Footing**				
	Total Service load (P_u) = Service load + Self Weight of Footing				
	$P_u = P + \gamma_s * D_f * \frac{P}{q}$				
	P_u	1782.1028	KN		
	$Area\ of\ Footing\ required\ (A_{required}) = \frac{P_u}{q}$				
	$A_{required}$	11.880686	m^2		
	Providig Square Footing				
	Length of Footing (L) = Breadth of Footing (B) =				
	$L = B = (A_{required})^{\frac{1}{2}}$				
	L=B=	3.446837	m		
	Take L=B=	3.5	m		
		3500	mm		
	Area of Footing Provided ($A_{provided}$) = L*B				
	$A_{provided}$	12.25	m^2		
3	**Calculation of Net Earth Pressure Acting Upward and Eccentricity.**				
	Net upward pressure (w) $= \frac{P_u}{A}$				
		145.47778	KN/m^2		
	Size of Footing is ok				
	Factored Soil pressure	218.21667	KN/m^2		
	w	0.2182167	N/mm^2		
	$Eccentricity\ about\ X - DIR\ (e_X) = \frac{M_X}{P}$				
	e_x	1.7511335			
	$Eccentricity\ about\ Y - DIR\ (e_Y) = \frac{M_Y}{P}$				

	e_y	0.1045394	mm		
4	**Calculation of Maximum Bending Moment**				
	$Moment\ at\ Y-Y\ face\ (M_{Y-Y}) = w*B*\left\{\left(\frac{L}{2}\right)-\left(\frac{l}{2}\right)+e_X\right\}^2*\frac{1}{2}$				
	M_{Y-Y}	861.23548	KN m		
	$Moment\ at\ X-X face\ (M_{X-X}) = w*L*\left\{\left(\frac{B}{2}\right)-\left(\frac{b}{2}\right)+e_Y\right\}^2*\frac{1}{2}$				
	M_{X-X}	859.34792	KN m		
	M_u = Greater moment of M_{X-X} and M_{Y-Y}				
	M_u	861.23548	KN m		
5	**Calculation of Depth of Footing**				
	Mmax = 0.133*f_{ck}*b*d^2				
	$d_{required}$	272.03896	mm		
	Adopt d	650	mm		
	Cover	50	mm	assume	
	Total Depth (D)	700	mm		
6	**Check for one way shear**				
	$Shear\ Force\ at\ Y-Y\ face\ (V_Y) = w*B*\left(\frac{L}{2}-\frac{l}{2}+e_Y-d\right)$				
	V_Y	649.27444	KN		
	$Shear\ Force\ at\ Y-Y\ face\ (V_Y) = w*L*\left(\frac{B}{2}-\frac{b}{2}+e_X-d\right)$				
	V_X	650.53204	KN		
	Critical One way Shear(V_u) = Maximum of V_X and V_Y				
	V_u	650.53204	KN		
	$Shear\ Stess\ \tau_V = \frac{V_u}{B*d}$	0.2859482	N/mm^2		
	$For\ Safe\ in\ Shear\ \tau_V \le \tau_c'$				
	where, $\tau_c' = k_s\tau_c$				
	Let us assume P_t = 0.2%				

	For M25 concrete (τ_c)	0.325	N/mm^2	By Interpolation Using Table 19 of IS 456:2000	
	$k_s=(0.5+\beta_c)\leq 1$ where βc is the ratio of short side to long side of column	1		For square column	Cl 31.6.3.1 of IS 456:2000
	τ_c'	0.325	N/mm^2		
	Safe in one way Shear				
7	**Check for two way Shear (Punching Shear)**				
	$V_u = w*(L*B-(l+d)(b+d))$	2384.5627	KN		
	$b_o = 2*(l+d+b+d)$	4600	mm		
	$\tau_V = \frac{V_u}{b_o * d}$	0.7975126	N/mm^2		
	k_s	1			
	$\tau_c' = k_s * 0.25 * \sqrt{f_{ck}}$	1.25	N/mm^2		
	Safe in two way Shear				
8	**Calculation of Reinforcement**				
	$A_{st,min}$=0.12%	2730	mm^2		
	Bending Moment (Mu)	861.23548	KN/m		
	A_{st1}	3133.7836	mm^2		
	Assumed Ast=0.2%*B*d	4550	mm^2	$A_{st,min}$=0.12%	
	Ast Required ($A_{st,req}$)	4550	mm^2		
	Provide diameter of main bars (ϕ)	16	mm		
	Provide diameter of main bars (ϕ)	16	mm		
	Area of Single main Bar	201.06176	mm^2		
	Area of Single Distribution Bar	201.06176	mm^2		
	Spacing in X-X direction for main bars				
	$S = \frac{L * \frac{\pi}{4} * \phi^2}{A_{st,req}}$	154.66289	mm		
	Provide (s)	140	mm		

	$A_{st,prov}$	5026.544	mm^2		
	Ast Provided is ok				
	Number of Bars (n)	26			
	Provide (n)	26	nos		
	Provide 16 mm diameter of main bars @ 140mm c/c spacing in both direction				
9	**Check for Development Length**				
	τ_{bd}=1.6*1.4	2.24			Cl 26.2 of IS 456:2000
	$L_{d,required} = \frac{\phi * 0.87 * f_y}{4 * \tau_{bd}}$	776.78571	mm		
	$L_{d,provided}$	800	mm		
	$L_{d,available} = \frac{L}{2} - \frac{l}{2} - e_X - cover$				
		1448.2489	mm		
	Since available space is greater than Ld provided so ok				
10	**Check for Bearing Capacity**				
	$Bearing\ Stress\ on\ column\ (\sigma_{br}) = \frac{P_u}{A_c}$				
		8.8393086	N/mm^2		
	Allowable Bearing capacity =0.45 * f_{ck}	11.25	N/mm^2		
	Hence ok				

Table 84: Detailed Design of Isolated Footing (Footing F4,39)

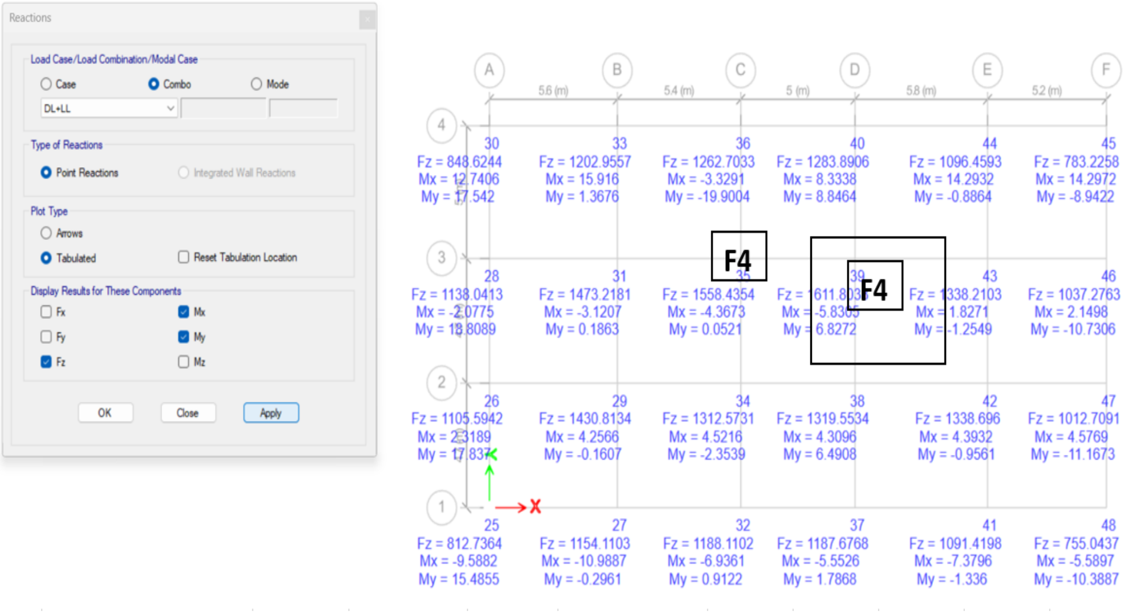

Fig 73: Base Reaction in footing *Source: ETABS*

	Footing ID		F4,39		
S.No.	CALCULATIONS	VALUES	UNIT	REMARKS	INFORMATION
	Column Size	550*550	mm*mm		
	Length of Column(l)	550	mm		
	Width of Column (b)	550	mm		
1	**Known Data**				
	Type of Footing	Isolated Footing			
	Service Load (P)	1611.8942	KN		From Etabs
	Grade of Concrete (f_{ck})	25	N/mm^2		
	Grade of Steel (f_y)	500	N/mm^2		
	Effective Cover	50	mm		
	Safe Bearing Capacity of Soil (q)	150	KN/m^2		Assume
	Moment in X-DIR (M_x)	5.8292	KNm		From Etabs
	Moment in Y-DIR (M_y)	6.8386	KNm		From Etabs
	Diameter of Column Bar	25	mm		
	Depth of Foundation (D_f)	1.7	m		

	Unit Weight of Soil (γ_s)	18.5	KN/m^3		
2	**Size of Footing**				
	Total Service load (P_u) = Service load + Self Weight of Footing				
	$P_u = P + \gamma_s * D_f * \frac{P}{q}$				
	P_u	1949.8547	KN		
	$Area\ of\ Footing\ required\ (A_{required}) = \frac{P_u}{q}$				
	$A_{required}$	12.999031	m^2		
	Providig Square Footing				
	Length of Footing (L) = Breadth of Footing (B) =				
	$L = B = (A_{required})^{\frac{1}{2}}$				
	L=B=	3.6054169	m		
	Take L=B=	3.7	m		
		3700	mm		
	Area of Footing Provided ($A_{provided}$) = L*B				
	$A_{provided}$	13.69	m^2		
3	**Calculation of Net Earth Pressure Acting Upward and Eccentricity.**				
	Net upward pressure (w) $= \frac{P_u}{A}$				
		142.42912	KN/m^2		
	Size of Footing is ok				
	Factored Soil pressure	213.64368	KN/m^2		
	w	0.2136437	N/mm^2		
	$Eccentricity\ about\ X - DIR\ (e_X) = \frac{M_X}{P}$				
	e_x	2.9895561			
	$Eccentricity\ about\ Y - DIR\ (e_Y) = \frac{M_Y}{P}$				

	e_y	3.507235 7	mm		
4	**Calculation of Maximum Bending Moment**				
	$Moment\ at\ Y-Y\ face\ (M_{Y-Y}) = w * B * \left\{\left(\frac{L}{2}\right) - \left(\frac{l}{2}\right) + e_X\right\}^2 * \frac{1}{2}$				
	M_{Y-Y}	984.1698	KNm		
	$Moment\ at\ X-Xface\ (M_{X-X}) = w * L * \left\{\left(\frac{B}{2}\right) - \left(\frac{b}{2}\right) + e_Y\right\}^2 * \frac{1}{2}$				
	M_{X-X}	984.8156 5	KNm		
	M_u = Greater moment of M_{X-X} and M_{Y-Y}				
	M_u	984.8156 5	KNm		
5	**Calculation of Depth of Footing**				
	Mmax = $0.133*f_{ck}*b*d^2$				
	$d_{required}$	282.9311 6	mm		
	Adopt d	650	mm		
	Cover	50	mm	assume	
	Total Depth (D)	700	mm		
6	**Check for one-way Shear**				
	$Shear\ Force\ at\ Y-Y\ face\ (V_Y) = w * B * \left(\frac{L}{2} - \frac{l}{2} + e_Y - d\right)$				
	V_Y	733.9679 1	KN		
	$Shear\ Force\ at\ Y-Y\ face\ (V_Y) = w * L * \left(\frac{B}{2} - \frac{b}{2} + e_X - d\right)$				
	V_X	733.5587	KN		
	Critical One way Shear(V_u) = Maximum of V_X and V_Y				
	V_u	733.9679 1	KN		
	$Shear\ Stess\ \tau_V = \frac{V_u}{B * d}$	0.305184 2	N/mm²		
	$For\ Safe\ in\ Shear\ \tau_V \leq \tau_c'$				
	where, $\tau_c' = k_s \tau_c$				

	Let us assume P_t = 0.2%				
	For M25 concrete (τ_c)	0.325	N/mm^2	By Interpolation Using Table 19 of IS 456:2000	
	k_s=(0.5+β_c)≤1where βc is the ratio of short side to long side of column	1		For square column	Cl 31.6.3.1 of IS 456:2000
	τ_c'	0.325	N/mm^2		
	Safe in one-way Shear				
7	**Check for two-way Shear (Punching Shear)**				
	V_u = w*(L*B-(l+d)(b+d))	2617.1351	KN		
	b_o = 2*(l+d+b+d)	4800	mm		
	$\tau_V = \frac{V_u}{b_o * d}$	0.8388254	N/mm^2		
	k_s	1			
	$\tau_c' = k_s * 0.25 * \sqrt{f_{ck}}$	1.25	N/mm^2		
	Safe in two-way Shear				
8	**Calculation of Reinforcement**				
	$A_{st,min}$=0.12%	2886	mm^2		
	Bending Moment (Mu)	984.81565	KN/m		
	A_{st1}	3592.0311	mm^2		
	Assumed Ast=0.2%*B*d	4810	mm^2	$A_{st,min}$=0.12%	
	Ast Required ($A_{st,req}$)	4810	mm^2		
	Provide diameter of main bars (ϕ)	16	mm		
	Provide diameter of main bars (ϕ)	16	mm		
	Area of Single main Bar	201.06176	mm^2		
	Area of Single Distribution Bar	201.06176	mm^2		
	Spacing in X-X direction for main bars				
	$S = \frac{L * \frac{\pi}{4} * \phi^2}{A_{st,req}}$	154.66289	mm		

	Provide (s)	140	mm		
	$A_{st,prov}$	5313.775 1	mm^2		
	Ast Provided is ok				
	Number of Bars (n)	27.42857 1			
	Provide (n)	28	nos		
	Provide 16 mm diameter of main bars @ 140mm c/c spacing in both direction				
9	**Check for Development Length**				
	τ_{bd}=1.6*1.4	2.24			Cl 26.2 of IS 456:2000
	$L_{d,required} = \frac{\phi * 0.87 * f_y}{4 * \tau_{bd}}$	776.7857 1	mm		
	$L_{d,provided}$	800	mm		
	$L_{d,available} = \frac{L}{2} - \frac{l}{2} - e_X - cover$				
		1522.010 4	mm		
	Since available space is greater than Ld provided so ok				
10	**Check for Bearing Capacity**				
	$Bearing\ Stress\ on\ column\ (\sigma_{br}) = \frac{P_u}{A_c}$				
		7.992863 8	N/mm^2		
	Allowable Bearing capacity =0.45 * f_{ck}	11.25	N/mm^2		
	Hence ok				

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