STRUCTURAL DESIGN AND ANALYSIS OF EARTHQUAKE RESISTANT MULTI-STORIED RCC FRAMED HOSPITAL BUILDING

A Project report submitted to the

SCHOOL OF ENGINEERING
FACULTY OF SCIENCE AND TECHNOLOGY
POKHARA UNIVERSITY, NEPAL

By

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In partial fulfillment of the requirements

for the award of the degree of

BACHELOR OF CIVIL AND RURAL ENGINEERING
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STRUCTURAL DESIGN AND ANALYSIS OF EARTHQUAKE RESISTANT MULTI-STORIED RCC FRAMED HOSPITAL BUILDING

Supervised by
Dr. Hemchandra Chaulagain
Er. Dipesh Jaisi Poudel

A report submitted in partial fulfillment of the requirements for the Bachelor Degree in Civil and Rural Engineering

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DEDICATION

To our esteemed professors and mentors, who have imparted invaluable knowledge and wisdom, we are eternally grateful. Their dedication to teaching and guidance has not only enriched our academic experience but also shaped us into the engineers we aspire to become.

This project is dedicated to our beloved families and friends, whose unwavering support and encouragement have been the cornerstone of our educational journey. Their constant inspiration and faith in our abilities have been a source of strength, motivating us to strive for excellence in every endeavor.

Lastly, we dedicate this work to the countless professionals in the field of civil engineering who have paved the way with their innovations and tireless efforts. Their contributions to the field inspire us to continue learning and to one day contribute meaningfully to the built environment.

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DECLARATION

We hereby declare that the project report titled "**STRUCTURAL DESIGN AND ANALYSIS OF EARTHQUAKE RESISTANT MULTI-STORIED RCC FRAMED HOSPITAL BUILDING**" is our own work, completed under the guidance of Dr.Hemchandra Chaulagain and Er. Dipesh Poudel, carried out as part of the academic requirement for the completion of our Bachelor's degree course. This report has not been submitted to any other university or institution for any degree or diploma.

We confirm that all the information in this report is based on our research, analysis, and design work. We have acknowledged all sources of information and properly referenced any work from other researchers.

We take full responsibility for the accuracy of the information and data presented in this report, and we have acknowledge any help we received during the project.

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RECOMMENDATION

This is to certify that this project report entitled "**STRUCTURAL DESIGN AND ANALYSIS OF EARTHQUAKE RESISTANT MULTI-STORIED RCC FRAMED HOSPITAL BUILDING"** prepared and submitted by **Santona Baral, Bishnu Subedi, Sandhya Thapa, Sanchita Acharya, Niruta KC, Kriti Kumari Sapkota**, in partial fulfillment of the requirements of the Bachelor degree in Civil and Rural Engineering awarded by Pokhara University, has been completed under our supervision. We recommend the same for acceptance by Pokhara University.

Assoc.Prof. Dr. Hemchandra Chaulagain

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Date:2025/07/02

POKHARA UNIVERSITY
FACULTY OF SCIENCE AND TECHNOLOGY
SCHOOL OF ENGINEERING
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CERTIFICATE

This thesis entitled **"STRUCTURAL DESIGN AND ANALYSIS OF EARTHQUAKE RESISTANT MULTI-STORIED RCC FRAMED HOSPITAL BUILDING"** prepared and submitted by **Santona Baral, Bishnu Subedi, Sandhya Thapa, Sanchita Acharya, Niruta KC, Kriti Kumari Sapkota** has been examined by us and is accepted for the award of the Bachelor in Civil and Rural Engineering in by Pokhara University.

Er. Sushil Adhikari
External Examiner

Dr. Hemchandra Chaulagain
Supervisor

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ACKNOWLEDGEMENT

It is our great pleasure to submit this report on "**STRUCTURAL DESIGN AND ANALYSIS OF EARTHQUAKE RESISTANT MULTI-STORIED RCC FRAMED HOSPITAL BUILDING**" to the Faculty of Science and Technology in partial fulfillment of the requirement for the Bachelor's degree in Civil and Rural Engineering.

Completing this project has been a wonderful experience. The success of this project required a lot of guidance and assistance from many people, and we would like to express our deep gratitude to every individual for their support and coordination.

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We would also like to thank the School of Engineering Department, Pokhara University for providing us with the best environment for us to complete the project.

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ABSTRACT

This report entitled **"STRUCTURAL DESIGN AND ANALYSIS OF EARTHQUAKE RESISTANT MULTI-STORIED RCC FRAMED HOSPITAL BUILDING"** is a project work carried by students from BCRE 8th semester (2020 batch) for the partial fulfillment of Bachelor degree in Civil and Rural Engineering's fourth year course demand. This work outlines a comprehensive study on the structural design and analysis of an earthquake-resistant, multi-storied reinforced cement concrete (RCC) framed hospital building located in Birauta, Pokhara , Nepal. The main purpose of this work is to analyze and design of seismic resistant building that ensures safety, functionality, and resilience against seismic forces. The analysis and design of earthquake resistant buildings are considered to be very important not only to mitigate their effects but also to prevent from the future earthquake.Hospitals, being critical infrastructure, require uninterrupted functionality during and after seismic events; thus, their structural integrity is a must. The project will utilize current seismic codes, advanced modeling software and performance-based design principles to analyze load combinations, lateral forces, and dynamic responses. Special attention will be given to ductility, redundancy, and proper detailing to enhance earthquake resistance. This project shall help us, as future engineers, to be better acquainted with the ways and principles of design of earthquake resistant buildings

Keywords: NBC 105:2020, Shear strength, Exterior beam-column joint. Numerical Database, Monotonic loading

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LIST OF ACRONYMS

	Description
α_x, α_y	BM coefficients for Rectangular Slab Panels
ϕ	Diameter of Bar, Angle of internal friction of soil
δ_m	Percentage reduction in moment
τ_c	Shear Stress in Concrete
$\tau_{c,max}$	Max. shear stress in concrete with shear reinforcement
τ_{bd}	Design Bond Stress
σ_{ac}	Permissible Stress in Axial Compression (Steel)
σ_{cbc}	Permissible Bending Compressive Strength of Concrete
σ_{sc}, σ_{st}	Permissible Stress in Steel in Compression and Tension respectively
γ_m	Partial Safety Factor for Material
γ_f	Partial Safety Factor for Load
γ	Unit Weight of Material
A_b	Area of Each Bar
A_g	Gross Area of Concrete
A_{sc}	Area of Steel in Compression
A_{st}	Area of Steel in Tension
A_{sv}	Area of Stirrups
B or b	Width or shorter dimension in plan
b_f	Effective width of flange
d	Effective Depth
d′	Effective Cover
D	Overall Depth
D_f	Thickness of Flange
e_x	Eccentricity along x-direction
e_y	Eccentricity along y-direction
EL_x, EL_y	Earthquake Load along X and Y direction respectively
f_{ck}	Characteristics Strength of Concrete
f_y	Characteristic Strength of Steel
I	Importance Factor (For Base Shear Calculation)
I_{XX}, I_{YY}	Moment of Inertia (along x and y direction)
k	Coefficient of Constant or factor

K	Stiffness
L	Length of Member
l_{ef}	Effective Length of member
L_d	Development Length
M	Modular Ratio
M or BM	Bending Moment
N_u or P_u	Ultimate Axial Load on a compression member
P_c	Percentage of Compression Reinforcement
P_t	Percentage of Tension Reinforcement
q, q_u	Permissible and Ultimate bearing capacity of soil
SR, r_{min}	Slenderness Ratio, (minimum) for structural steel section
R	Response Reduction Factor
S_v	Spacing of Each Bar
V_B	Design Seismic Base Shear
V	Shear Force
W_i	Seismic Weight ofi^{th} Floor
WL	Wind Load
Z	Seismic Zone Factor
CM	Center of Mass
CR	Center of Rigidity
D.L	Dead Load
IS	Indian Standard
L.L	Live Load
RCC	Reinforced Cement Concrete
NBC	Nepal Building Code
ULS	Ultimate Limit State
SLS	Serviceability Limit State
RS	Response Spectrum
EQx	Earthquake Load in X direction
EQy	Earthquake Load in Y direction

UNITS AND CONVERSION

m^3	Meter cube (Cubic meter)
Sq.ft	Square.feet
m^2	Square meter
kN	Kilo Newton
mm	Millimeter
cm	Centimeter
m	Meter
kg	Kilo gram
N	Netwon

CHAPTER 1
INTRODUCTION

1.1. Background

Healthcare systems around the world are under increasing pressure to provide high-quality, accessible, and efficient services to growing and diverse populations. In many regions, existing hospital facilities are either outdated, overcrowded, or unable to meet the demands of modern medical care[1]. In this context, there is a compelling need to develop a new hospital building that addresses these challenges while promoting innovation in healthcare delivery. The proposed hospital design seeks to create a state-of-these-art facility that not only meets the highest standards of clinical care but also supports patient well-being, staff efficiency, and operational sustainability.

In addition to prioritizing patient care and staff workflow, the new hospital will integrate environmentally sustainable practices such as energy-efficient systems, water conservation technologies, and materials with low environmental impact[2]. Flexibility for future expansion and adaptability to technological advancements will be key features of the design, ensuring that the hospital remains relevant and effective in the long term.

Given Nepal's location in a seismically active region, the proposed hospital design places a strong emphasis on earthquake-resistant structural planning. The building will be designed in accordance with NBC 105:2020 and IS 456:2000, to ensure structural integrity and life safety during seismic events[4] [5].

Advanced analysis tool, ETABS will be used for dynamic modeling, including response spectrum to evaluate the building's performance under different earthquake scenarios[3]. The structural system will incorporate ductile detailing, shear walls, and lateral load-resisting elements to absorb and dissipate seismic energy efficiently.

By developing a thoughtfully designed hospital building, this project aims to strengthen the healthcare infrastructure, reduce health disparities, and contribute to the broader goal of community well-being and resilience.

1.1.1. Importance of structural safety in seismic zones

In our building project, ensuring structural safety is a top priority, especially because the structure is located in a seismic zone. Earthquakes pose a serious threat to buildings, and without proper design and construction, even a moderate tremor can lead to severe damage or collapse. Therefore, our project focuses on earthquake-resistant design principles to safeguard lives and property. We have considered factors like soil type, building geometry, and material strength to ensure the structure can absorb and dissipate seismic energy effectively. Key features include strong foundations, ductile materials, and reinforcement techniques that allow controlled deformation without failure. By following national seismic codes and safety standards, this project not only aims to withstand seismic forces but also to maintain functionality after an earthquake. Structural safety in seismic zones is not just a design requirement—it is a critical responsibility to ensure long-term resilience, reduce disaster impact, and protect the community.

1.1.2. Challenges in designing earthquake resistant hospital

Designing earthquake-resistant buildings comes with several challenges. Earthquakes are unpredictable in terms of location, intensity, and duration, and different soil types can affect how the ground shakes, making it harder to design a one-size-fits-all solution. Structurally, buildings must be able to resist both vertical and horizontal forces, requiring not just strength but flexibility (ductility) to avoid collapse. Irregular building shapes or layouts can increase the risk of damage during an earthquake. Choosing the right construction materials and ensuring high-quality workmanship are also crucial, as poor construction can lead to failure even with a good design. Retrofitting older buildings to meet modern standards is often expensive and technically demanding. Additionally, earthquake-resistant buildings generally cost more to build and maintain, which can be a challenge for budget-constrained projects. Designers must also follow strict building codes and deal with limitations in urban planning. Finally, human behavior, such as misuse of space, and the need for sustainable, eco-friendly materials add to the complexity of earthquake-resistant design.

1.1.3. Earthquake Design Philosophy

The earthquake-resistant design philosophy is based on the principle that while buildings may suffer damage during seismic events, they must not collapse, thereby ensuring the safety of occupants. Instead of designing solely for elastic behavior, modern structural engineering emphasizes controlled inelastic deformation through mechanisms like ductility, energy dissipation, and structural redundancy [6]. This approach ensures that buildings can withstand seismic energy without catastrophic failure.

The main goals of seismic design are:

1. To ensure life safety during major earthquakes,
2. To limit damage under moderate shaking, and
3. To maintain functionality of critical structures such as hospitals, fire stations, and emergency control centers.

Recent advancements also promote performance-based seismic design (PBSD), which evaluates a structure's response across multiple seismic hazard levels, aligning design choices with acceptable performance targets (FEMA, 2004) [6]. This is especially important in high-risk countries like Nepal, where ensuring the resilience of urban infrastructure is crucial.

Figure 1: Destruction of Hospital building in Myanmar due to earthquake

(Source: Xinhua News Agency)

Figure 2 : Soft-Storey failure of Olive View Hospital during 1971

(Source: P. Yanev)

The collapse of the hospital building shown in above two figures is a clear example of soft storey failure, a common and dangerous structural deficiency in seismic zones. In this case, the ground floor lacked adequate stiffness and lateral resistance due to large openings or fewer structural elements, making it significantly weaker than the upper stories.

During an earthquake, this stiffness irregularity leads to excessive deformation at the base, causing the ground floor to collapse while the upper floors remain relatively intact. Such failures are particularly catastrophic in hospital buildings, where structural integrity is not only essential for safety but also for continued post-earthquake operation. This highlights the critical need for detailed seismic analysis in hospital design. Hospitals are essential service structures that must remain functional during and after earthquakes. Seismic analysis ensures that both structural and non-structural components can withstand dynamic loading, minimizing damage and life-threatening disruptions. By identifying vulnerabilities like soft storey early in the design phase, engineers can implement proper lateral load-resisting systems, improve overall stability, and ensure the hospital's resilience during seismic events.

1.2. Objective of the project

The primary objective of the project is to analyse and design the earthquake resistant multi-storied RCC framed hospital building.

The specific objectives of the project are :

1. To define load cases and combination according to the NBC 105:2020 and IS 875(Part I).
2. To check the irregularities (plane and vertical) occurring in the building.
3. To determine drift and displacement of the building under seismic load.
4. To perform seismic detailing and standard design checks provided by the NBC 205 and NBC 105:2020.

1.3. Scope of the project

With increasing urbanization and lack of sufficient construction space in existing urban areas, there is shortage of free area to run to during earthquake, the trend towards high-rise construction is increasing and the risk of earthquake in a county like Nepal which has three major fault lines passing through it entails many problems and challenges so, to overcome this, an expert engineering knowledge in earthquake resistant design of any structure is absolutely essential [7].

The scope of the project, the activities it involves, can be summarized as:

1. Application of course of study in real field basically on structural design and analysis of RCC building.

2. Application of various commercial tools and software for designing seismic resistant structure.

3. Detail design of cost effective safe structural member such as slab column, staircase footing etc.

1.4. Significance of the project

The significance of the project are as follows:

1. It promotes earthquake resilience in critical public infrastructure.
2. It aids in designing a building that remains operational during emergencies.
3. It supports efficient evacuation planning and post-disaster response.
4. It encourages the use of standard codes (NBC, IS codes, etc.) in structural design.
5. It strengthens academic and professional knowledge in structural analysis and safety engineering.

1.5. Limitation of the project

The limitation of the projects are as follows:

1. The building design was carried out without considering soil investigation results .
2. The results depend on the software used, which may have some limitations in accuracy.
3. The drawing don't include service drawings like electrical, sanitary, water supply etc.

CHAPTER 2
LITERATURE REVIEW

2.1. Overview

Designing of every engineering structure follows certain principles with check in like strength and stability, serviceability, durability, economy, aesthetic appearance, sustainability, safety and compliance with relevant codes and regulations to ensure that the structure is safe, functional, and cost-effective and ultimately, all structures are designed to satisfy the various inadequacies and desires of human society,

The structural design of hospitals in seismic regions requires meticulous consideration of both safety and functionality. Hospitals must not only resist collapse during earthquakes but also continue to function to provide critical medical services. This literature review examines studies related to seismic design, ductility requirements, dynamic analysis, code compliance (particularly NBC 105:2020 and IS 456:2000), and design challenges in multi-storey RCC-framed hospital buildings.

2.2. Earthquake Design Principles for RCC Structures

According to **Agarwal and Shrikhande (2010)** [8] , earthquake-resistant structures should exhibit four essential characteristics: adequate strength, stiffness, ductility, and structural configuration. The performance of RCC buildings under seismic action is governed by the interaction of these factors.

Paulay and Priestley (1992) [6], argue that seismic energy must be dissipated through controlled inelastic deformations. They emphasize the importance of ductility and detailing, especially in critical buildings like hospitals.

Chopra (2017) [3], highlights the need for proper dynamic analysis for taller or irregular buildings, noting that equivalent static analysis is insufficient in many real-world cases.

2.3. Importance of Hospitals and Design Complexity

Hospitals are classified as essential facilities with an importance Factor (I) = 1.5 in both NBC 105:2020 and IS 1893:2016. These structures are expected to remain operational even after severe earthquakes. As noted by **FEMA P-750 (NEHRP Provisions)**[5]**,** failure of hospital buildings leads to double disasters: structural loss and loss of emergency services.

Jain and Sinha (2000) [9] ,stress that the post-disaster functionality of hospitals requires a dual design approach: structural safety and non-structural stability (e.g., equipment, power supply, emergency exits).

2.4. Seismic Codes and Local Practice in Nepal

NBC 105:2020 provides detailed procedures for seismic design, accounting for zone factor (Z), importance factor (I), soil type (S), and response reduction factor (R). It mandates ductile detailing for buildings in Zone 5, Pokhara.

A study by **Shrestha & Adhikari (2022)** [10], on post-earthquake performance of healthcare buildings in Nepal found that many older hospital buildings lacked ductile detailing, leading to structural damage and service interruption.**2.5 Dynamic Analysis for Irregular RCC Hospital Buildings**

Hospitals often have complex, non-uniform layouts (e.g., U-shaped, L-shaped, or plus-shaped plans). **Bhagat & Bhende (2019)** [11],note that such buildings require response spectrum analysis or time history analysis as per NBC 105, due to irregular mass and stiffness distribution.

A study by **Gautam et al. (2020)** [12], in the context of Nepal used ETABS to model a 10-storey RCC hospital in Pokhara. The authors found that buildings with vertical irregularities showed higher inter-storey drift and stress concentration at transfer levels unless shear walls and core walls were strategically placed.

2.5. Role of Shear Walls and Core Stability

According to **Ali and Moon (2007)** [13], shear walls are crucial in resisting lateral loads in high-rise hospital structures. When shear walls are placed symmetrically and continuous across floors, they significantly reduce torsional effects.

In seismic design of hospitals, shear walls are typically concentrated around:

1. Elevator cores
2. Staircases

A case study by **Khadka (2018)** [14], on a hospital in Kathmandu showed that shifting shear walls from periphery to internal core zones improved torsional stability by 18% under earthquake loading.

2.6. Use of Software Tools in Earthquake Design

Structural modeling software like **ETABS, SAP2000, and STAAD.Pro** are widely used for seismic design. These tools enable engineers to:

1. Model complex geometries
2. Apply dynamic loads (response spectrum, time history)
3. Track inter-storey drift, shear forces, moment diagram
4. Check compliance with ductility and load combinations

Shrestha & Sharma (2021) [15], conducted comparative analysis of a hospital structure using ETABS and STAAD.Pro. Their results showed similar displacement patterns, but ETABS provided more efficient workflow for complex geometry and load combinations.

CHAPTER 3
METHODOLOGY

The construction of the building involves careful consideration of various factors such as people, geographical requirements, safety, security, privacy and amenities. Architects and Engineers aim to create structures that combine functionality, aesthetics and efficiency to meet the specific needs and aspiration of each project.

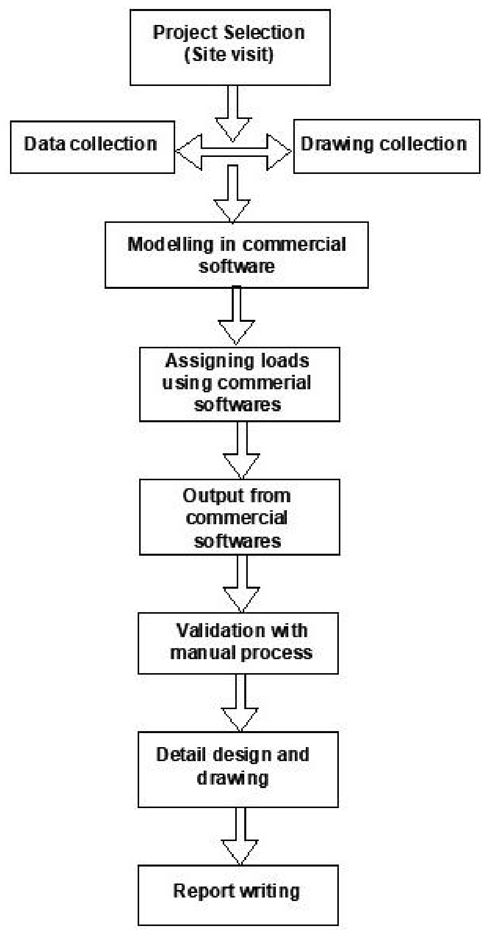


Figure 3 : Flowchart showing Methodology

This flowchart outlines the step-by-step process followed in a structural or civil engineering project, especially one involving structural analysis using software. This flow ensures a systematic approach, combining software efficiency with manual accuracy, resulting in a safe and well-documented design.

a) Project Selection (Site Visit):

The process begins with the selection of a suitable project, which is a critical initial step in the design workflow. A detailed site visit is conducted to gather firsthand information about the location, such as its geographical orientation, surrounding structures, soil conditions, access routes, and any environmental factors that may influence the design. Understanding the physical context of the site helps in making informed design decisions and ensures the proposed structure aligns with both functional needs and environmental considerations.

b) Data Collection & Drawing Compilation:

Once the site is selected, the next phase involves collecting all necessary data required for design and analysis. This includes measuring site dimensions, identifying the type and availability of construction materials, understanding climate-related factors. Accurate data collection at this stage lays the foundation for a realistic and reliable design model.

c) Modeling in Commercial Software:

After compiling the data, the structure is modeled digitally using commercial software,ETABS, which is widely used for structural analysis and design. [18] The model replicates the geometry, support conditions, and material properties of the building. This stage helps visualize the structural framework and provides a platform for applying various loads and performing simulations that mirror real-world behavior.

d) Assigning Loads Using Software:

Following modeling, the next step is to assign all relevant loads to the structure. These typically include dead loads (self-weight of the structure), live loads (occupancy loads), wind loads, and seismic loads, in accordance with national or international building codes. These loads are critical to evaluating how the structure will perform under different conditions and help identify the stress and strain distributions throughout the framework.

e) Output from Commercial Software:

Once all loads are applied, the software performs a comprehensive analysis of the model. It generates detailed results such as internal forces (bending moments, shear forces), displacements, support reactions, and required reinforcement. These outputs are essential for understanding the behavior of each structural component under the given loading conditions and for ensuring the structure meets safety and performance standards.

f) Validation with Manual Process:

To ensure the accuracy and reliability of the software-generated results, a manual validation process is carried out. This involves performing key hand calculations and cross-checking results using relevant building codes and engineering principles. This step acts as a verification mechanism, confirming that the automated analysis aligns with traditional design practices and ensuring that no critical errors are overlooked.

g) Detailed Design and Drawing Preparation:

Once the analysis results are validated, the detailed design phase begins. Structural components such as beams, columns, slabs, and staircase are dimensioned and reinforced according to the analysis. Construction-ready drawings are then prepared, showing all necessary design details, material specifications, and annotations. These drawings are crucial for guiding the construction process and for communicating the design intent to engineers and contractors.

h) Report Writing:

The final step involves compiling a comprehensive project report that documents the entire methodology. This report includes the design assumptions, data collected, software models, load applications, analysis outputs, validation checks, finalized designs, and drawings. It serves not only as a summary of the work completed but also as a reference for future projects or audits.

CHAPTER 4

FUNCTIONAL AND STRUCTURAL PLANNING OF THE BUILDING

1.1. Functional Planning

Functional planning of the building is governed by the client requirement, site conditions, municipal by-laws, etc. It is carried out in two steps in detail as below.

4.1.1. Planning of Space and Facilities

- The layout of the building plan was prepared and finalized as per the requirements of hospital authorities (clients) and Building by-laws 2072.
- For vertical mobility, two open welled staircases were provided.
- Two elevators were provided in the central part of the building for vertical movement.
- Shear wall was provided in the elevators.
- Washroom for ladies and gents are provided in each floor.

4.1.2. Architectural planning of 3D framework of Building

- The building to be designed is a multistory RCC Hospital Building.
- For reinforced concrete frames, a grid layout of beams is made considering the above functional variables. In all of grid intersection points, columns are placed.
- A total of 121 numbers of columns are provided.
- The overall dimension of the building is 40 m X 40 m.
- Arrangement of beams is done along the grid interconnecting the columns at grid intersections.
- The overall height of building is 33m including 3.2 m on each floor while 3m basement floor.
- The slab is provided of thickness 150 mm.
- With this framework of beam and column having RCC slab in the floor and roof, architectural planning of the building is complete and 3D framework is thus complete.

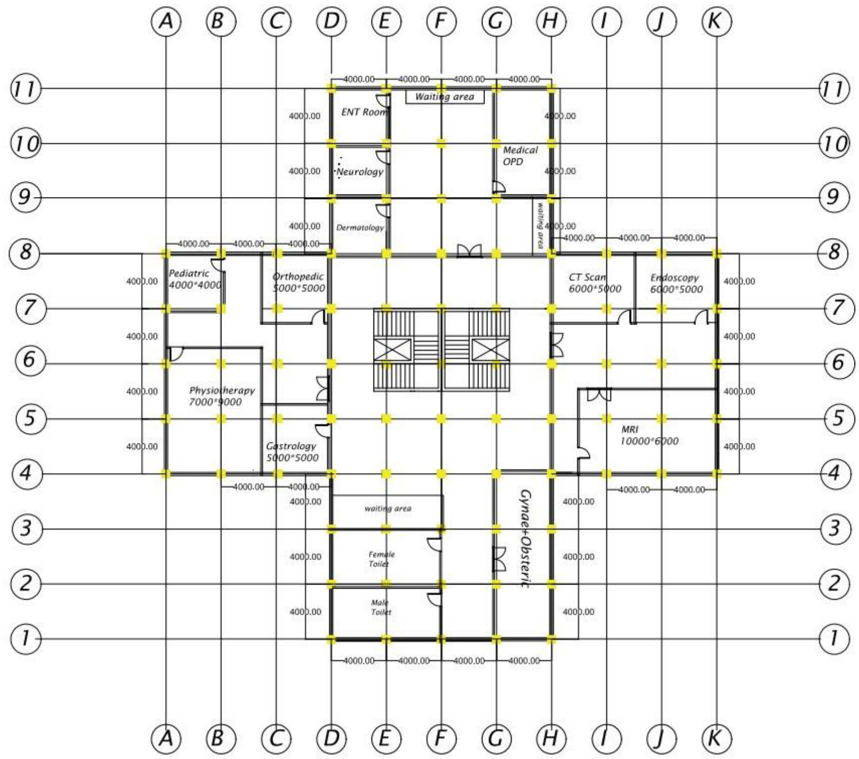


Figure 5: 1st floor architectural planning

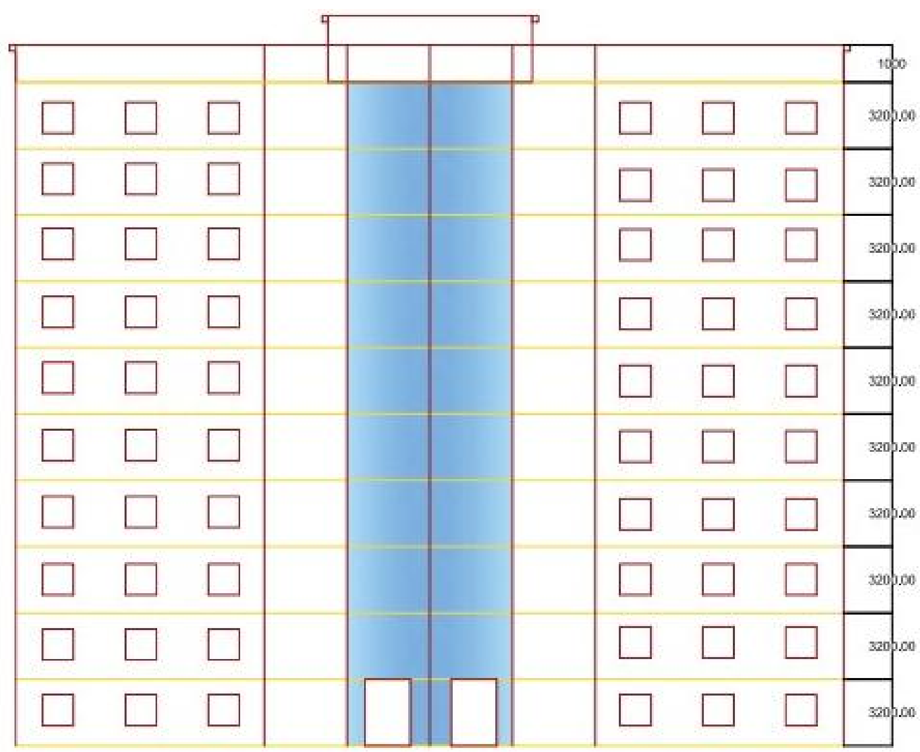


Figure 6: East elevation

4.1.3. Compliance to NBC

All the functional planning of building is done conforming to NBC 105:2020.

Specific points in the by-laws that need special focus of designer are:

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

These variables are also dictated by specific location of site in different wards.

Building height is a restricted by the position of widest road along the site .

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

4.2. Structural Planning

4.2.1. Structural System

- The building system is functionally and legally planned appropriately as mentioned in detail in previous section.
- Our focus in the current section is the structural orientation of the building in horizontal and vertical plane avoiding irregularities mentioned in IS 456:2000.
- The following types of irregularities mentioned in Table 4 & 5 of IS 1893 (part 1):2002 should be avoided as far as practicable during functional planning.

Table 1: Irregularities in structural system

Plan Irregularities		Vertical Irregularities	
1.	Torsional irregularity	1.	Stiffness irregularity -Soft storey
2.	Re-entrant corners	2.	Stiffness irregularity -Extreme soft storey
3.	Diaphragm discontinuity	3.	Mass irregularity
4.	Out of plane offset	4.	Vertical geometry irregularity
5.	Non-parallel systems	5.	In plane discontinuity in vertical elements resisting lateral force
		6.	Discontinuity in capacity-Weak storey

- The aim of design is the achievement of an acceptable probability that structures being designed will perform satisfactorily during their intended service life. With an appropriate degree of safety, they should sustain all the loads and deformations of normal construction and use and have adequate durability.
- Structural planning of the building is done over the proposed architectural plan for providing and preserving the structural integrity of the entire building. This is dealt in detail for each structural element with necessary justification.
- Finalized structural plan is then employed for load assessment and preliminary design of structural members for modeling in ETABS2020.

4.2.2. Building Summary

Location:	Pokhara-17 ,Birauta
Building Type:	Multi-storey
Structural System:	RCC Frame Structure
Purpose of Building:	Hospital Building
Plinth area covered:	944.423 m^2
Type of Foundation:	Mat Foundation
No. of Story:	Basement +Ground floor + 8 storey + staircase cover
Floor Height:	3.2 m (3m for Basement)
Total height of building:	33m + Basement floor (including stair cover)
Type of Sub-Soil:	Soft Soil (Type B)(fluvial gravel)
Seismic zone:	V
Width of walls:	230 mm (Main wall)
Partition wall:	115mm
Types of loads:	Dead, Live, Earthquake ,lateral loads
Analysis software:	ETABS v20, AutoCAD21
Beam size	300mm×550mm
Column Size:	550mm×550 mm
Depth of slab :	150 (overall)
Type of staircase:	Open well Dog legged
Grade of concrete:	M35
Grade of steel:	Fe 500
Lift wall dimension:	230 mm
Volume of water tank:	20000 litres
Height of parapet wall:	1m

4.2.3. Planning of Beam-Column Frame

- The numbers of beams and columns in the plan of building are obtained after careful planning of spaces to meet client requirements. Beams are provided of size 300mm × 550mm .
- Columns are 121 in number in each storey.
- Columns are of sizes of 550mm square cross section.
- The orientation of beams and column grid in plan is in square shape.
- Thus, the bare frame model of the building can now be created in ETABS2020 with the structural plan and elevation.
- The area element occupying the floor space is also modeled in the program. The image generated from ETABS2020 is shown below.
- Completion of Structural Planning is achieved with numeration of frames for identification of building elements in the course of design.

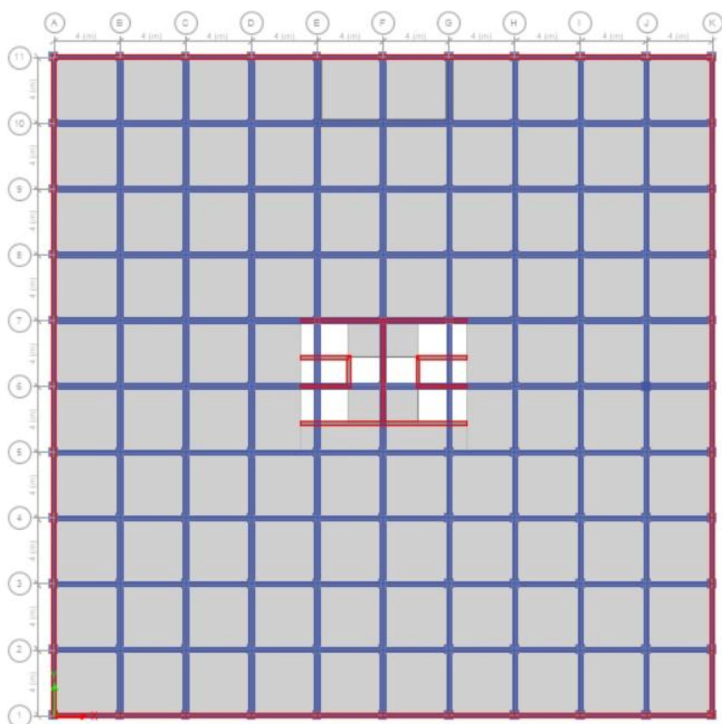


Figure 7: Ground floor plan extracted from Etabs

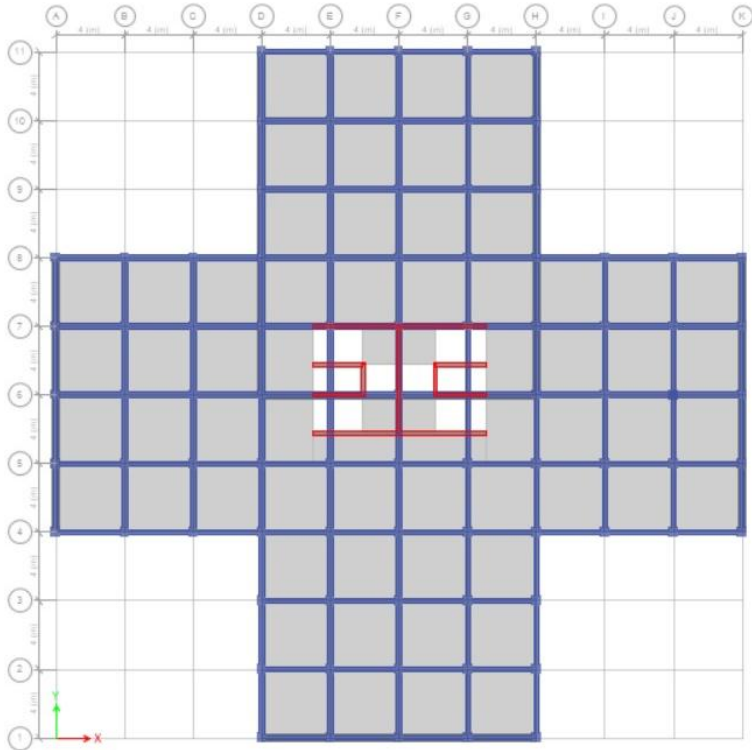


Figure 8: 1st to top floor plan extracted from Etabs

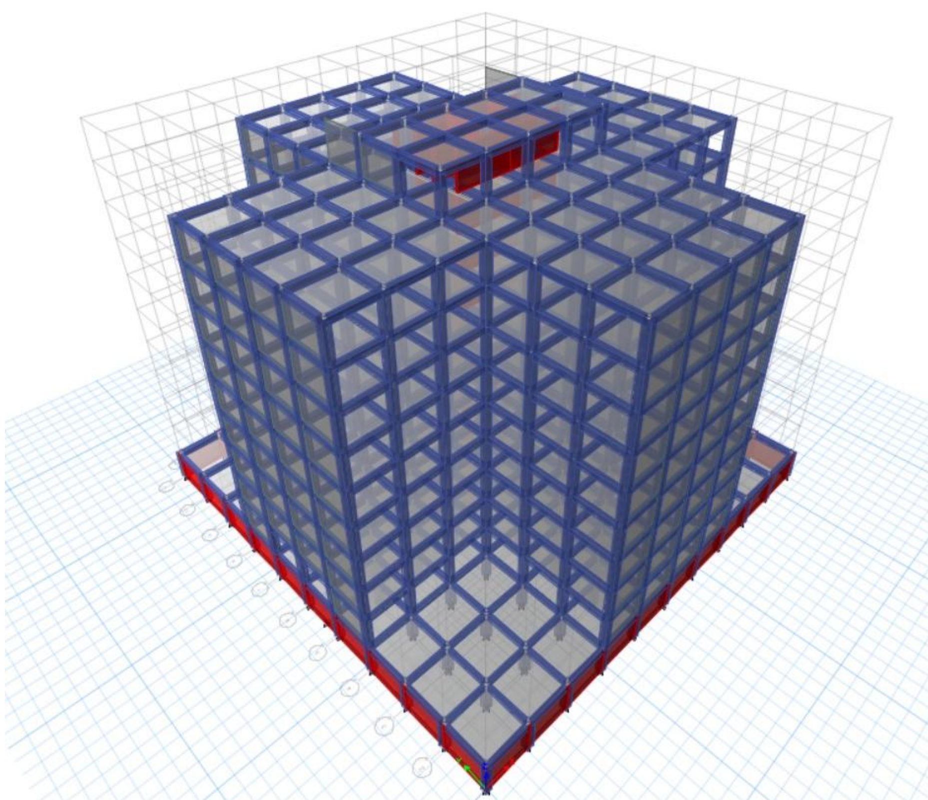


Figure 9: 3D modelling extracted from Etabs

CHAPTER 5
PRELIMINARY DESIGN

The preliminary design is carried out to estimate approximate size of the Structural members before analysis of structure. The preliminary sizing of structure elements was carried out based on the deflection control criteria and approximate loads obtained using the tributary area method.

The gravity loads on the structural members are taken as per IS 875-Part I (unit weight of materials) and IS 875-Part II (Imposed Loads). The unit weights of materials taken for the calculation of dead load of the structure are as tabulated below:

Table 2: Unit Weight of. materials

S.N	Materials	Unit Weight	Members loaded
1.	RCC	25 KN/m^3	Beams, Column, Slabs
2.	Brick (230×110×55mm)	18.85 KN/m^3	Outer & partition walls
3.	Granite	26.4 KN/m^3	All flooring
4.	Cement mortar / Plaster	20.4 KN/m^3	Wall and plastering

The imposed load on the floors and roof has been taken as per the IS 875-1987 Part II.

Here for residential building, the load Intensity will be as specified in code as under:

Table 3: Imposed Load as per occupancy

S.N	Materials	Intensity	Member loaded
1.	All rooms and kitchen	2 KN/m^2	Floor slab, Terrace floor and Staircase slab
2.	Toilet and Bath room	2 KN/m^2	
3.	Corridors, Passages, Staircase including fire escape and store room.	3 KN/m^2	

Calculation for Floor Finish load:

The floor finish generally done with cement mortar and tile, marble or granite. Here we take the floor finish with granite.

Thickness of screeding = 10 mm,

unit weight = 20 kN/m^3

So,

UDL intensity $=20\times0.01$

$=0.2\ kN/m^2$

Thickness of granite slab= 20 mm,

unit weight = 26.68 kN/m^3

So,

UDL intensity $=26.68\times0.02$

$= 0.5336\ kN/m^2$

Thickness of ceiling plaster = 12.5 mm,

unit weight = 20 kN/m^3

So,

UDL intensity $=20\times0.0125$

$= 0.25\ kN/m^2$

Total load = 0.2+0.5336+0.25 = 0.9836 kN/m^2,

Adopt, Floor Finish Load = 0.9836 kN/m^2

5.1. Preliminary Sizing of Slab

The projected building has largest span of 4 m × 4 m

lx = 4 m = 4000 mm, i.e, the smallest of the two dimension of the slab

ly = 4 m = 4000 mm, i.e, the largest of the two dimension of the slab

Then,

$\frac{lx}{ly} = \frac{4000}{4000} = 1 < 2$

Hence, the slab is a two-way continuous slab.

The depth of slab is obtained from IS 456-2000, Cl.no. 23.2.1 the deflection control criteria.

$$\frac{\text{Span(l)}}{\text{effective depth(d)}} \le \alpha\times\beta\ \times\gamma\times\lambda\times\delta$$

Here, l = ly = 4000 mm,

α = 26 for continuous slab

β = 1 for span below 10m

γ = 1.4 (assuming 0.2% reinforcement and fs=290 from fig.4 IS 456-2000)

λ = 1 for no compression steel

δ = 1 for no flanged beam,

So,

$$d_{eff} = \frac{lx}{(\alpha\times\beta\ \times\gamma\times\lambda\times\delta)}$$

=4000/ (26×1×1.4×1×1)

=109.89 mm ≈ 110 mm

Then, $\frac{l}{\text{deff}} = \frac{4000}{110} = 36.4$ NOT OK.

So, adopt d_{eff}=125 mm (≥For Earthquake Resistent Design)

So, Let us Adopt overall depth (D)= 150mm

Calculation of depth from Design Moment:

Live load on Slab, LL = 4 kN/m^2

Floor Finish Load, FF = 0.9836 kN/m^2

Dead load of slab, DL = 25×0.15 =3.75 kN/m^2

Factored load intensity, W_u= 1.5 (4 + 0.9836 + 3.75) = 13.1004 kN/m^2

From Table 26 (IS 456 2000), Our considering slab is case no. 1

i.e. adjacent edges are continuous

$\frac{lx}{ly} = \frac{4000}{4000}$ = 1, then from Table 26, IS 456:2000

Along shorter span,

Neg. Moment at Cont. edge (α_x-) =0.032

Pos. Moment at mid span: (α_x+) =0.024

Along longer span,

Neg. Moment at Cont. edge (α_x-) = 0.032

Pos. Moment at mid span: (α_x+) = 0.024

Calculating Moment

$Mu= \alpha_x.Wu.lx^2$

Here the maximum coefficient is 0.032 so, the maximum bending moment is,

$Mu = 0.032\times13.1004\times4^2 = 6.7074$ kN-m

We have,

$$d = \sqrt{\frac{Mu}{0.138\, fck.b}} \text{ for Fe500}$$

then, $d = \sqrt{\frac{6.7074\times106}{0.138\times35\times1000}} = 37.96 < 125$ mm, **OK**

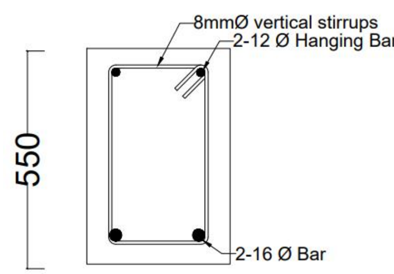
Hence Provide, Overall Depth of Slab (D) = 125 mm.

5.2. Preliminary Sizing of Beam

The flexural load on the beam is calculated by uniformly distributing the loads from the effective slab area and walls throughout the beam. The beam is analyzed as a continuous beam with maximum bending moment of $wl^2/12$.

As per IS 456-2000, cl. No 23.2.1 the vertical deflection limit may generally be assumed to satisfied,

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



Here, l = ly = 4000 mm,

$\alpha = 26$ for continuous slab

$\beta = 1$ for span below 10m

$\gamma = 0.7$ (assuming 2% reinforcement and fs=290 from fig.4 IS 456-2000)

$\lambda = 1.22$ (assuming compression reinforcement as 0.8%)

$\delta = 1$ for no flanged beam, so

$$d_{eff} = \frac{lx}{(\alpha\times\beta\ \times\gamma\times\lambda\times\delta)}$$

$=4000 / (26\times1\times0.7\times1.22\times1) =180.15$ mm

Consider, clear cover = 40 mm for longitudinal bar of dia. 20 mm

then, Overall depth of Beam, $D = d_{eff}$+ dia/2 + clear cover

$D = 180.15+(20/2)+40 = 230.14$ mm,

Adopt, Beam depth (D) = 550 mm

Taking b/d ration as 0.5, then

Breadth of beam, b = 550×0.5 =275 mm, Adopt 300 mm

Hence, Size of beam is 550 mm × 300 mm.

5.3. Preliminary Sizing of Column

For Column C65 (2307),

Considering 550 × 550 mm size of column for self-weight approximation.

Floor Height= Length of column = 3200 mm

For approximation, we assume the column is axially loaded.

L/d= 5.81 < 12 (short column)

Load calculation (as per IS 875 part I and II)

- Due to Slab = 25× 16 ×0.15 = 60 kN

  Take all slabs around the column to be of same dimensions.

  ∑ Slab= 4× 60 KN = 240 kN

- Due to Beam (300×550)

  → 4×(0.3×0.55×25)= 16.5 KN = 17 kN (say)

  Take all the beams around the column to be of same dimensions.

  ∑ Beam= 4×17 KN = 68 kN

- Due to Column

  Self- weight = 3.2×25×0.55×25 = 24.2 kN = 25 kN (say)

  Total load taken by the corresponding representative column in the top floor = $\frac{1}{4} * [\, 2 * \Sigma\ \text{Beam} + \Sigma\ \text{Slab}] + \text{Self weight}$

  = 93.5 kN

Then,

Total load on the lowermost column (P)= No. Of floors× 93.5 kN

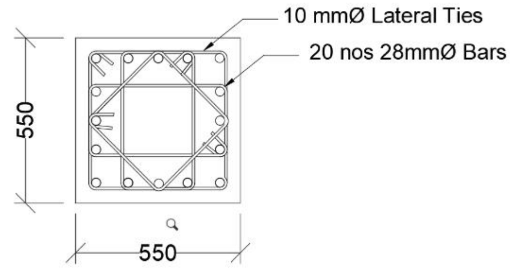
= 11× 93.5

= 1028.5 kN

$P_u = 1.5 \times P = 1542.75$ kN

- Live Load

  LL on influence area = 16×2 = 32 kN



  Reduction of Live Load as per IS 875- Part II, as

Table 4: Reduction factor for different floors

Floor	**Reduction Factor**	**Live Load(kN)**
Plinth	1	32
Ground Floor	0.9	28.8
First Floor	0.8	25.6
Second Floor	0.7	22.4
Third Floor	0.6	19.2
Fourth Floor	0.6	19.2
Fifth Floor	0.6	19.2
Sixth Floor	0.6	19.2
and upto Ninth Floor		
	Total	**243.2** kN

Roof Live Load = 2 × 16 = 32 kN

Hence total live load, LL = (243.2+32) = 275.2 kN

Therefore, Total factored load, $P_u = 1.5\,(DL + LL)$

$= 1.5\,(1542.75+275.2)$

$= 2726.925$ kN

Considering 25% for earthquake load = 1.25×2726.925 = 3408.656 kN

We have, $P_{uz} = 0.4 \times f_{ck} \times A_c + 0.67 \times f_y \times A_{sc}$

Or, $P_{uz} = 0.4 \times f_{ck} \times (A_g - A_{sc}) + 0.67 \times f_y \times A_{sc}$

Considering 1.5% of reinforcement,

$P_{uz} =$ 3408.656 kN

$f_{ck} =$ 35Mpa

$A_g =$ 198062.5218 mm^2

Asc = 2970.94 mm^2

fy = 500N/mm^2

So,Puz= 0.4×35×(198062.5218-2970.94)+0.67×500×2970.94=3726.55 KN

>3408.656 kN

OK

Hence, Provide Column of size 550 mm × 550 mm.

CHAPTER 6
IDEALIZATION OF STRUCTURE

6.1. Idealization of Structure

Idealization of the structure can be defined as the introduction of necessary constraints/restraints in the real structures as postulates to confirm the design of this structure within the domain of available theories assuring required degree of performance to some probabilistic measure.

The type of idealization helps us constrain infinite number of design variables to those that we can address properly with the available design philosophies. In design of RCC structures, chiefly two idealizations are employed namely: Idealization of Load and Idealization of Structure.

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

6.1.1. Idealization of Supports

In general, idealization of supports deals with the assessment of fixity of structure at the foundation level. In more detail terms, this idealization is adopted to assess the stiffness of soil bearing strata supporting the foundation. Although the stiffness of soil is finite in reality and elastic foundation design principles address this property to some extent, our adoption of rigid foundation overlooks it [16]. Elastic property of soil is addressed by parameters like Modulus of Elasticity, Modulus of Sub grade reaction etc. [17], addressing all these parameters are beyond the scope of this project. This is where idealization comes into play, equipping us with the simplified theory of rigid foundation in soil.

6.1.2 .Idealization of Slab

Idealization of slab element is done in earthquake resistant design to perform as a rigid floor diaphragm[18]. This idealization is done for the slab to behave as a thin shell element subjected to out-of-plane bending only under the action of gravity loads. Due to infinite in-plane stiffness of the shell element, lateral loads are not taken by the floor slab and hence resisted completely by the columns. Hence, such an idealized slab is then modeled in ETABS 20 program for analysis.

6.1.3 .Idealization of Staircase

Open welled staircase used in the building is idealized to behave as simply supported slabs, supported on beams at the floor and landing levels. This idealization helps us analyze the staircase slab in strips subjected to distributed loading on the landing strip and going of the slab.

Detailing rules are then followed to address the negative bending moment that are induced on the joint of going and top flight in the staircase, the rigorous analysis of which is beyond or scope.

Staircase being an area element is also assumed not to be a part of the integral load bearing frame structure. The loads from staircase are transferred to the supports as vertical reactions and moments.

6.1.4.Idealization of Beam and Column

Beam column idealization is one of the most critical aspects of structural idealization to achieve the desired behavior of the overall integrated structure.

Beams and columns are idealized to behave as linear elements in 3D. Beam column joints in the structural planning are assumed to behave as perfectly rigid joints[3]. In reality, perfectly rigid joints do not exist. Effects of partial fixity can be addressed in modeling by rigorous analysis of sectional and material properties, which is beyond the limits of this project.

Assumptions of rigid joints are also found to perform well in nature, seen from years of practice.

Main beam and secondary beam joints are idealized as hinged joints owing to the detailing adopted in such joints. Hinge beam assumption can have two impacts on structural behaviour of secondary beams. Firstly, lateral loads aren't transferred to the secondary beams from main beams and hence they can be idealized as flanged sections. Secondly, hinge connection at their extremities lets us address the partial fixity-of the beams in taking moments due to gravity loads.

Another idealization is addressing the section of main beam as rectangular in shape despite being integrally connected with the slabs. The flange portions of these beams when subjected to reversal of loading during earthquakes become ineffective in taking the tension induced in them and hence we ignore their contribution in design[19].

6.1.5 Idealization of Structural System

After idealizing individual elements, we idealize the structural system in its entirety to behave as our theoretical approximation for first order linear analysis and corresponding design.

The building is idealized as unbraced space frame. This 3D space framework is modeled in ETABS 2016 for analysis[20]. Loads are modeled into structure in several load cases and load combinations.

The building then, subjected to gravity and lateral loads are analyzed for necessary structural responses to design the members.

CHAPTER 7
ANALYSIS

After the preliminary sizing calculations, the next step is to analyze the building model. Analysis primarily involves calculating the various responses resulting from different loading conditions on the building members. This process helps evaluate the structural behavior and performance of the building under different loads and ensures that it meets the required safety and design criteria. The analysis manually is so tedious and have chance of high mistaken, so, For the purpose of seismic analysis of our building, we use the structural analysis program ETABS and seismic code IS 1893- Part-I. where modeling is done at first and after applying loads the analysis will run. We also calculate some data manually to verify result given by ETABS.

Before analyzing the building model, there should be necessary to define the various load pattern acting on the building. And also, the response of building in various load will be check through the concept of Load Combination as per IS 1893 – Part I.

7.1. Load Pattern

The various kinds of loads coming on building can be categories as dead load and live load, under this, individual components can be grouped and calculated them respective intensity that means load on the basis of lengthwise or area wise[21]. Generally, the loads found in building can be summarized as following: -

1. Dead Load Intensity

a) Wall load
b) Partition wall
c) Parapet wall
d) Floor finish load
e) Roof finishing
f) Water tank
g) Waist slab (due to steps)

h) Partition wall load on slab
i) Lift load (Machine load)
j) Curtain wall glazing

2. Live Load Intensity

a) Live Load on stairs = 5 kN/m^2
b) Live load on slab = 4 kN/m^2
c) Roof live load = 1 kN/m^2 (for maintainance / inaccessible)

The imposed load on the floors and roof has been taken as per the IS 875-1987 Part 2 for high rise hospital building

1. Dead Load Intensity

In these sections, we calculate load intensity for the various load patterns which area later used for analysis in ETABS. The various loads and their respective intensities are calculated as under:-

a. Wall Load

Full brick wall thickness = 0.23m

Half brick wall thickness = 0.115 m

Floor to floor height = 3.2m

Beam depth = 0.55m

Slab thickness = 0.15 m

Height of wall resting on beam = 3.2 - 0.55

= 2.650 m

Unit weight of brick masonry = 19.2 kN/m^3 *(IS 875 Part 1, 1987, Table 1 Item no. 36)*

Brick Properties: 230 mm × 115 mm × 55 mm

Beam Properties: 300 mm × 550 mm

Wall	WL (without opening) Unit : kN/m		25% opening	
	230 mm	115 mm	230 mm	115 mm
Resting on beam	0.23× 2.65 ×19.2 =11.70	0.115× 2.65×19.2 =5.85	0.75×11.7 = 8.775	1×0.115×19.2 = 2.208

b. Parapet wall Load

Assuming the size is 115 mm

Let, Height of parapet wall is = 1 m,

then,

UDL due to parapet wall = 1× 0.115 × 19.2

= 2.208 kN/m

Parapet wall Load = 2.208 kN/m

c. Floor Finish Load

This is already calculated in the preliminary design section.

Thickness of screeding = 10 mm

Unit weight of screeding =20 kN/m^3

Marble Flooring = 20 mm

Unit weight of marble flooring =26.68 kN/m^3

Thickness of plaster on ceiling =12.50 mm

Unit weight of plastering = 20 kN/m^3

So,

Total floor finish load = 0.01×20+0.02×26.68+0.0125×20

= 0.9836 kN/m^2

Floor Finish Load = 0.9836 kN/m^2

d. Top floor finish

Thickness of screeding = 50 mm

Unit weight of screeding = 20 kN/m^3

Thickness of plaster on ceiling = 12.50mm

Unit weight = 20kN/m^3

Floor finishing = 20 mm

Unit weight of flooring = 24 kN/m^3

Water proofing layer = 5 mm

Unit weight =0.1 kN/m^3

So,

Total = 0.005×20+0.0125×20+20×24+0.005×0.1

=1.73 KN/m^2

Top Finish Load = 1.73 kN/m^2

e. Water tank load

This load is here applied as shell load.

Slab Length = 4 m

Slab Breadth = 4 m

Slab area = 4×4

=16 m^2

Water Tank capacity = 10000 Ltr.

=100 kN

Water tank load = $\frac{100}{16}$

= 6.25 kN/m^2

Water Tank Load =6.25 kN/m^2

f. Waist slab

a) For 1^{st} and 3^{rd} flight

Riser (R) = 0.118 m

Tread (T) = 0.325m

Width of Stair = 2.00 m

Unit weight of concrete = 25 kN/m^3 (*IS: 875 (Part 1)- 1987 , Table – 1 Item no.22)*

Load on Waist slab =(0.118×0.325×25)/{2×$\sqrt{0.118^2 \times 2 + 0.325^2 \times 2}$

= 1.386 KN/m^2

Staircase step load =** 1.386* ***kN/m^2

b) For 2^{nd} and 3^{rd} flight

Riser (R) = 0.120 m

Tread (T) = 0.335 m

Width of stair = 2.000 m

Unit Weight of concrete = 25 KN/m^3 *(IS:875(PART 1)-1987, Table-1 Item no. 22)*

Load on Waist slab = $\frac{(0.12 \times 0.335 \times 25)}{2 \times \sqrt{0.12^2 + 0.335^2}}$ = 1.412 kN/m^2

Staircase step load =** 1.412* ***kN/m^2

g. Machine load for Lift

From (*IS code),*

According to our lift dimension and people capacity, we assumed the load to be 7 kN/m^2.

Machine Load for Lift = 7 kN/m^2

h. Partition wall load on slab

This is applying as shell load.

Slab length = 4m

Slab breadth = 4m

Wall thickness = 0.115m

Wall height = 3.2m

Wall length = 4m

Unit weight of brick masonry = 19.2 kN/m^2 *(IS:875(PART 1)-1987, Table-1 Item no. 36)*

Slab area = 4×4

= 16 m^2

And,

Wall area without opening = 0.115 × 4

=0.46 m^2

Wall area with 20% opening = 0.8 × 0.46

= 0.368 m^2

Load on slab without opening = (0.46 × 3.2 × 19.2)/16

= 1.766 kN/m^2

Load on slab with opening = (0.368 × 3.2 × 19.2)/16

= 1.413 kN/m^2

Partition wall load= 1.766 kN/m^2

i. Curtain wall glazing load

Unit weight of full aluminum framing and single glazing of 10mm= 0.75 kN/m^2

Height = 3.2m

So,

UDL on beam = 0.75 × 3.2

= 2.4 kN/m

Curtain wall glazing load = 2.4 kN/m

7.2. Load Combination

Load combinations consider the different types of loads acting on the structure and their various magnitudes and probabilities of occurrence. These combinations help engineers ensure that the building can withstand the most critical loading conditions and meet the required safety standards.

As per NBC 105:2020 following types of load combination are defined:

There are two categories according to NBC 105:2020 for parallel and non-parallel structure system. Since our building lies in parallel structural system. Hence, we use the following combination system: The parallel load combination are:

1. 1.0DL
2. 1.0LL
3. WL
4. 1.2DL + 1.5LL
5. 1.5DL + 1.5LL
6. DL + λ×LL + EQX SLS
7. DL + λ×LL + EQX ULS
8. DL + λ×LL + EQY SLS
9. DL + λ×LL + EQY ULS
10. DL + λ×LL + RSx
11. DL + λ×LL + RSy
12. DL + λ×LL - EQX SLS
13. DL + λ×LL - EQX ULS
14. DL + λ×LL - EQY SLS
15. DL + λ×LL - EQY ULS

Also, The load combinations in non-parallel structural system are as follows:

1. DL + λ×LL - (0.3EQX SLS + EQY SLS)
2. DL + λ×LL - (0.3EQX SLS - EQY SLS)
3. DL + λ×LL - (0.3EQX ULS + EQY ULS)
4. DL + λ×LL - (0.3EQX ULS - EQY ULS)
5. DL + λ×LL - (EQX SLS - 0.3EQY SLS)
6. DL + λ×LL - (EQX SLS + 0.3EQY SLS)
7. DL + λ×LL - (EQX ULS - 0.3EQY ULS)
8. DL + λ×LL - (EQX ULS + 0.3EQY ULS)
9. DL + λ×LL + (0.3EQX SLS - EQY SLS)
10. DL + λ×LL + (0.3EQX ULS + EQY ULS)
11. DL + λ×LL + (0.3EQX ULS - EQY ULS)
12. DL + λ×LL + (0.3RSx + RSy)
13. DL + λ×LL + (EQX SLS - 0.3EQY SLS)
14. DL + λ×LL + (EQX SLS + 0.3EQY SLS)
15. DL + λ×LL + (EQX ULS - 0.3EQY ULS)
16. DL + λ×LL + (EQX ULS + 0.3EQY ULS)
17. DL + λ×LL + (RSx + 0.3RSy)
18. DL+λ×LL-EQY SLSDL+λ×LL-EQY ULS
19. DL+λ×LL-EQY ULS

7.3. Response Spectrum

A response spectrum is a simple yet powerful graph used in earthquake engineering to estimate how different types of structures will react during an earthquake [3]. It shows the maximum acceleration, velocity, or displacement that a structure might experience based on its natural time period—essentially, how flexible or stiff it is [22].

Engineers use this graph to quickly predict how much shaking a building might endure during a quake, without needing to run full simulations each time. It's especially useful for comparing many structures with different characteristics using the same ground motion input. For complex buildings with many natural modes of vibration, engineers perform modal analysis and use the spectrum to get peak responses for each mode, which are then combined using standard methods like SRSS.[23]

The response spectra are mainly suited for linear structures. However, because they don't capture timing or phase differences in ground motion, the results are approximate. Still, due to their speed and reliability, response spectra remain a core tool in modern seismic design, as adopted by standards like NBC 105:2020.

Figure 10: Response Spectrum Curve for soil type "D"

Table 5: Base Shear Coefficient Calculation using NBC 105:2020 for Pokhara

For Hospital Building			
Zone Factor	Z=	0.3	Clause 4.1.4
Importance Factor	I=	1.5	Clause 4.1.5
	Soil type:	D	Clause 4.1.3.4
Height of Building	H=	33 m	
For Moment Resisting Concrete Frame	K_t=	0.075	Clause 5.1.2
Time Period	$T= 1.25*K_t*H^{0.75}$	1.291 sec	Clause 5.1.2 & 5.1.3
For Modal Response Spectrum Method	Ta=	0.5	Table 4-1
	Tc=	2	Table 4-1
	α=	2.25	Table 4-1
	K=	0.8	Table 4-1
Spectral Shape Factor	Ch(T)	2.25	Clause 4.1.2
Elastic Site Spectra	C(T)=Ch(T)*Z*I	1.0125	Clause 4.1.1
Elastic Site Spectra for serviceability limit state	Cs(T)=0.2*C(T)	0.2025	Clause 4.2
Horizontal Base Shear Coefficient			
Moment Resisting Frame Systems	**(i) For SLS**		Clause 5.4.2
	Ru=	4	Table 5-2
(Reinforced Concrete Moment Resisting Frame)	Ωu=	1.5	Table 5-2
	Ωs=	1.25	Table 5-2
	Cd(T1)= Cs(T1)/Ωs	**0.1620**	Clause 6.1.2

	(ii) For ULS		Clause 5.4.1
	Ru=	4	Table 5-2
	Ωu=	1.5	Table 5-2
	Ωs=	1.25	Table 5-2
	Cd(T1)= C(T1)/(Ru*Ωu)	**0.1688**	Clause 6.1.1
Building Height exponent	k=	1.65	Clause 6.3
Accidental Eccentricity	e=	0.1	Clause 5.7
Allowable story drift			
	(i) For ULS: 0.025/Ru	0.00625	Clause 8.1.3.1
	(ii) For SLS	0.006	Clause 8.1.3.1
Allowable story displacement			
	(i) For ULS: 0.025*(H/Ru)	206.2500 mm	Clause 5.6.1.1
	(ii) For SLS: 0.006*(H/Rs)	198.0000 mm	Clause 5.6.1.2

We calculated the base shear coefficient as 0.1688 (for ULS) and 0.1620 (for SLS) which implies a significant seismic demand on the structure, likely influenced by the site's seismicity, structural system, and soil characteristics. This value dictates that 16% of the building's seismic weight must be resisted laterally, ensuring the structure meets safety requirements under dynamic loading. In the response spectrum analysis, this base shear acts as a scaling benchmark, ensuring that modal combinations do not underestimate seismic forces and that the design remains code-compliant and structurally robust .

7.4. Lateral Load Distribution to stories

This is story response output for a specified range of stories and a selected load case or load combination. Auto lateral loads refer to the automated application of horizontal forces in structural engineering to assess the stability and performance of buildings or structures under lateral forces such as those from wind or earthquakes[3]. These loads are applied in a manner that simulates the effects of seismic activity or wind pressure on a building's joints and elements, often through computational methods and software tools. The main purpose of auto lateral loads is to evaluate how well a structure can resist these forces and ensure it meets safety and design standards[3]. This process can be visualized in a tabular format or through detailed load patterns that help engineers analyze the structural response under these simulated conditions.

Auto lateral loads in the X and Y directions are a critical aspect of structural analysis for buildings and other structures. These loads simulate horizontal forces acting on a structure due to environmental conditions such as wind and seismic events. In structural design, auto lateral load analysis involves applying forces along both the X (horizontal) and Y (vertical) axes to evaluate the structure's stability and resilience[3]. The X-direction typically represents lateral loads along the width of the building, while the Y-direction accounts for forces along the length. Engineers use these load patterns to create load combinations and assess how the structure will perform under different scenarios, ensuring safety and compliance with building codes. The process often involves defining load patterns, designating load types, and using software to apply these forces in a systematic manner to determine the structure's response to various loads[3].

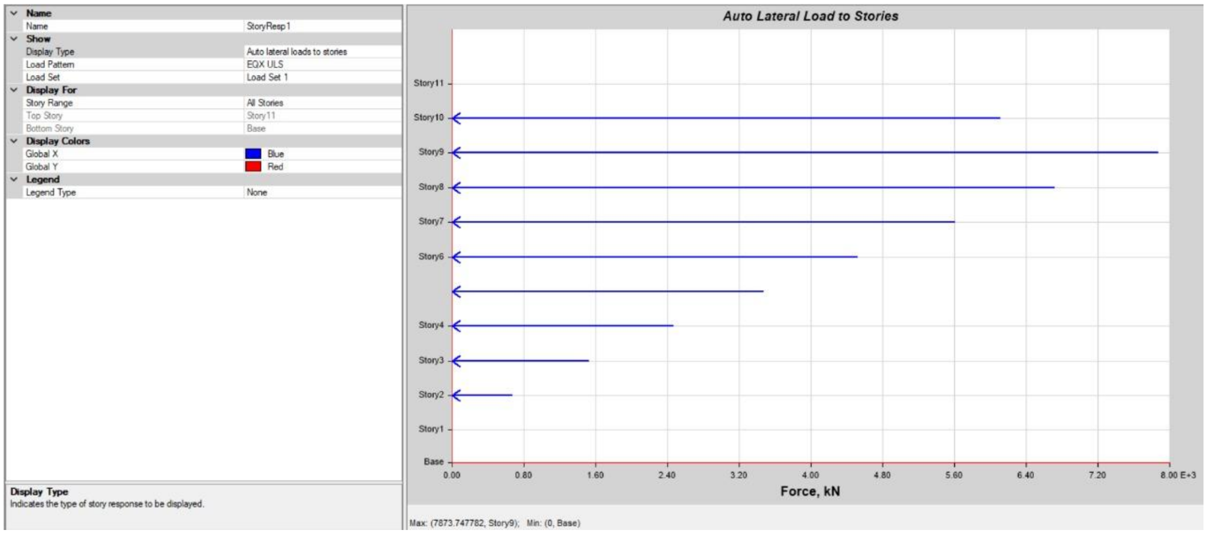


Figure 11: Auto Lateral Load to stories X direction (EQX ULS)

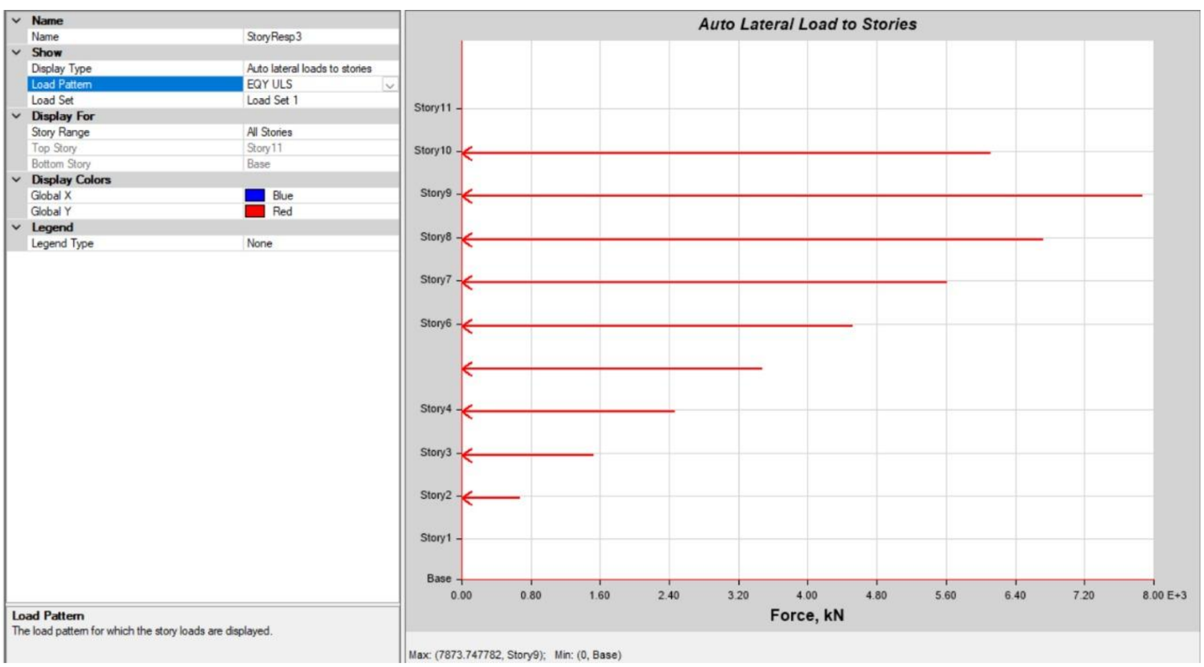


Figure 12: Auto Lateral Load to stories Y direction (EQX ULS)

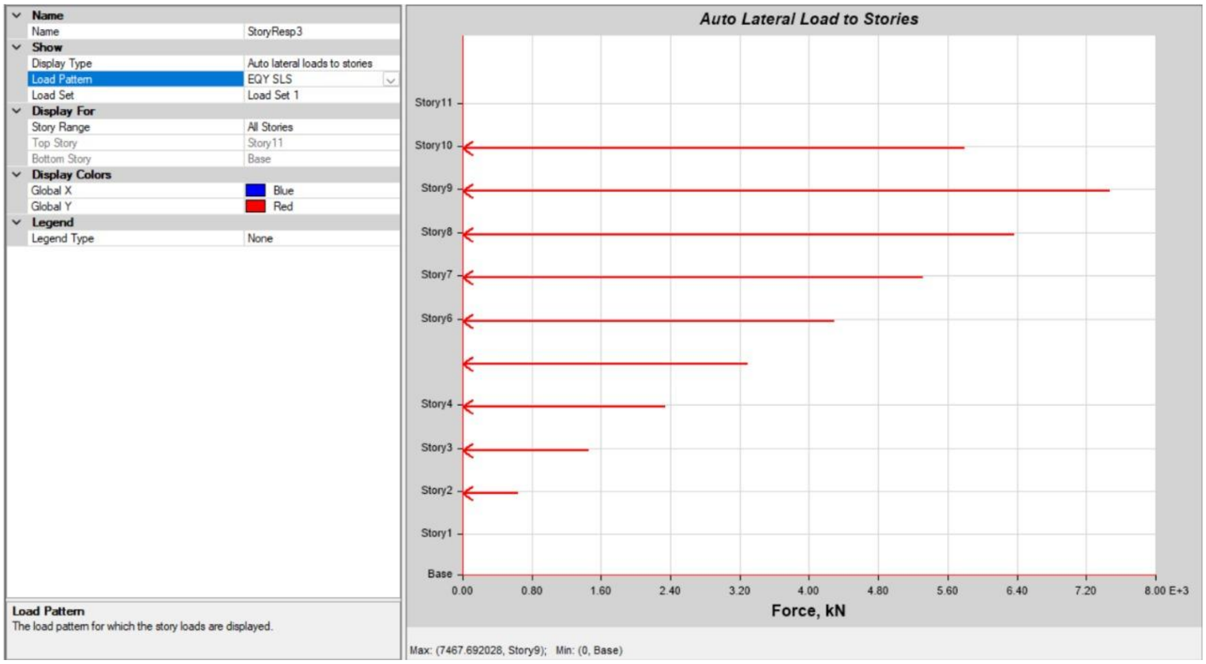


Figure 13: Auto Lateral Load to stories Y direction (EQX SLS)

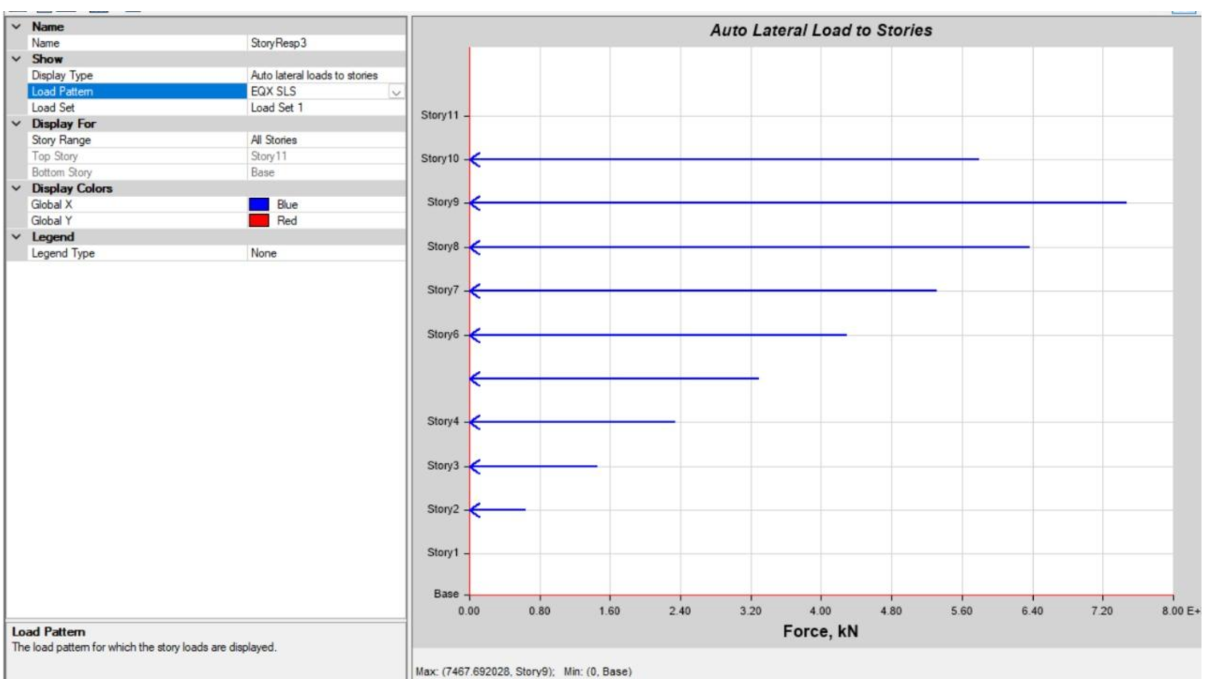


Figure 14 : Auto Lateral Load to stories x direction (EQX SLS)

7.5. Storey Drift check

As per Clause 6.4.3 of the Nepal National Building Code – NBC 105:2020,
inter-storey drift is defined as the relative horizontal displacement between two successive floors of a structure under lateral seismic loads, divided by the storey height.

To ensure adequate performance under earthquake loading and to prevent non-structural damage or collapse, the drift limits must be checked under both:

Serviceability Limit State (SLS) – for minor to moderate earthquakes

Ultimate Limit State (ULS) – for severe earthquakes (collapse prevention)

Inter-storey drift is computed as:

$$\delta=(\Delta i-\Delta i-1)/h$$

Where, Δi = Displacement at top of the i-th storey

Δ_{i-1}= Displacement at base of the i-th storey

h = Storey height (in mm)

This gives a dimensionless drift ratio, typically expressed in %.

From NBC 105:2020,

The drift limit of SLS limit state is given by :

Allowable displacement (δ) = 0.004×3200

= 12.8 mm

The drift limit of ULS limit state is given by :

Allowable displacement (δ) = 0.02×3200

= 60 mm

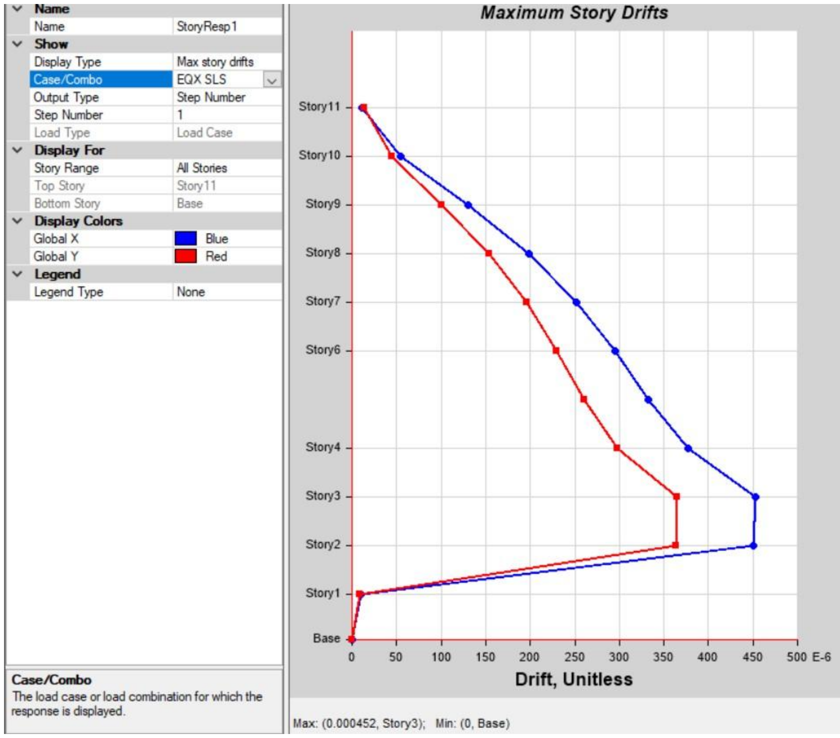


Figure 15: Maximum Storey Drift (EQX SLS)

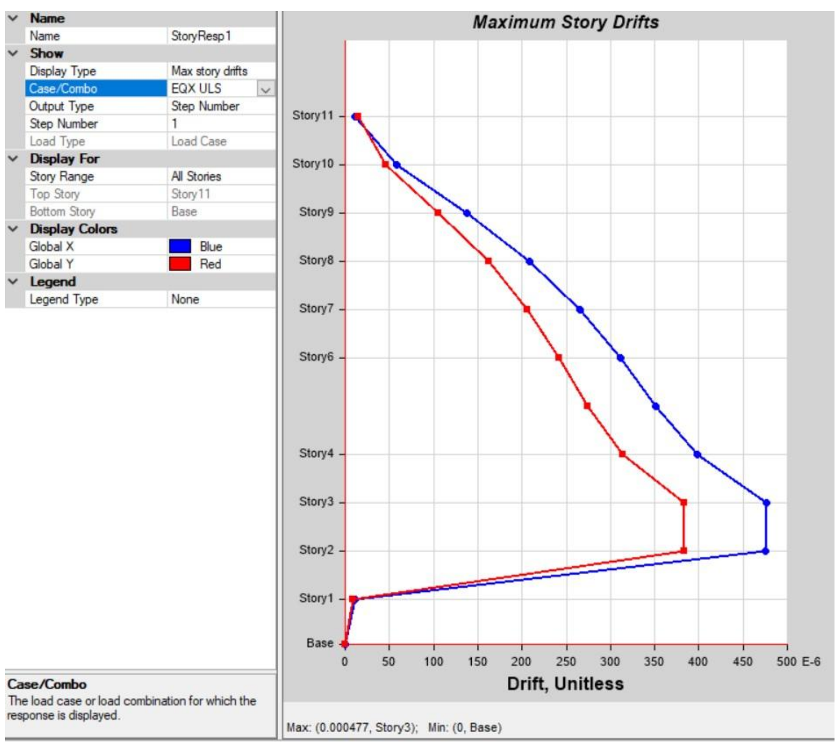


Figure 16: Maximum Storey Drift (EQX ULS)

7.6. Modal Participating Mass Ratio

Modal Participating Mass Ratio is a key concept in modal response spectrum analysis used in seismic design and structural dynamics. When a structure undergoes earthquake shaking, it vibrates in multiple natural modes, each with a specific natural frequency and mode shape.

The modal participating mass ratio represents the percentage of the total mass of the structure that effectively participates in the seismic response for a particular mode or combination of modes in a given direction (usually X or Y). In simpler terms, it quantifies how much of the building's mass is involved in motion during a specific vibration mode.

This ratio is important because it helps engineers determine the number of vibration modes to include in the analysis to capture the majority of the seismic response. Typically, the analysis continues until the combined modal participating mass ratios in each principal direction reach at least 90% of the total building mass, ensuring that the dynamic response is accurately represented [3].

By considering sufficient modal mass participation, engineers ensure that the calculated seismic forces and displacements reflect the real behavior of the structure, leading to safer and more efficient designs.

Table 6: Modal load Participation Ratio

TABLE: Modal Load Participation Ratios				
Case	ItemType	Item	Static	Dynamic
			%	%
Modal	Acceleration	UX	100	100
Modal	Acceleration	UY	100	100
Modal	Acceleration	UZ	0	0

Table 7: Modal Participation Mass Ratio

TABLE: Modal Participating Mass Ratios								
Case	Mode	Period	UX	UY	UZ	SumUX	SumUY	SumUZ
		sec						
Modal	1	0.662	0.024	0	0	0.024	0	0
Modal	2	0.247	0.0075	0	0	0.0315	0	0
Modal	3	0.167	0.0081	0	0	0.0397	0	0
Modal	4	0.152	0	0.6503	0	0.0397	0.6503	0
Modal	5	0.129	0.0225	0	0	0.0622	0.6503	0
Modal	6	0.12	0.6238	0	0	0.686	0.6503	0
Modal	7	0.105	0.0025	0	0	0.6885	0.6503	0
Modal	8	0.088	0.00001838	0	0	0.6885	0.6503	0
Modal	9	0.076	0.00001345	0	0	0.6885	0.6503	0
Modal	10	0.068	0.0001	0	0	0.6886	0.6503	0
Modal	11	0.063	0.00001234	0	0	0.6886	0.6503	0
Modal	12	0.063	0	0.0668	0	0.6886	0.7172	0
Modal	13	0.053	0.00004395	0	0	0.6887	0.7172	0
Modal	14	0.05	0	0.0159	0	0.6887	0.733	0
Modal	15	0.048	0.0672	0	0	0.7558	0.733	0
Modal	16	0.045	0	0.0181	0	0.7558	0.7512	0
Modal	17	0.04	0	0.0118	0	0.7558	0.763	0
Modal	18	0.037	0	0.0072	0	0.7558	0.7702	0
Modal	19	0.037	0.0184	0	0	0.7742	0.7702	0
Modal	20	0.035	0	0.0043	0	0.7742	0.7746	0
Modal	21	0.034	0	0.0023	0	0.7742	0.7769	0
Modal	22	0.033	0	0.0008	0	0.7742	0.7777	0
Modal	23	0.033	0.0119	0	0	0.7861	0.7777	0
Modal	24	0.032	0	0.000007423	0	0.7861	0.7777	0
Modal	25	0.029	0.0093	0	0	0.7954	0.7777	0
Modal	26	0.027	0.0072	0	0	0.8026	0.7777	0
Modal	27	0.025	0.0071	0	0	0.8096	0.7777	0
Modal	28	0.024	0.0217	0.000002322	0	0.8313	0.7777	0
Modal	29	0.024	0.0004	0.00001066	0	0.8317	0.7777	0
Modal	30	0.024	0	0.2223	0	0.8317	1	0
Modal	31	0.023	0.0004	0	0	0.8321	1	0
Modal	32	0.022	0	0	0	0.8321	1	0
Modal	33	0.02	0.1679	0	0	1	1	0
Modal	34	0.007	0	0	0	1	1	0
Modal	35	0.007	0	0	0	1	1	0
Modal	36	0.006	0	0	0	1	1	0
Modal	37	0.006	0	0	0	1	1	0
Modal	38	0.006	0	0	0	1	1	0
Modal	39	0.006	0	0	0	1	1	0
Modal	40	0.005	0	0	0	1	1	0

Table 8: Modal Periods and Frequencies

TABLE: Modal Periods And Frequencies					
Case	Mode	Period	Frequency	CircFreq	Eigenvalue
		sec	cyc/sec	rad/sec	rad^2/sec^2
Modal	1	0.662	1.511	9.4926	90.1087
Modal	2	0.247	4.051	25.4511	647.7583
Modal	3	0.167	5.998	37.6859	1420.2263
Modal	4	0.152	6.584	41.3702	1711.4895
Modal	5	0.129	7.736	48.6092	2362.8554
Modal	6	0.12	8.358	52.5159	2757.9233
Modal	7	0.105	9.514	59.7776	3573.3648
Modal	8	0.088	11.301	71.0032	5041.4507
Modal	9	0.076	13.075	82.154	6749.2784
Modal	10	0.068	14.655	92.0777	8478.3102
Modal	11	0.063	15.783	99.1672	9834.139
Modal	12	0.063	15.881	99.7802	9956.0916
Modal	13	0.053	18.933	118.9578	14150.9663
Modal	14	0.05	19.861	124.7916	15572.9522
Modal	15	0.048	20.818	130.802	17109.1666
Modal	16	0.045	22.365	140.5237	19746.9006
Modal	17	0.04	25.06	157.4573	24792.7987
Modal	18	0.037	27.044	169.9244	28874.3178
Modal	19	0.037	27.274	171.3688	29367.2541
Modal	20	0.035	28.541	179.3291	32158.9201
Modal	21	0.034	29.693	186.5663	34806.9709
Modal	22	0.033	30.501	191.6438	36727.3404
Modal	23	0.033	30.647	192.5614	37079.8773
Modal	24	0.032	31.126	195.5686	38247.0675
Modal	25	0.029	34.001	213.637	45640.7764
Modal	26	0.027	36.914	231.9385	53795.4875
Modal	27	0.025	39.361	247.3116	61163.0309
Modal	28	0.024	41.035	257.83	66476.2887
Modal	29	0.024	41.546	261.0425	68143.1729
Modal	30	0.024	41.912	263.3378	69346.82
Modal	31	0.023	42.877	269.4038	72578.3859
Modal	32	0.022	45.956	288.752	83377.7376
Modal	33	0.02	49.283	309.6545	95885.939
Modal	34	0.007	149.437	938.9401	881608.443
Modal	35	0.007	151.732	953.3592	908893.7568
Modal	36	0.006	163.359	1026.4176	1053533.14
Modal	37	0.006	170.586	1071.8225	1148803.544
Modal	38	0.006	175.885	1105.1206	1221291.471
Modal	39	0.006	177.032	1112.3246	1237265.91
Modal	40	0.005	192.899	1212.0184	1468988.537

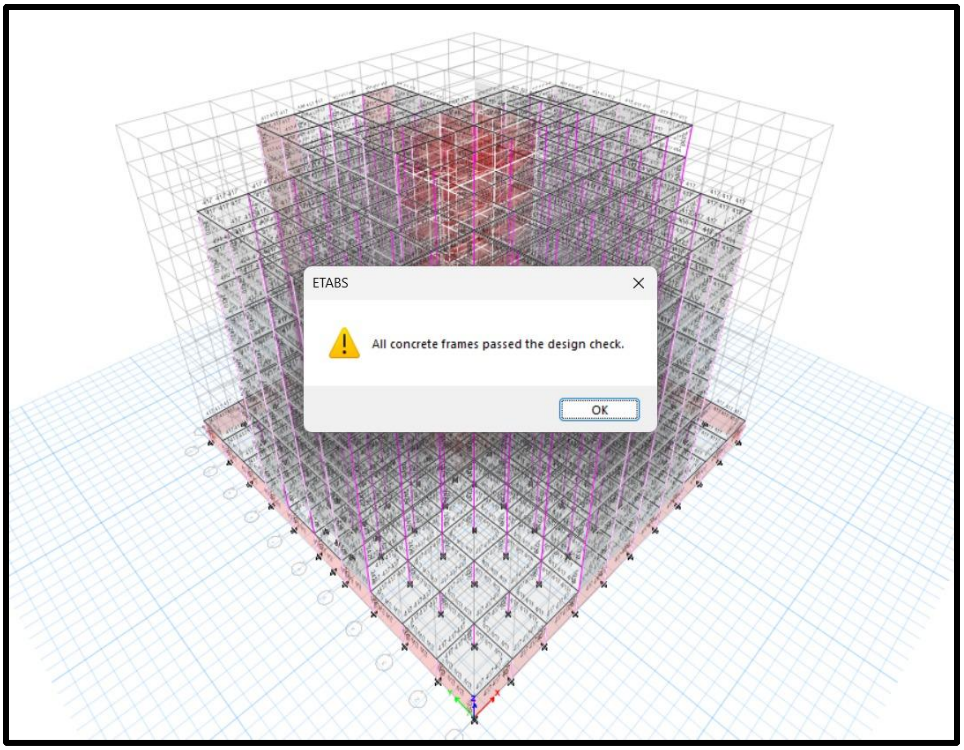


Figure 17: All concrete frames passed the design check

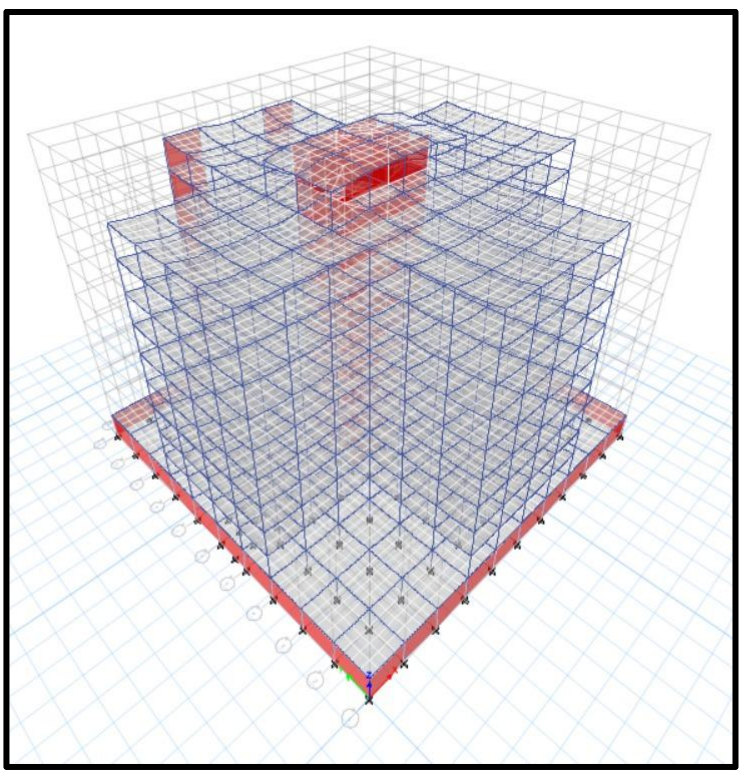


Figure 18: Deformed frame structure after analysis

7.7. Lump Mass Calculation

Since. lump mass is same for all the blocks so a single calculation is done for all.

Column : $\gamma\times h\times b\times d$

$=25\times3.2\times0.55\times0.55$

$=24.2$ kN

Since we have 121 columns

i .e total lump mass $=121\times24.2=2928.2$ kN

No of beam on each floor =148

Beam : $\gamma\times L\times B\times D$

$=25\times148\times0.55\times0.3$

$=606.375$ kN

No of slab in each floor =64

Slab : $\gamma\times L\times B\times D$

$=25\times0.150\times4\times4$

$=60\times64$

$=3840$ kN

Live load : 50% of LL(stairs+ward)

$=256\times5+4\times192\times3$

$=3584\times0.5$

$=1792$kN

F. F :$1\times(256+4\times192)$

$=1024$kN

Masonry : $\gamma\times t_w\times h_w\times$60%of opening $\times448$

$=19\times0.23\times3.2\times0.6\times448$

$=3758$kN

Hence total lump mass on each floor = 13948,575kN

On top floor

Column : $\gamma \times h \times b \times d$

$=25 \times 121 \times 0.55 \times 0.55 \times 1.6$

$=1464.1$kN

F.F= 1024 kN

Total =2488.1 kN

Hence,

Total lump mass of the building =141973.85 kN

CHAPTER 8

DESIGN AND DETAILING OF STRUCTURE

8.1: Design of Beam

Known data:

Load from beam= 11.70km/m

Load from slab= 11.70 KN/m

Floor finish =0.9836 KN/m

Live Load =4 KN/m

Overall depth(D) = 550mm

Effective cover = 70mm

Effective depth d = 550-70=480 mm

Width of beam: 300 mm

Grade of concrete = M35

Grade of steel (fy) = 500 N/m

Effective length: 3.77 m

Clear span =3.54 m

Step 1 :Load calculation

self wt. of beam= ϒbD

$=25\times0.3\times0.55$

$= 4.125$ KN/m

Total Load $=4.125+11.70+0.9836+4$

$= 45.8086$ KN/m

Factored Load $=1.5\times45.8066 =68.7129$KN/m

Step 2: Calculation of bending moment

$M_u = (w \times l_{eff}^2)/8$

$= (68.7129 \times 3.77 \times 3.77)/8$

$M_u = 448.277$ KNm

Step 3: Calculation of reinforcement

$M_{u1\ lim} = 0.36\ Fck\ b\ Xm\ (d - 0.42Xm)$

$= 0.36 \times 35 \times 300 \times 0.46 \times 4805(480 - 0.42 \times 0.46 \times 497.5)$

$= 333.8496$KNm

Since,

$Mu > Mu_1$Limit,

So, we have to designs for doubly reinforcement bar .

Step 4: Calculation of Tension area.

$Mu = 0.87 \times Fy \times Ast_1 \times (d - 0.42x_m)$

$333.84 \times 10^6 = 0.87 \times 500 \times Ast(480 - 0.42x0.46x480)$

$Ast_1 = 1981.72 mm^2$

$M_2 = M_{u-} Mu_{lim}$

$= 448.277 - 333.8496$

$= 144.4274$kNm

$Ast_2 = \frac{M_2}{0.87 xfy (480-70)}$

$= 641.59 mm^2$

$Ast = 1981.72 + 641.59 = 2623.31 mm^2$

Provide dia of 2-25mmϕ and 6-20mmϕ

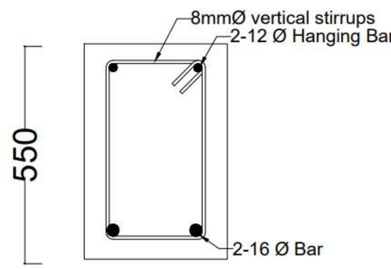
Ast provide$= 2x\frac{\pi x 25^2}{4} + 6x\frac{\pi x 20^2}{4} = 2866.70 mm^2$

For Fe500 $\frac{d'}{d}=\frac{70}{480}=0.145$

Fsc=396.36Mpa

Fcc=0.446fck

Area of compressive bar



Asc=$\frac{M_2}{(fsc\text{-}fcc)(d\text{-}d')}$

$=\frac{114,427x10^6}{(396.36\text{-}15.61)(480\text{-}70)}$

$=733\,mm^2$

Provide 4-16mm bars

Asc provide=4x$\frac{\pi x16^2}{4}$

$=804.24\,mm^2$

Check for the deflection

Pt % steel =$\frac{100Astprovide}{bd}$

$=\frac{100\ 2866.7}{300x480}$

=1.99%

Psc=$\frac{100Ascprovide}{bd}$

$=\frac{100x804.24}{300x480}$

=0.55%

For no of flanged section ,

factork_1=1

for 1 fs =0.58×fy×$\frac{Ast\ required}{Ast\ provided}$

$=0.58\times500\times\frac{2623.31}{2866.70}$

=265.38 Mpa

Pt =1.99%

From fig 4,

k_2=0.88

Pc =0.55%

k_3 =1.17 from fig 5

$(\frac{span}{depth})$ max = $k_1\times$ $k_2\times$ k_3 ×basic value

= 1× 0.88× 1.17 ×20

=20.592

$(\frac{span}{depth})$ provided $=\frac{3770}{480}$ =7.85

Hence,

$(\frac{span}{depth})$ max > $(\frac{span}{depth})$ provided (**so the design is okay)**

Design of shear force

$Vu = \frac{w\times l_{eff}}{2}$

$=\frac{68.7129\times3.77}{2}$

=129.52kN

$\tau_u=\frac{Vu}{bd}$

$=\frac{129.52\times10^3}{300\times480}$

=0.89N/mm^2

From table 20,

τ_{cmax} = 3.7N/mm^2 (M 35)

$\tau_{cmax} > \tau_u$ **ok**

Ast at support, $6 \times \frac{\pi x 20^2}{4} = 1884.95 mm^2$

Pt $= \frac{100x1884.95}{300x480}$ $= 1.3\%$

From table 19 M35,

$\tau_c = 0.74$N/mm2

$\tau_u > \tau_c$

So, shear reinforcement is to be provide

Vus =Vu-ꞇ cxbd

=$129.52 \times 10^3 - 0.74 \times 300 \times 480$

=22.96kN

Using legged dia 8mm vertical strriups

Asv=2x$\frac{\pi x 8^2}{4} = 100.5 mm^2$

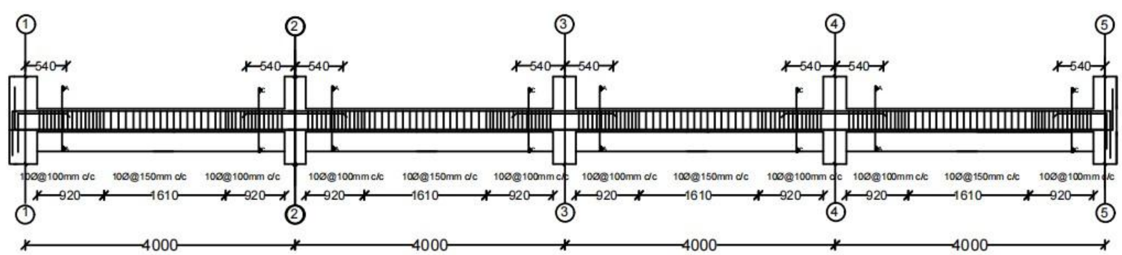


Figure 19: Reinforcement detailing of Beam section of Floor 1 to floor 10

Spacing of strriups

Sv=$\frac{0.87 fyAsvxd}{Vus}$

$= \frac{0.87x500x100.5x480}{22.96x10000}$

=332mm

Maximum spacing of strriups

Sv=$\frac{0.87 fyAsvxd}{0.4 b}$

=364,3125mm

The max spacing should be less of

i)0.75d=360mm

ii)300mm

provide 2 legged 8mm strriup @300mm c/c

$L_d = \frac{0.87 f_y \emptyset}{4 \tau_{bd}}$=988,63 OK

Also, Detailing of Beam is attached in Annex no. 3.

8.2: Design of Column

Here, For Column C65 2307,

Dimension of column = 550 × 550 mm

Length of the column(L)=3200 mm

Thickness of slab =150 mm

Unsupported length of the column (l) = 3200-150

= 3050 mm

Effective length of the column (l_{eff}) = 0.8 × 3050

= 2440 mm

F_{ck} = 35 N/mm^2

F_y = 500 N/mm^2

Pu =5503.95 KN (Maximum axial load)

Step 1 : Check whether the column is long or short:

$$\frac{\text{leff}}{D} = \frac{2400}{550} = 4.13 < 12$$

∴ It is a short column.

Step 2 : Check for eccentricity:

From ***IS 456: 2000, Cl. 25.4,***

$$e_{min} \frac{L}{500} + \frac{D}{30} =$$

$$= \frac{3050}{500} + \frac{550}{30}$$

$$= 24.93 \text{ mm} \geq 20 \text{ mm} \qquad (\text{ OK })$$

Adopt 24.43mm eccentricity.

$$\frac{\text{emin}}{D} = \frac{24.93}{550} = 0.05 \qquad (\text{ OK })$$

Step 3 : Calculation of Moment Due to minimum eccentricity:

Moment due to minimum eccentricity ($Mu_{ex} = Mu_{ey}$) = $P_u \times e_{min}$

$= 5503 \times 0.02493$

$= 137.21$ kNm

The design moment is greater of given moment and moment due to eccentricity.

Design moment are :

$Mu_x = 137.21$ kNm $= Mu_y$

Step 4: Column Reinforcement design :

Assuming reinforcement is distributed equally on four

sides,

Clear cover(cc) =50mm

Diameter of reinforcing bar (ø) = 25mm

Then, Effective cover (d') = cc + $\frac{ø}{2}$

$$= 50 + \frac{25}{2}$$

$$= 62.5 \text{ mm}$$

Let us assume percentage of reinforcement (P)= 3.5%

$$\frac{d'}{D} = \frac{62.5}{550} = 0.112$$

$$\frac{p}{fck} = \frac{3.5}{35} = 0.1 D^2$$

$$\frac{Pu}{fck\ b\ d} = \frac{5503.95 \times 1000}{35 \times 550 \times 550} = 0.519$$

From ***IS 456: 2000, Chart 48 and 49,***

By interpolation, we get,

$$\frac{Mux1}{fck\ b\ D\text{^}2} = \frac{Muy1}{fck\ b\ D\text{^}2} = 0.115$$

Mux1 = Muy1 = 669.66 KNmm

From ***IS 456: 2000, Clause 39.6,***

P_{uz}=0.45 f_{ck} A_c + 0.75 $f_y A_{sc}$

=0.45 × 35 × (97.5% of 550 × 550)+0.75 × 500 × (3.5% of 550 × 550)

= 8615.58 kN

$\frac{Pu}{Puz} = \frac{550.395}{8615.58} = 0.63$

By interpolation, α_n = 1.72

$$\left[\frac{M_{ux}}{M_{ux1}}\right]^{\alpha_s} + \left[\frac{M_{uy}}{M_{uy1}}\right]^{\alpha_s} \leq 1$$

Or, $\left[\frac{137.2}{669.66}\right]^{1.72} + \left[\frac{137.2}{669.66}\right]^{1.72} \leq 1$

∴ 0.13 ≤ 1 which is true .

(OK)

Step 4: Longitutional Reinforcement design :

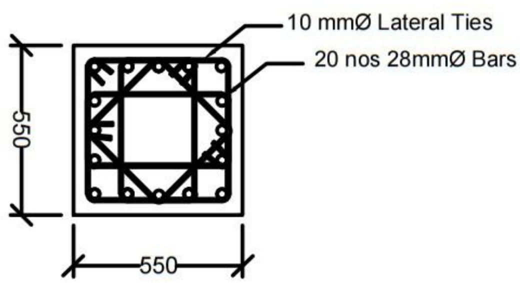
Area of reinforcement A_{sc} = 3.5% of 550 × 550

= 10587.5 mm^2

$A_{st,min}$ = 0.8% of bD

=0.8 % × 550×550

= 2420 mm^2 < A_{sc} (OK)



Adopt Ø 28mm bar

No. Of bars = $\frac{10587.5}{\frac{\pi \times 28 \times 28}{4}} = 17.19 \sim 20$ nos

$\frac{1}{4}$× Diameter of longitudinal bar = $\frac{1}{4}$× 28 =7mm ~ 6mm

Here, Provide 10mm diameter of lateral tie bar.

For spacing,

From ***(IS 456 : 2000, Clause 26.5.3.2)***

a) 16 × Diameter of longitutional bar = 16 × 28 = 448 mm

b) 300mm

Hence, Adopt spacing of 300mm c/c at support

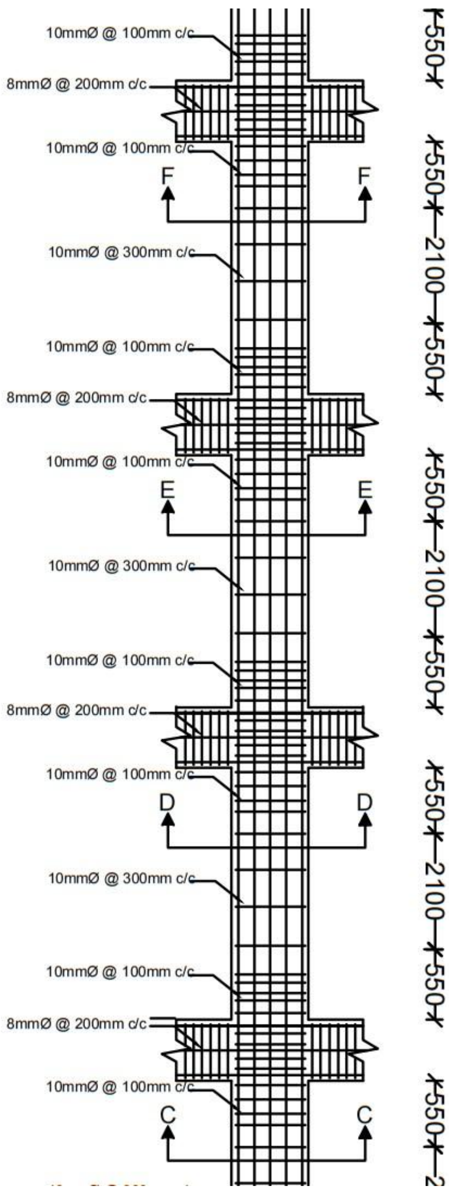


Figure 20: Reinforcement detailing of column section from Floor 1 to Floor 4

Also, The complete detailing of Column of all floors is attached in Annex no. 3.

8.3: Design of Slab

Here, For Slab section S792 F85

Depth of the beam = 550 mm
Width of the beam = 300 mm
Assumed overall depth of slab = 150 mm
Dimensions of the selected Slab panel :
Lx = 4000 mm,
Ly = 4000mm
Dead Load (i.e. Self-weight of RCC) is taken as 25 kN/m^3 *(IS 875 Part 1)*
Live (or Imposed) Load that act on the slab is taken as 4 kN/m^2 *(NBC 103:1994)*

From preliminary design, two way slab design is done.

Step 1: Determination of depth or thickness of slab

The thickness of slab is assumed from deflection control criteria for further design. From ***IS 456: 2000, Clause 23.2.1***
$\alpha = 26$
$\beta = 1$
$\gamma = 1.4$ (supposed)
$\delta = 1$
$\theta = 1$

$$\frac{Span}{d} = \alpha \times \beta \times \gamma \times \delta \times \theta$$

$$or, \text{d} = \frac{4000}{26 \times 1.4}$$

$$\therefore d = \frac{4000}{36.4} = 109.89 \sim 110\text{mm}$$

Also, For Earthquake resistant design D ≥ 125 mm

Let us assume overall Depth (D) = 150 mm

Provided nominal cover 15 mm ***(Table 16 IS 456:2000).***
Using 10 mm diameter rebar.
Therefore**,** d = 150 – 15 – 10/2 = 130 mm

Step 2: Calculation of actual effective span

Clear span = 4000 - $\frac{115}{d2}$ **-** $\frac{115}{d2} = 3885\text{mm}$

Along x direction,

a) $(l\,x)_{eff}$ = 4000 mm
b) $(l\,x)_{eff}$ = 3885 + 143 = 4028mm

Take smaller value,

$(l\,x)_{eff}$ = 4000 mm

Similarly, $(l\,y)_{eff}$ = 4000 mm

Step 3: Load Calculation

Self weight = 25 × 0.15 × 1
= 3.75 KN/m
LL = 4 × 1
= 4 KN/m
Floor finish = 0.9836 × 1
= 0.9836 4 KN/m
Partition wall load on slab = 1.766 4 KN/m

Total load = 10.4996 4 KN/m

Total Factored load = 1.5 × 10.4996 = 15.75 kN/m

Step 4: Design Bending Moment Calculation

The slab is rigidly supported (monolithic casting of concrete) by beam on four sides.

The moment coefficients are taken from ***Table 26 (IS 456:2000)***.

It is the interior panel so case 1 is taken.

We have, $\frac{ly}{lx} = 1$

From table, we have :

$$\alpha^+{}_x = 0.024\,, \quad \alpha^-{}_x = 0.032$$

$$\alpha^+{}_y = 0.024\,, \quad \alpha^-{}_y = 0.032$$

Hence,

Positive moment in midspan $(M^+{}_x)\ and\ (M^+{}_y) = \alpha^+{}_x w\, lx^2$

$$= 0.024 \times 15.75 \times 1 \times 4^2$$
$$= 6.048\ kNm$$

Negative moment at support $(M^-{}_x)\ and\ (M^-{}_y) = \alpha^-{}_x wlx^2$

$$= 0.032 \times 15.75 \times 4^2$$

$$= 8.064\ kNm$$

Step 5: Check of effective depth

For Fe 500, $X_m = 0.46 \times d$

$(Mx)_{max} = 0.36$ fck b X_m u $(d-0.42\ X_m)$

or, $8.064 \times 10^6 = 0.36 \times 35 \times 1000 \times 0.46d\ (d-0.42 \times 0.46d)$

On solving,

$D_{required} = 41.52$ mm $< d_{adopted}$

(OK)

Step 6: Calculation of tensional Area

We have from ***(IS 456 : 2000, Annex G 1.1)***

$$M_u = 0.87 f_y A_{st}(d - \frac{f_y A_{st}}{f_{ck} b})$$

$$\text{or, } M_u = 0.87 \times 500 \times A_{st}(130 - \frac{500 \times A_{st}}{35 \times 1000})$$

For $M^+{}_x = 6.048\ kNm$,

fck = 35 N/mm^2,

fy = 500 N/mm^2,

we get

$Ast^+{}_x = 108.23$ mm^2

Similarly, Respective Area Calculation

$Ast^+{}_x = Ast^+{}_y = 108.23$ mm^2

$Ast^-{}_x = Ast^-{}_y = 144.90$ mm^2

Now,

$$\text{Spacing (S)} = \frac{1000}{\text{Ast} \times 4} \times \pi \times \emptyset^2$$

$$= \frac{1000}{\text{Ast} \times 4} \times \pi \times 10^2$$

Respective Spacing Calculation

$$S^+_x = S^+_y = 725.67 \text{ mm}$$

$$S^-_x = S^-_y = 542.02 \text{ mm}$$

From ***(IS 456 : 2000, Clause 26.3.3)***

Spacing should be smaller than

a) 3d = 3 x 130 = 393 mm.

b) It should be less than 300 mm.

Hence, Adopt spacing of 200mm c/c at support

Provide 10mm φ bars @ 200 mm c/c at support along both span.

Hence,

$$\text{Ast}_{\text{provided}} = \frac{1000}{200} \times \pi \times 10^2 = 392.69 \, mm^2$$

Check for Ast,

$(\text{Ast})_{min} = 0.12 \text{ \% of bD}$

$= (0.12\%) \times 1000 \times 150$

$= 180 \text{ mm}^2$

$(> \text{Ast}_{\text{required}})$

$(< \text{Ast}_{\text{provided}})$ (OK)

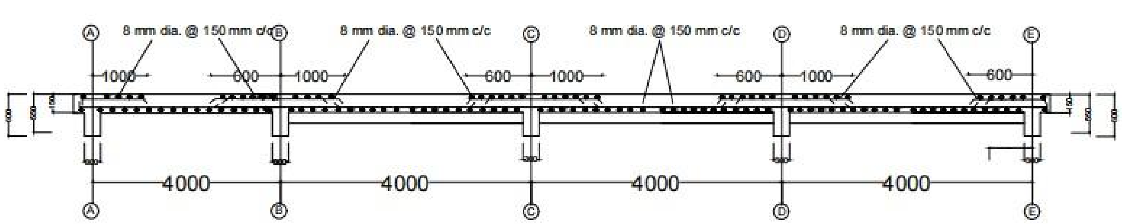


Figure 21: Reinforcement detailing of Slab section along Y-Y

Step 7: Check for Shear:

We have,

$$Vu = \frac{Wu \times lx}{2} = \frac{15.75 \times 4}{2} = 31.5 \text{ kN/m}^2$$

$$\tau u = \frac{Vu}{bd} = \frac{31.5 \times 1000}{1000 \times 130} = 0.2423\ N/mm^2$$

At support,

Percentage of steel (Pt) = $\frac{100Ast}{2\,bd} = \frac{100 \times 392.69}{2 \times 1000 \times 130}$ =0.15 %

From ***Table 19,Clause 40.2.1 of IS 456:2000,***

for Pt = 0.15 % and M35 Concrete;

$\tau c = 0.29$ N/mm^2

From ***Table 20,Clause 40.2.3 of IS 456:2000,***

for M35 Concrete;

$\tau c_{max} = 3.7$ N/mm^2

Again, from ***Clause 40.2.1.1 of IS 456:2000,***

for slab thickness = 150 mm;

k = 1.30

Therefore,

Permissible shear stress (τ‘c) = k × τc

= 1.30 x 0.29

= 0.377 N/mm^2

So, 0.2423 < 0.377 < 3.7

$\tau v < \tau c' < \tau c_{max}$

Hence, shear reinforcement is not required.

Step 8: Check for Development Length :

From (IS 456: 2000, Cl. 26.1.1);

For deformed bar, τ_{bd} =1.6 × 1.7 = 2.72 N/mm^2

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

$$L_d = \frac{0.87 f_y \emptyset}{4\tau_{bd}}$$

$$or, L_d = \frac{0.87 \times 500 \times 10}{4 \times 2.72} = 399.81\ mm$$

Also, $M_l = 0.87 \times fy \times \text{Ast} \times \left(d - \frac{fy\, Ast}{fck\, b}\right)$

$$= 0.87 \times 500 \times 392.69 \times \left(130 - \frac{500 \times 392.69}{35 \times 1000}\right) = 21.24\ kNm$$

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

$$L_d \leq 1.3\frac{M_l}{V_u} + l_o$$

Now, Taking $l_o = 0$,

$$1.3\frac{M_l}{V_u} + l_o = \frac{21.24 \times 10^3}{31.5 \times 10^3} = 674.285\ mm \quad > L_d$$

(OK)

Step 9: Check for Deflection Criteria :

$$\frac{l}{d} = \frac{4000}{130} = 30.77$$

From ***IS 456:2000 Cl. 23.2.1,***

α = 26 (For Continious Slab)

β = 1

δ = 1

θ =1

For γ ,

$$fs = 0.58 \times fy \times \frac{\text{Ast required}}{\text{Ast Provided}}$$

$= 0.58 \times 500 \times \frac{144.90}{392.69}$

$= 107.008$

$P\ \% = 100\ \frac{Ast}{D \times b}$

$= 100\ \frac{392.69}{150 \times 1000}$

$= 2.61$

So, from ***From IS 456:2000 Figure 4,***

$\gamma = 1.5$

Then, $\alpha \times \beta \times \gamma \times \delta \times \theta = (\frac{Span}{d})_{required}$

$= 1.5 \times 26$

$= 39$,,

$(\frac{Span}{d})_{provided} = \frac{4000}{130} = 30.76$

So, $(\frac{Span}{d})_{required} \leq \alpha \times \beta \times \gamma \times \delta \times \theta$

(OK)

The design is safe in deflection control criteria.

And, Detailing of Slab is attached in Annex no. 3.

8.4: Design of Staircase

Known Data,

Riser Height ,(first and third) R = 118 mm

(second) R=120mm

Tread Length (first and third) = 325mm

(second)=335mm

Floor Height = 3.2 m

Flight Width = 2 m

No. of Risers in the flights,(first and third) = 10

(second) =8

No. of Treads in the flights,(first and third) = 9

(second)=6

Length of the landings = 2 m

Step 1: Load Calculation

i. Flight

Assuming Slab Thickness, t = 150 mm

Self Wt. of Steps

(first and third) = $\sqrt{(\mathbf{0.118^2+0.325^2})}\times225\times25$= 2.66 KN/m per step

(second)= $\sqrt{(\mathbf{0.120^2+0.335^2})}\times225\times25$= 2.7106 KN/m per step

Wt. of Waist Steps-

(first and third)=0.5×0.118×0.325×25=0.479 KN/m per step

(second)= 0.5×0.120×0.335×25=0.5025479 KN/m per step

Total load per step

(first and third)=3.139 KN/m per step

(second)=3.2131 KN/m per step

Total load per m^2 = 3.139/0.325= 9.65 KN/m^2 ,3.2131/0.335=9.5913 KN/m^2

Considering 1m Width of Stab, Floor Finishing - 1.0 KN/m

Live Load - 4 KN/

Total Characteristics Load – 14.65 KN/m ,14.5913 KN/m

Design Load-1.5 x 14.65 =21.975 KN/m ,1.5×14.5913=21.8877 KN/m

Step 2: Design of landing:

Considering 1m Width of Slab,

Self Wt of Slab = 25 x 0.15 =3.75 KN/m

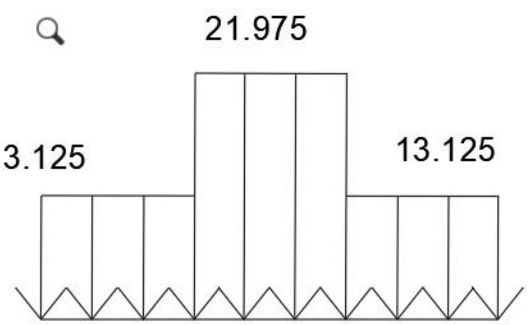
Floor Finishing = 1.0 KN/m

Live Load = 4 KNm

Total Characteristics Load =8.75 KN/m

Design Load = 1.5 x 8.75 = 13.125 KN/m

Step 3: Analysis



For upper and lower flight

Moment at about mid span

RA+RB =13.125×2.115+21.975×2.925+23.125×2.115 =119.79 KN/m

RA=RB =59.89 KN/m

Bending Moment (BM) at mid span

M_o =59.89×2.115-13.125×2.115×2.52-21.975×1.069 =120.81 KN/m

Clear cover =20 mm

Diameter of bars=16 mm

Effective depth =225-20-8 =197 mm

Check for the depth

dmin = $\sqrt{\frac{Mmax}{0.138*Fck*b}}$ = $\sqrt{\frac{120*10^6}{0.138*35*1000}}$ = 157.62mm (d provided)

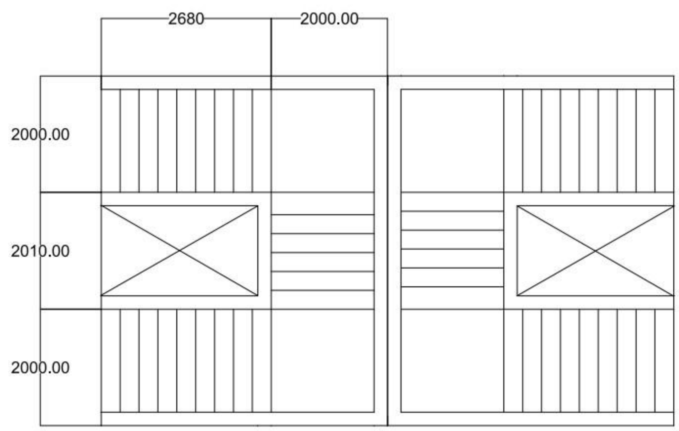


Figure 22:Architectural design of staircase

Step 4: .Design for the main reinforcement

for mid span ,

Mmax = 0.87 Fy ×Ast ×d ×(1- $\frac{Fy*Ast}{Fck*b*d}$)

Or,120×10^6 =0.87×500× Ast×197××(1- $\frac{500*Ast}{35*1000*197}$)

Ast req =1581.745 mm^2

Ast min = 0.0012×1000×197 =270 mm^2

Required spacing of dia 16mm bars,

c/c spacing = $\frac{1000*201.06}{1581.745}$ =127.11 mm (adopt 120 mm)

Area of steel provided =$\frac{1000*201.06}{120}$ =1675.5 (OK)

Step 5:: **Distributor Reinforcement**

Ast min =0.0012×1000×225 =270 mm^2

Required spacing of 10 mm bars

c/c spacing =$\frac{1000*78.546}{270}$ = 290.91 mm^2 =250 mm^2(adopt)

Step 6: Development Length Check

Development length

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

= $\frac{0.87\times 500\times 16}{4*12.72}$

= 639.706 mm

Provided development length = 700 mm

Step 7: Checking for the deflection of the slab

Percentage of steel = $\frac{100*Ast}{bd}$ = $\frac{100*1675.5}{1000*197}$ =0.85

Fs = 0.58×Fy× $\frac{1581.745}{1675.5}$

=273.77 MPa

From fig 4 , modification factor mt=0.85

We have , l/d =20

d = $\frac{4000}{1.1*20}$ =181.81mm (< 197 mm) OK

8.5: Design of Basement Wall

Known data:

Clear height between the floor (h) = 3 m

unit weight of soil, (Saturated), γ = 17 KN/m3

Angle of internal friction of the soil, ɵ = 30^0

surcharge produced due to vehicular movement is, Ws = 10 kN/m^2

safe bearing capacity of soil (q_s)=150 KN/m

Step 1: Moment Calculation

Ka = $\frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30}$ = 0.333

Lateral load due to soil pressure,

Pa = 0. 333.γ. h

=0.333×17×3

= 16.983kN/m^2

Lateral load =area of Δ

=1/2 ×16.983 ×3

Lateral Load due to surcharge load,

Ps = Ka ×Ws× h

= 0.333×10×3

= 9.99kN/m width of wall

Now, bending moment at the base of wall due to unit width of wall section can be calculated as:

Mx= {9.99n× (3/2)} + {25.4575× (3/3)} = 40.4425kN-m

Factored moment (Mu) = 1.5×40.4425 = 60.66 kN-m

Step 2: Design of Section

We have,

Let effective depth of wall =d

$$BM=0.36\times Fck\times b\times Xm(d-0.416Xm)$$

For fe500 ,Xm=0.46d

$$60.66\times10^6=0.36\times35\times1000\times0.46d(d-0.42\times0.46d)$$

$$D=119.4mm$$

Let clear cover is 40 mm and bar is 16 mm ϕ

Provide, overall (D) = 119.4+40+8=167.4

Take D= 200 mm

So, effective depth (d) = 200 – 40 – (16/2) = 152 mm

So two curtain of reinforcement is to be provided.

Step 3: Design of Main Reinforcement

$$Mu = 0.87\times fy\times Ast\times (d - \frac{fy\,Ast}{fck\,b})$$

$$60.66\times10^6 = 0.87\times500\times Ast\times (152 - \frac{500\times Ast}{35\times1000})$$

On solving, we get

$Ast = 1014.07\ mm^2$

Step 4: Check for minimum reinforcement and max. diameter of Bars

Acc. To cl.no 26.5.2.2 (IS 456-200)

$A_{st,\,min} =$ 0.12 % of total cross-sectional area for HYSD bars.

So, $A_{st,\,min} = 0.12\%\times bD = (0.12/100)\times1000\times200 = 240\ mm^2 < Ast$ OK

Maximum diameter of bar =D/8=200/8=25 mm, OK

Here, cross sectional area of a 16 mm dia. bar = $\frac{\pi \times 16^2}{4}$ =201.06 mm^2

Spacing of bars = $\frac{Breadth \times cross.\,sectional\ area\ of\ a\ bar}{Area\ of\ rein.\ provided}$

= 198.4 mm

Check According to *Cl. 32.5 (b) IS456-2000*, Maximum spacing for vertical reinforcement is not farther apart than three times the wall thickness or 450 mm, i.e, 3D or 450mm,

Since, spacing = 198.4 < 3×230 = 690 mm or 450, Hence OK

Hence, provide 16 mm dia. Bars @ 150 mm c/c spacing at earth face.

Hence, Provided A_{st}=1340.41mm^2

Step 5: Calculation of horizontal reinforcement

Minimum reinforcement required=0.0025×D×h (IS456:2000 Cl.32.5 (c) (2))

=0.0025×200×3000

= 1200 mm^2

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

Horizontal Reinforcement steel at front face, = (2/3) × 1200 = 800 mm^2

No of bars req,N=800×4/π×10^2=10.18

Adopt N=15

Spacing of 10 mm dia bar = (3000-80-10/14) = 207.85 mm

Provide 10 mm dia bar @ 200 mm c/c (inner face)

Horizontal Reinforcement steel at inner face,

=1/3 ×1200 =400 mm^2

No of bars =5.09 =10 nos

Spacing of 10 mm dia bar =3000-80-10/10-1= 323.33 mm

Provide 10mm dia bar @ 300 mm c/c

Step 6: **Check for Shear**

Assuming the section will be critical at distance d from face of support . then

Critical section is at d=0.152 from top of mat

Shear force at critical section is ,

$Vu=1.5\{Ka \times Ws \times h+Ka \times \gamma \times (h^2/2)\}$

$=1.5(0.333 \times 10 \times 2.848+0.333 \times 17 \times (2.848^2/2)$

$= 48.66$ kN

Nominal shear stress, $\tau v= 48.66 \times 10^3/(bd) = 48.66 \times 10^3/(1000 \times 174)$

$=0.32$ N/mm^2

From IS456:2000 Table-19, for M25,

Permissible shear stress, τ is from table 19, by interpolation

On interpolation, we get, $\tau_= 0.7016$ N/mm^2>$\tau_=$00.32N/mm^2 , Hence OK

Step 7: Check for deflection

(cl.no: IS456:2000 Cl.22.2.c& IS456:2000 Cl.23.2.a)

Allowable deflection = 3076/250=12.304 mm

Actual deflection = $\frac{3073^{3} * 12 * 1000}{1000 * 152 \times 5000 \times \sqrt{25}} \times (\frac{9.990}{8} + \frac{25.47}{30})$

= 8.21 mm< 12.30 mm, OK

Step 8: Curtailment of vertical reinforcement

From cl. No Cl.26.2.1., IS 456 2000,

Development length, Ld=$\frac{0.87fy\,\phi}{4\times1.6\times\tau bd}$=$\frac{0.87\times500\times\phi}{4\times1.6\times2.72}$

= 399.82 mm

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

Calculation of moment at curtailment point:

Lateral load due to soil pressure, Pa = Ka x γ x $h^2/2$

= 0.333x17x2^2/2

= 11.32kN/m

Lateral Load due to surcharge load,

Ps = Ka x Ws x h

= 0.333x10×2

= 6.66 KN/m

Characteristic Bending moment at the point of curtailment:

Mx= 11.322×2/3+6.66×2/2

= 14.20 kN-m

Factored moment = 1.5×14.20 = 21.312 kN-m

8.6: Design of Shear Wall

Known data:

Clear height between the floor (h) = 3 m

Unit weight of soil, (Saturated), γ = 17 KN/m3

Angle of internal friction of the soil, $\theta = 30^0$

Surcharge produced due to vehicular movement is, Ws = 10 kN/m^2

Safe bearing capacity of soil (q_s)=150 KN/m

Step 1: Moment Calculation

$Ka = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = 0.333$

Lateral load due to soil pressure,

Pa = 0. 333.γ. h

=0.333×17×3

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Lateral load =area of Δ

=1/2 ×16.983 ×3

Lateral Load due to surcharge load,

Ps = Ka ×Ws× h

= 0.333×10×3

= 9.99kN/m width of wall

Now, bending moment at the base of wall due to unit width of wall section can be calculated as:

Mx= {9.99n× (3/2)} + {25.4575× (3/3)} = 40.4425kN-m

Factored moment (Mu) = 1.5×40.4425 = 60.66 kN-m

Step 2: Design of Section

We have,

Let effective depth of wall =d

$$BM=0.36\times Fck\times b\times Xm(d-0.416Xm)$$

For fe500 ,Xm=0.46d

Or. $60.66\times10^6=0.36\times35\times1000\times0.46d(d-0.42\times0.46d)$

$\therefore$ D=119.4mm

Let clear cover is 40 mm and bar is 16 mm ϕ

Provide, overall (D) = 119.4+40+8=167.4

Take D= 200 mm

So, effective depth (d) = 200 – 40 – (16/2) = 152 mm

So two curtain of reinforcement is to be provided.

Step 3: Design of Main Reinforcement

$Mu = 0.87\times fy\times Ast\times (d - \frac{fy\ Ast}{fck\ b})$

$60.66\times10^6 = 0.87\times500\times Ast\times (152 - \frac{500\times Ast}{35\times1000})$

On solving, we get

$Ast = 1014.07\ mm^2$

Step 4: <u>Check for minimum reinforcement and max. diameter of Bars</u>

Acc. To cl.no 26.5.2.2 (IS 456-200)

$A_{st,\ min\ =}$ 0.12 % of total cross-sectional area for HYSD bars.

So, $A_{st,\ min\ =}$ $0.12\%\times bD = (0.12/100)\times1000\times200 = 240\ mm^2 < Ast$ OK

Maximum diameter of bar =D/8=200/8=25 mm, OK

Here, cross sectional area of a 16 mm dia. bar = $\frac{\pi \times 16^2}{4}$ =201.06 mm^2

Spacing of bars = $\frac{Breadth \times cross.\, sectional\ area\ of\ a\ bar}{Area\ of\ rein.\ provided}$

= 198.4 mm

Check According to *Cl. 32.5 (b) IS456-2000*, Maximum spacing for vertical reinforcement is not farther apart than three times the wall thickness or 450 mm, i.e, 3D or 450mm,

Since, spacing = 198.4 < 3×230 = 690 mm or 450, Hence OK

Hence, provide 16 mm dia. Bars @ 150 mm c/c spacing at earth face.

Hence, Provided A_{st}=1340.41mm^2

Step 5: Calculation of horizontal reinforcement

Minimum reinforcement required=0.0025×D×h (IS456:2000 Cl.32.5 (c) (2))

=0.0025×200×3000

= 1200 mm^2

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

Horizontal Reinforcement steel at front face, = (2/3) × 1200 = 800 mm^2

No of bars required,N=800×4/π×10^2=10.18

Adopt N=15

Spacing of 10 mm dia bar = (3000-80-10/14) = 207.85 mm

Provide 10 mm dia bar @ 200 mm c/c (inner face)

Horizontal Reinforcement steel at inner face, =1/3 ×1200 =400 mm^2

No of bars =5.09 =10 nos

Spacing of 10 mm dia bar =3000-80-10/10-1= 323.33 mm

Provide 10mm dia bar @ 300 mm c/c

Step 6: **Check for Shear**

Assuming the section will be critical at distance d from face of support . then

Critical section is at d=0.152 from top of mat

Shear force at critical section is ,

$Vu=1.5\{Ka\times Ws\times h+Ka\times ɣ \times (h^2/2)\}$

$=1.5(0.333\times10\times2.848+0.333\times17\times (2.848^2/2)$

$= 48.66$ kN

Nominal shear stress, $\tau v= 48.66 \times 10^3/(bd) = 48.66 \times10^3/(1000\times174)$

$=0.32$ N/mm^2

From IS456:2000 Table-19, for M25,

Permissible shear stress, τ is from table 19, by interpolation

On interpolation, we get, $\tau_= 0.7016$ N/mm^2>$\tau_=00.32$N/mm^2 ,

Hence **OK**

Step 7: Check for deflection

(cl.no: IS456:2000 Cl.22.2.c& IS456:2000 Cl.23.2.a)

Allowable deflection = 3076/250=12.304 mm

Actual deflection = $\frac{3073^{3}*12*1000}{1000*152\times5000\times\sqrt{25}}\times(\frac{9.990}{8}+\frac{25.47}{30})$

= 8.21 mm< 12.30 mm, OK

Step 8: Curtailment of vertical reinforcement

From cl. No Cl.26.2.1., IS 456 2000,

Development length, Ld=$\frac{0.87fy\,\phi}{4\times1.6\times\tau bd}$=$\frac{0.87\times500\times\phi}{4\times1.6\times2.72}$

= 399.82 mm

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

Calculation of moment at curtailment point:

Lateral load due to soil pressure, $Pa = Ka \times \gamma \times h^2/2$

$= 0.333 \times 17 \times 2^2/2$

= 11.32kN/m

Lateral Load due to surcharge load,

$Ps = Ka \times Ws \times h$

$= 0.333 \times 10 \times 2$

= 6.66 KN/m

Characteristic Bending moment at the point of curtailment:

$Mx = 11.322 \times 2/3 + 6.66 \times 2/2$

= 14.20 kN-m

Factored moment = 1.5×14.20 = 21.312 kNm

8.7. Design of Parking Ramp

Known Data,

Concrete Grade= M35

Steel Grade= Fe500

Dia. of bar = 12 mm

Clear cover = 15 mm

Floor Height=3000 mm

Flight Width, W=4000 mm

Effective length,,l = 4000 mm

Take, $\frac{l}{d}$ =40

$\therefore$ d = $\frac{400}{40}$=100 mm

Adopt depth of slab (D)=200mm

Effective depth(d)=179mm>100mm

(OK)

Step 1: Load Calculation

Live load = 4 KN/m

DL for waist slab= 25×0.20 ×$\frac{\sqrt{1.5^2+8^2}}{8}$

= 5.088 KN/m

$\therefore$Total load =4 + 5.088

= 9.088 KN/m

Factored load = 1.5×9.088

$=13.632$ KN/m

Maximum bending moment $= \frac{wl^2}{10}$

$$= \frac{(13.632 \times 4^2)}{10}$$

$$= 21.8112 \text{ kNm}$$

Step 2: Check for depth

$Mu = 0.36\, f_{ck} b x_m (d- 0.42x_m)$

Or, $21.8112 \times 10^6 = 0.36 \times 35 \times 1000 \times 0.46\, d \times (d- 0.42 \times 0.46d)$

$\therefore$ d =68.9 mm <179 mm

(OK)

Step 3: Calculation of Area of steel

$M_u = 0.87 f_y A_{st} (d- \frac{fy*Ast}{fck*b})$

or, $21.8113 \times 10^6 = 0.87 \times 500 \times A_{st} \times (179 - \frac{500*Ast}{35*1000})$

$\therefore A_{st} = 286.67 \text{ mm}^2$

Now,

$A_{st.min.} = 0.12\%$ of bD

$= 0.12\% \times 1000 \times 200$

$= 240 \text{mm}^2 < 286.67 \text{ mm}^2$ **(OK)**

Spacing $= \frac{1000}{\text{Ast}} \times \frac{\pi \times \phi^2}{4}$

$= \frac{1000}{240} \times \frac{\pi \times 12^2}{4}$ $=394.53$mm

Provide 12mm bar@ 300 mm c/c.

Step 4: Distributors Reinforcement

$A_{st,provided} = \frac{1000}{300} \times 113.097$

$= 376.99 \text{ mm}^2$

Provide 12mm bars @200mm (4 bars)

$P_t = \frac{1000 \ Ast}{bD}$

$= \frac{100 \times \ 376.99}{1000 \times 200}$

$= 0.188 \ \%$

Step 5: Check For Shear

$V_u = \frac{wl}{2}$

$= \frac{13.632 \times 4}{2}$

$= 27.264 \text{ KN}$

$\tau_v = \frac{Vu}{bd}$

$= \frac{27.264 \times 1000}{1000 \times 179}$

$= 0.124 \text{ N/mm}^2$

According to ***IS 456: 2000, Cl. 40.2.1,***

By interpolation,

Minimum τ_c at M35= 0.32 N/mm^2 > τ_v

Step 6: Development Length Calculation

From (IS 456: 2000, Cl. 26.1.1),

For deformed bar, τ_{bd} =1.6 × 1.7

$= 2.72 \text{ N/mm}^2$

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

$$L_d = \frac{0.87 f_y \emptyset}{4\tau_{bd}}$$

$$or, L_d = \frac{0.87 \times 500 \times 12}{4 \times 2.72}$$

$$= 479.77\ mm$$

CHAPTER 9
CONCLUSION

In conclusion, this project has effectively met all of its stated objectives by providing us with the theoretical knowledge and practical experience necessary to design and analyze an earthquake-resistant multi-storeyed RCC building. We successfully modeled and analyzed the structure using commercial software tool ETABS, gaining valuable hands-on experience in professional engineering applications.

The major outcomes of this project can be summarized as follows:

1. Load cases and combinations were accurately defined according to NBC 105:2020 and IS 875 (Part I), ensuring compliance with seismic and other design requirements.
2. Structural irregularities, both planar and vertical, were checked and addressed to enhance the building's seismic performance.
3. Drift and displacement under seismic loads were determined and found to be within permissible limits, confirming the building's stability during earthquakes.
4. Seismic detailing and design checks were performed as per NBC 205 and NBC 105:2020, ensuring adequate reinforcement and ductility for seismic resistance.

These outcomes reflect our successful completion of the project goals and readiness to apply these skills in real-world structural engineering tasks.

The projected hospital building of 10 story with basement and lift provision by shear wall was designed. During analysis, we found that most of the loads are acted through the shear wall's and due to of which beam's failures mainly occurs at this location. Hence, We increased grade of concrete and steel as well as increased the dimension of frame structures.

The further results are briefly summarized as under:

Conclusion on Beam Design

- The beam size was designed as 300mm x 550mm.
- The ductile detailing and design were done as per ***IS 456:2000.***

Conclusion on Column Design

- The size of column was found as 550mm x 550mm
- The axial load found increasing on lower floor column. The reinforcement design is done considering the axial and moment on the column.

Conclusion on Slab Design

- The floor slab thickness was calculated as 150mm
- All of the slab panel were found two ways.
- The design of reinforcement was done manually and checked for shear and deflection.

Conclusion on Staircase Design

- The waist slab thickness was calculated as 160 mm.
- The analysis and design were done manually.
- The reinforcement detailing was done as per ***IS456:2000***

CHAPTER 10

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ANNEX A: ETABS DRAWING

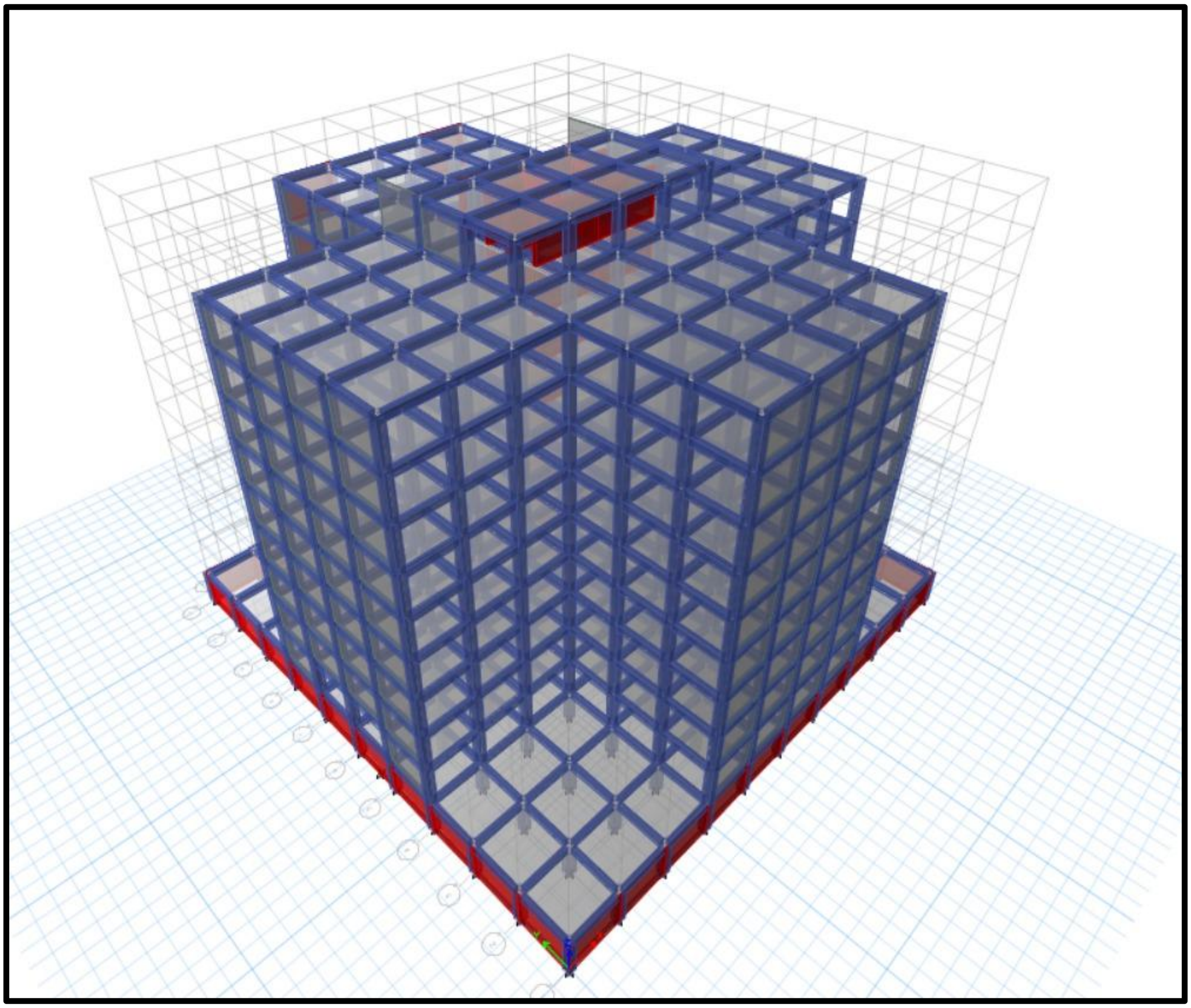

Figure 23 Etabs Modelling

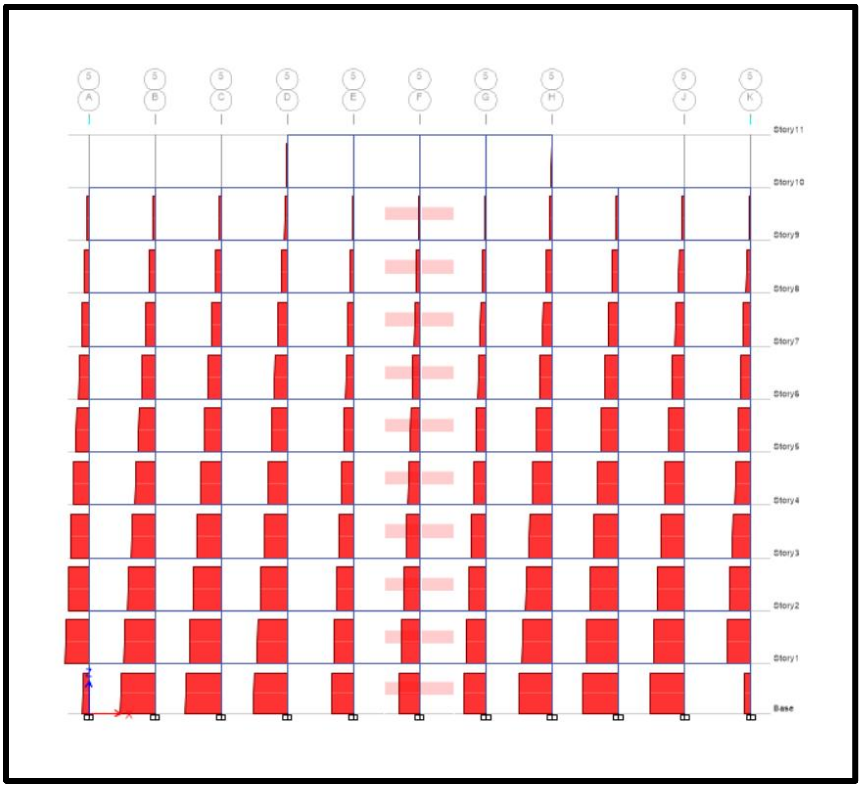

Figure 24 Axial load diagram at '5' grid elevation

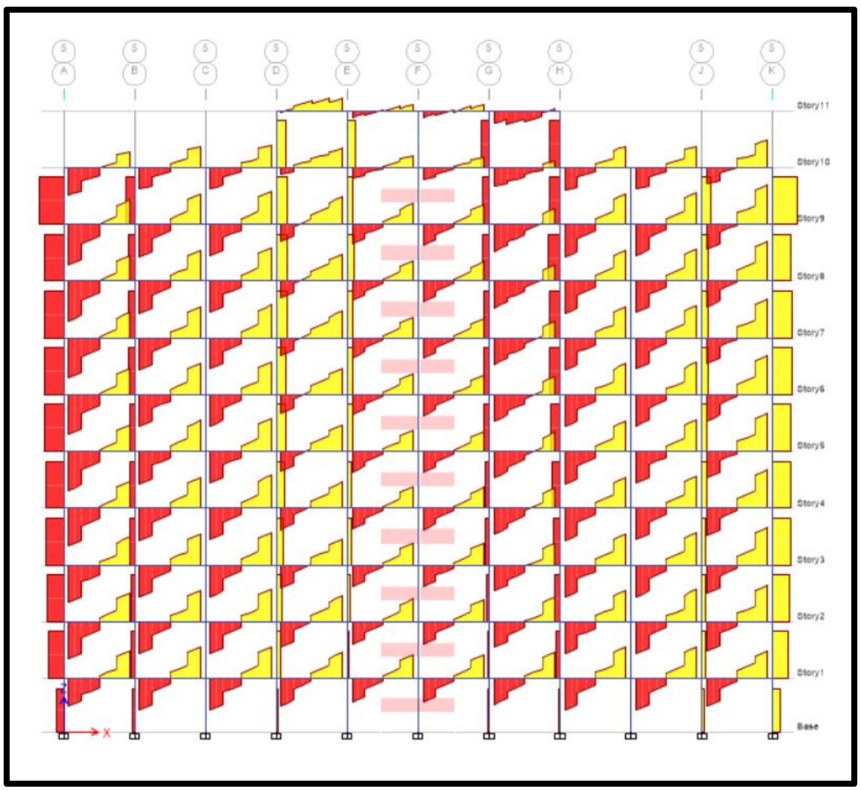

Figure 25 Shear force diagram at '5' grid elevation

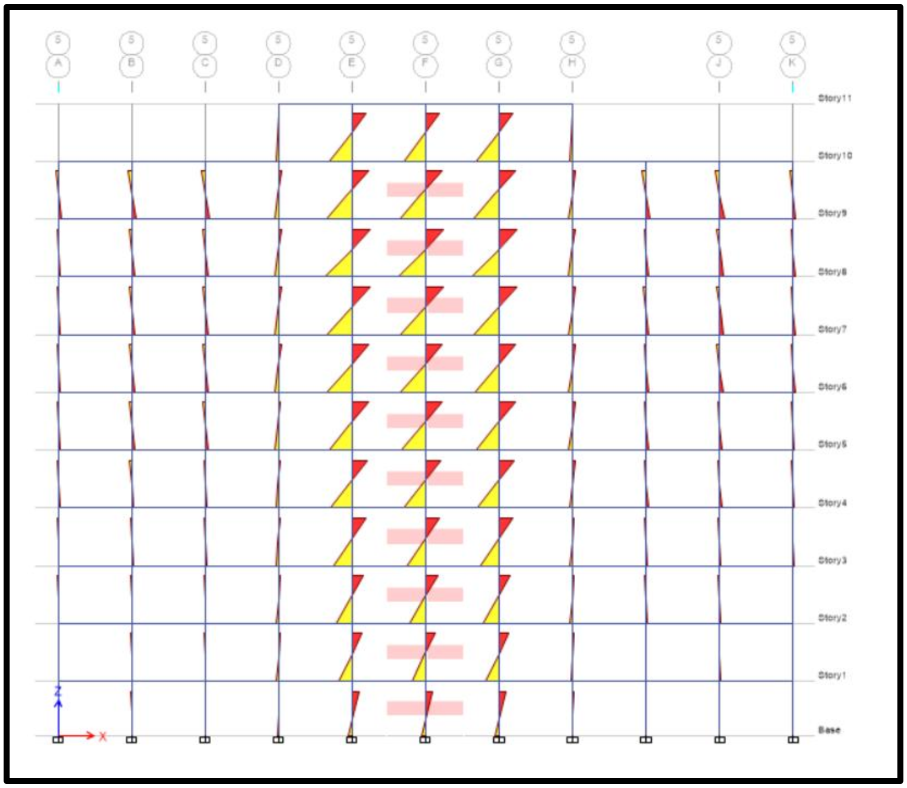

Figure 26 Bending Moment diagram at '5' grid elevation

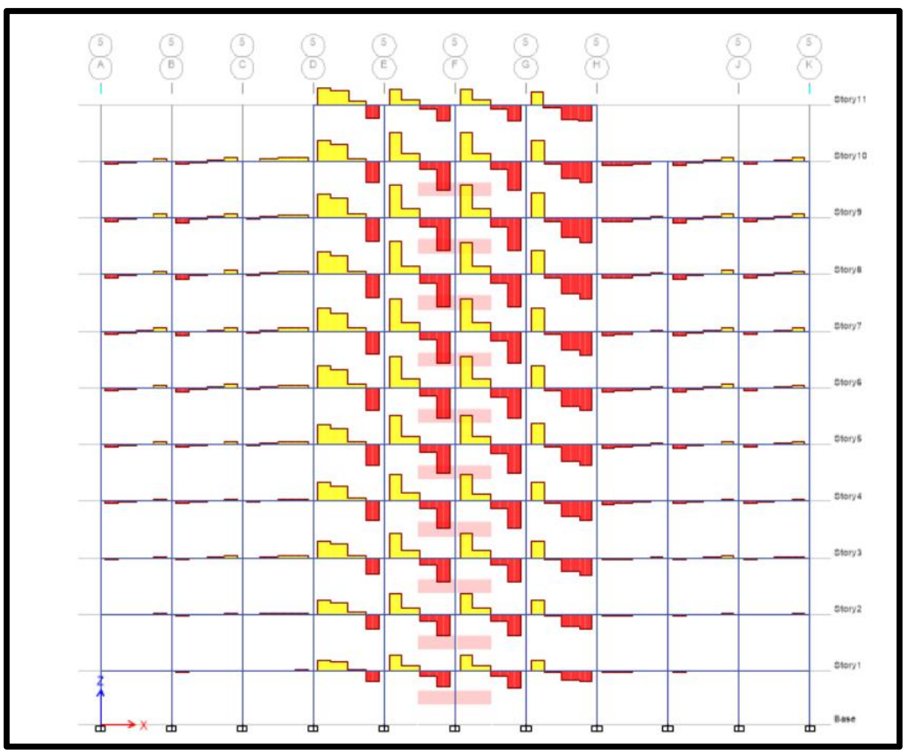

Figure 27 Torsion diagram at '5' grid elevation

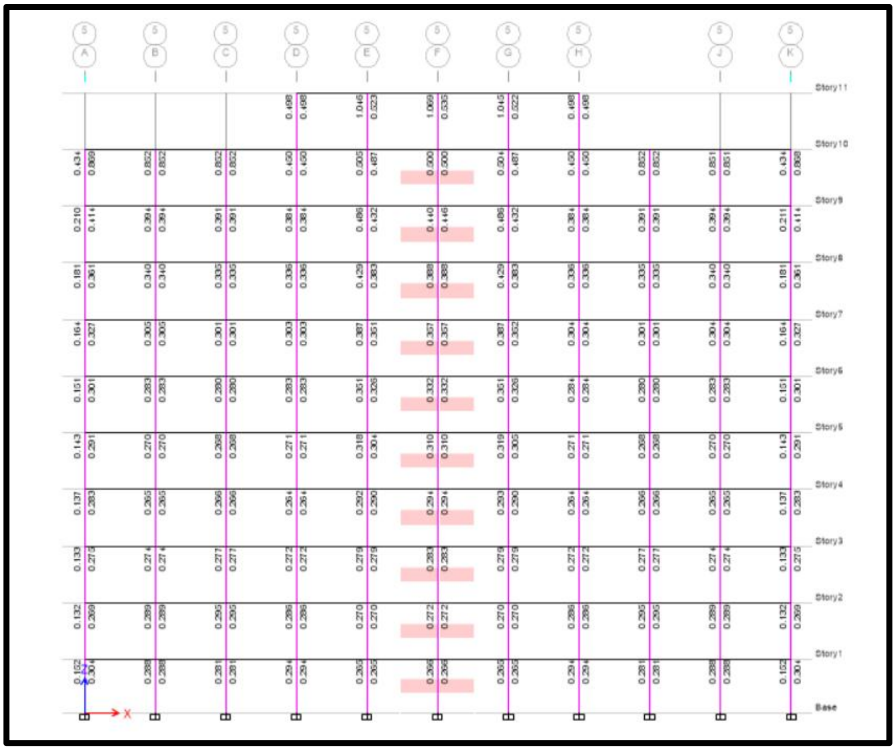

Figure 28 Beam /Column Capacity Ratio

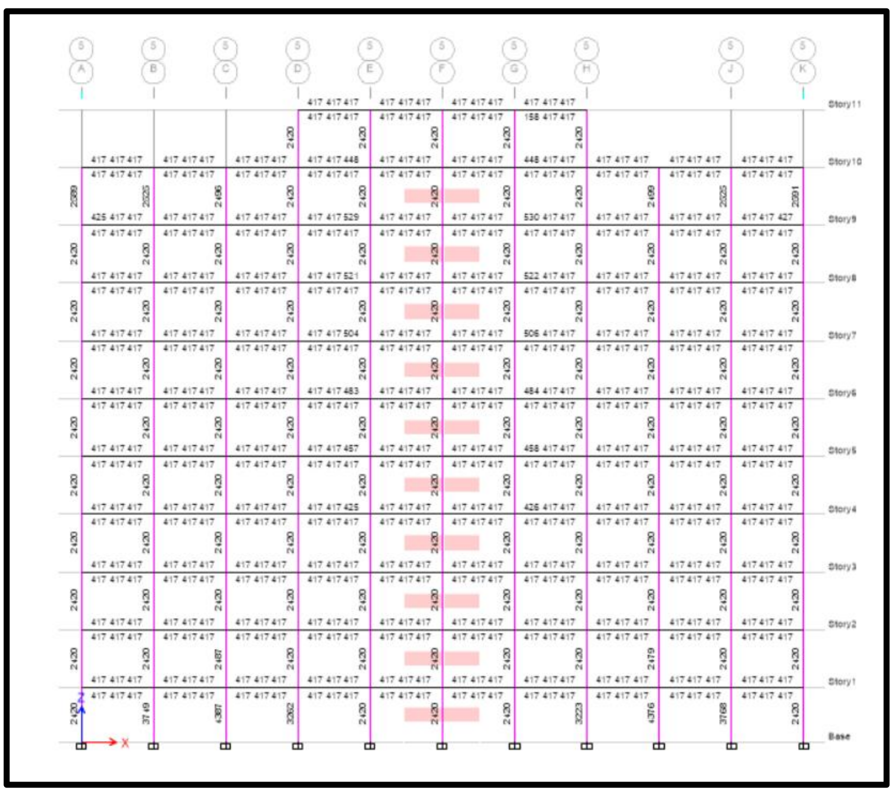

Figure 29 : Longitutional Reinforcing

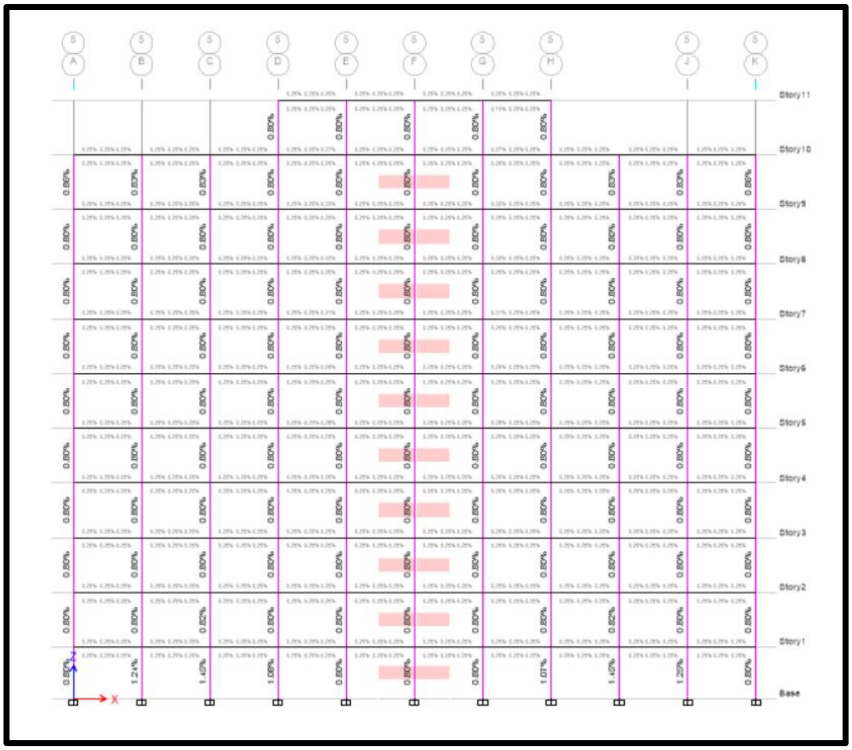

Figure 30 Rebar reinforcement percentage

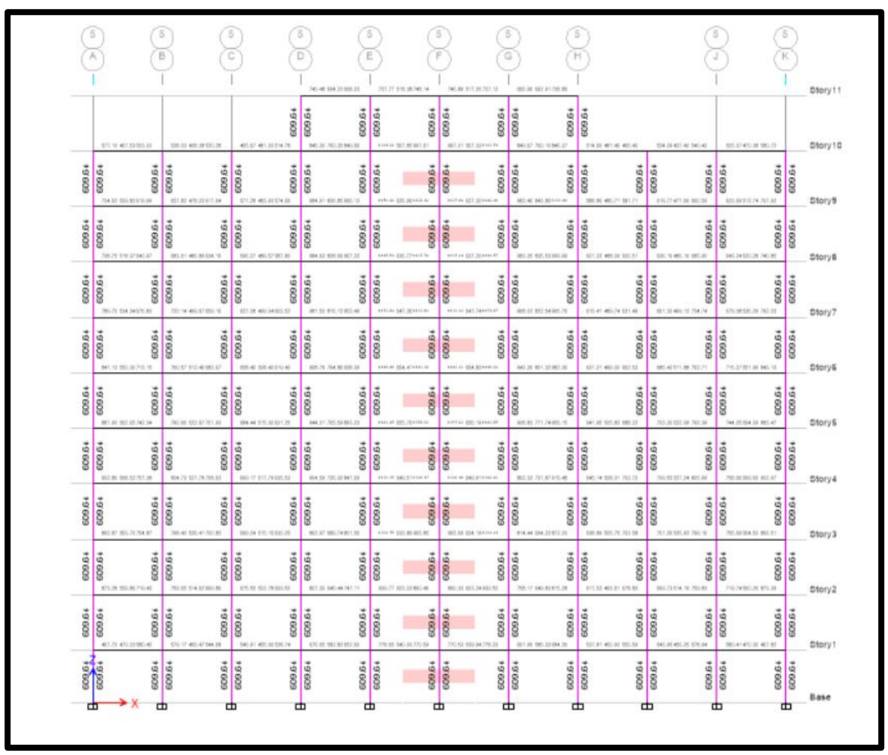

Figure 31 Shear Reinforcement percentage

ANNEX B: ARCHITECTURAL DRAWING

ANNEX C: RCC DETAILING