Tribhuvan University

Institute of Engineering

PASHCHIMANCHAL CAMPUS

Pokhara -16, Lamachaur

FINAL YEAR PROJECT REPORT ON

"SEISMIC ANALYSIS AND DESIGN OF HOSPITAL BUILDING"

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SUBMITTED TO:

DEPARTMENT OF CIVIL ENGINEERING

JESTHA, 2080

DECLARATION

We hereby declare that the project titled "Seismic Analysis and Design of Hospital Building" submitted to the Department of Civil Engineering, Pashchimanchal Campus, Institute of Engineering, Tribhuvan University, has been prepared under the supervision of Er. Deepak Thapa, submitted in partial fulfilment as per the requirement for Bachelor's Degree in Civil Engineering.

We affirm that the contents of this report are original and have not been submitted for any other purpose or to any other institution. Any references, sources or external material used in this report have been appropriately cited and acknowledged.

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ACKNOWLEDGEMENT

We would like to express our gratitude and appreciation to all those who have contributed to the successful completion of this report. Firstly, we would like to thank the Department of Civil Engineering for providing us with the necessary resources and support to carry out this project.

We are also immensely grateful to our project supervisor Er. Deepak Thapa and all the teachers, who have been a constant source of inspiration and guidance throughout our academic journey. Their valuable insights and constructive feedback have helped us refine our ideas and approach towards this report.

Once again, we express our sincere thanks to all those who have contributed to this report and helped us achieve this milestone.

ABSTRACT

To introduce the students to the real civil engineering practice and to give them confidence, the ability to tackle problems related to Civil engineering, and the idea of practical working in a professional field with the application of theoretical knowledge gained during the whole four years, there is a provision of project work in the syllabus of IOE, TU. The project titled "Seismic Analysis and Design of Hospital Building" is the outcome of our studies. It has seen its necessity as there are not enough hospitals for increasing patients since the country's topographical and sociological diversifications contribute to periodic epidemics of infectious diseases, epizootics, and natural hazards like floods, forest fires, landslides, and earthquakes. Our site is located in Pokhara-32, Kaski. Different methodologies like desk study, architectural drawing, preliminary design, structural analysis, detail drawing and documentation were adopted for the completion of the project.

The design of elements was done by using limit state design philosophy, which is economic, safe, and reliable. The designing tools used in this project were REVIT 2018, ETABS v18 and AutoCAD 2018. Loads were applied to the structure as per IS 875 (Part I) :1987[dead loads], IS 875 (Part II) :1987[live loads], IS 875 (Part-III) :1987[wind loads] and earthquake loads were applied based on NBC 105:2020. Materials properties were assumed as per the common practice and presumptive soil bearing capacity was considered as per IS 1904:1986.

Once the analysis was completed all the structural components were designed according to IS 456:2000. Footing, columns, beams, slab, staircase and shear wall were designed. Ductile detailing of the structural elements was done as per IS 13920:2016. Rates for estimation were taken from Kaski District Rate 2079/80.

Keywords: *AutoCAD, Drawings, Earthquake Resistant, ETABS, Structural Analysis*

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LIST OF SYMBOLS AND ABBREVIATION

Symbols	Meaning
α_x, α_y	BM coefficients for Rectangular Slab Panels
φ	Diameter of Bar, Angle of internal friction of soil
δ_m	Percentage reduction in moment
τ_c	Shear stress in concrete
$\tau_{c,max}$	Max. shear stress in concrete with shear reinforcement
τ_{bd}	Design Bond Stress
σ_{ac}	Permissible Stress in Axial Compression (Steel)
σ_{cbc}	Permissible Bending Compressive Strength of Concrete
σ_{sc}, σ_{st}	Permissible Stress in Steel in Compression and Tension respectively
Υ_m	Partial Safety Factor for Material
Υ_f	Partial Safety Factor for Load
Υ	Unit Weight of Material
Ωu	Over strength Factor for ULS
Ωs	Over strength Factor for SLS
A_B	Area of Each Bar
A_G	Gross Area of Concrete
A_H	Horizontal Seismic Coefficient

Symbols	Meaning
A_{SC}	Area of Steel in Compression
A_{ST}	Area of Steel in Tension
A_{SV}	Area of Stirrups
CQC	Complete Quadratic Combination
$C(T_1)$	Elastic Site Spectra
$C_d(T_1)$	Design Horizontal Base Shear Coefficient
$Cs(T_1)$	Elastic Site Spectra determined for Serviceability Limit State
B or D	Width or Shorter Dimension in Plan
b_F	Effective width of flange
d	Effective Depth
d'	Effective Cover
D	Overall Depth
D_F	Thickness of Flange
e_x	Eccentricity along x-direction
e_y	Eccentricity along y-direction
E_c	Modulus of Elasticity of Concrete
E_s	Modulus of Elasticity of Steel
EL_X, EL_Y	Earthquake Load along X and Y direction respectively
F_{BR}	Bearing stress in concrete

Symbols	Meaning
f_{ck}	Characteristics Strength of Concrete
f_y	Characteristic Strength of Steel
Fi	The lateral seismic force induced at each level ‘i’
FF	Floor finish Load
h_i	Height (m) from the Base to Level ‘i’
H	Height of building
I	Importance Factor (For Base Shear Calculation)
I_{XX}, I_{YY}	Moment of Inertia (along x and y direction)
J	Neutral Axis Depth Factor
k	Exponent Related to the Structural Period
K	Coefficient of Constant or factor
k_1, k_2, k_3	Coefficient for wind pressure
K_A, K_B	Active and Passive Earth Pressure
L	Length of Member
L_{EF}	Effective Length of member
L_D	Development Length
M	Modular Ratio
M or BM	Bending Moment
n	Total Number of Floors/Levels

Symbols	**Meaning**
N_U, P_U	Ultimate Axial Load on a compression member
P_c	Percentage of Compression Reinforcement
P_t	Percentage of Tension Reinforcement
P_Z	Wind Pressure
Q, Q_U	Permissible and Ultimate bearing capacity of soil
QI	Design Lateral Force in ith Level
SR_{MIN}	Slenderness Ratio (minimum) for structural steel section
SRSS	Square Root of the Sum of the Squares
R	Response Reduction Factor
R_μ	Ductility Factor
S_V	Spacing of Each Bar
T_I	Torsional Moment due to Lateral Force in ith-direction
T_A	Fundamental Natural Period of Vibrations
V_B	Basic wind speed
V_Z	Design wind speed
V_B	Design Seismic Base Shear
V	Shear Force
W_i	Seismic Weight of i^{th} Floor
WL	Wind Load

Symbols	**Meaning**
X_U	Actual Depth of Neutral Axis
X_{UL}	Ultimate Depth of Neutral Axis
Z	Seismic Zone Factor
CM	Center of Mass
CR	Center of Rigidity
DL	Dead Load
HSDB	High Strength Deformed Bars
IS	Indian Standard
LL	Live Load
RCC	Reinforced Cement Concrete
SPT, N	Standard Penetration Test
M25	Grade of Concrete
Fe500, Fe415	Grade of Steel

1. INTRODUCTION

1.1 Background

Hospitals are basic health care infrastructures, whose design plays an integral part in the lives of people and structural analysis is critical in terms of the civil engineering domain. During natural disasters and epidemics, their structural significant role is even more vital. For a country like Nepal, the design and development of a better hospital for people could mean a better improvement to the present very low indicators of health care infrastructure.

Nepal is prone to frequent earthquakes due to its position and lies in the convergent plate boundary between two active tectonic plates namely, the Eurasian plate and the Indian plate. Nepal has become seismically active. In the Nepal earthquake of 2015, more than 600,000 structures in Kathmandu and other nearby towns were either damaged or destroyed. There are more damage and more deaths from earthquakes primarily because of buildings that are poorly designed and constructed for earthquake regions. In 2015, earthquakes and aftershocks damaged and destroyed more than 1200 health facilities and affected access to healthcare services for an estimated 5 million people. So, for a seismic country like Nepal, it is crucial to effectively apply earthquake-resistant standards and guidelines have to be published and practiced for hospital facilities. Structural knowledge and its practice are a must in the present day.

The structural analysis aims to establish a wise distribution of internal forces, moments, stresses, strains, and displacements over the whole or part of a structure. The design of every building is fundamentally concerned with ensuring that the components of buildings, e.g., lateral force-resisting system, can adequately serve their intended functions. In the case of seismic design of the lateral resisting system, the limit state method (LSM) approach was adopted for the design of buildings among the different design philosophies. Indian Standard codes and Nepal Building codes for practice should be adopted thoroughly for proper design and detailing for safety, economy, stability, and strength. Hence, it is important to know structural analysis and design methods of buildings so that buildings stood up to be good in strength, stability, safety, and cost-effectiveness.

The project selected by our group is a seismic analysis and design of a 100-bed hospital located inside Pokhara Metropolitan City. As Pokhara lies in zone V, the severest, zone the effect of earthquakes is dominant over wind load. Thus, the building will be analyzed for earthquake as lateral load. All the theoretical knowledge of analysis and design acquired in the coursework will be utilized with practical application. The main objective of the project is to acquaint us with the practical aspects of Civil Engineering.

1.2 Need of Project

Hospital buildings are among the various types of buildings like public, residential, commercial, etc., that we see around us. The various buildings have their purposes and structural ability as the mass of people or loads attained are different from the other. Likewise, hospitals are an important part of the health care system. During disasters and epidemics, its role is even more vital. All of us who have gone through the catastrophe in 2015, and Covid 19 recently have felt the need for more hospitals in Nepal. Hospitals all around the world play an essential role in response to the Covid-19 pandemic. It seems we should look at the Covid 19 pandemic and past earthquakes as learning opportunities to re-frame what we expect from hospitals and to re-think the way we build, train, access, and evaluate them. The Covid 19 pandemic has created a multitude of acute challenges for hospitals, including inadequate capacity, shortage of intensive care beds, the need for care re-design, supply shortages, and financial losses. In Hospitals, sick people or physically disabled or weak people are present so, during an earthquake, they cannot rush outside like others. That is the reason why hospitals should be much safer which people can rely on and can get a safer place for treatment even in the post-earthquake period.

The design and execution of a new hospital building with 100 beds will lead to healthcare facilities targeting the people of Pokhara-32 in the Kaski district. According to official website of the Pokhara Metropolitan Pokhara-32 is home to 10677 people but only a few health care units are available there, so, the difficulties that the residents of this area are facing and their obligations to rush to the main Pokhara city are major. Hospitals are safe spaces for people during emergencies, disasters and crises. According to statistics recorded by the Pokhara Metropolitan in 2021, the total population of a metropolitan city is 518,452 and the population density is 1100 persons per square kilometer which are higher than the average population density (202 persons per square kilometers) of Nepal. The need for a stable and enough bed hospital is felt.

1.3 Objectives

The main objective of this project work is to design and analyze earthquake resistant hospital building conforming to prevailing code of practice in Nepal. The specific objectives of the project work are;

- Preliminary design of structural components and modeling of the building for structural analysis.
- To prepare a detailed architectural and structural drawing of the hospital building following IS codes and NBC codes.
- To prepare the estimation of a multistorey building.

1.4 Scopes of the Project

Summary of all the important parameters of the entire project is as follows:

- Identification of requirements and justification for building.
- Development of architectural drawings adhering to identified requirements, by laws and codes.
- Understanding the architectural drawing and load assessment.
- Initial design of geometry of building elements as per the architectural drawings.
- Definition of loads: dead loads, imposed loads and earthquake loads.
- Calculation of internal forces [Normal Force, Shear Force, Bending Moments] on each element.
- Calculation of base shear and vertical distribution of equivalent earthquake load.
- Design of foundation.
- Structural drawing and detailing of individual members as a part of working construction document.

2. SALIENT FEATURES

a. Name of the Project: Seismic Analysis and Design of Hospital Building
b. Location:
   i. Province: Gandaki
   ii. District: Kaski
   iii. City: Pokhara Metropolitan City
   iv. Parcel no: 4268
c. Type of Building: Institutional Building
d. Structural System: Special Moment Resisting Frame
e. Soil Type: C
f. Seismic zone: V
g. Size of Plot: 12849.609 m^2
h. Permissible Ground Coverage Area: 6424.805 m^2
i. Plinth area: 2815.616 m^2
j. No of Storey: 3
k. Floor height: 12’ (3.6576 m)
l. Type of Staircase: Open Well Staircase
m. Type of foundation: Isolated Footing
n. Infill wall: Brick Masonry
   i. Main wall: 230 mm
   ii. Partition wall: 115 mm
o. Design criteria: As per NBC code and IS code
p. No of columns: 111
q. Number of lifts available: 2
r. Ramp**:**
   i. Slope: 1:15
   ii. Landing: 2.8 m

3. LITERATURE REVIEW

Every engineering design is the outcome of the past experiences and observations. It is necessary to justify the result of the analysis and design properly with reference to the pre-existing standard results or the past experiences. Structural design is the methodical investigation of the stability, strength and rigidity of structures. The basic objective in structural analysis and design is to produce a structure capable of resisting all applied loads without failure during its service life. Safe design of structures can be achieved by applying the proper knowledge of structural mechanics and past experiences. It is needed to provide authentic reference to the design made i.e., the design should follow the provision made in codes of practices. Use of codes also keeps the designer to the safe side in case the structure fails within its service life. For this design, certain references and criteria are taken from these literatures

3.1 Nepal National Building Code (NBC 105:2020):

Nepal National Building Code was prepared during 1993 as part of a bigger project to mitigate the effect of earthquakes on the building of Nepal. It deals primarily with matters relating to the strength of buildings. However, there are some chapters on site considerations and safety during construction and fire hazards. This code aims to bring uniformity to the building construction by providing some bye-laws and mandatory rules. The code frequently refers to Indian Standard codes. The four different levels of sophistication of design and construction that are being addressed in this National Building Code are as follows.

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

This project belongs to the second part of NBC i.e. Professionally Engineered Structures. As the National Building Code defines the use of international codes which meets the requirements stated in NBC, different Indian Standard codes are used for the design and analysis purpose.

3.2 Indian Standard (IS) Codes of Practice:

For the analysis and design of the building references have been made to Indian Standard code since National Building Codes of Nepal do not provide sufficient information and refers frequently to the Indian standard codes. Indian Standard codes used in the analysis and design of this building are described below:

A. IS: 875- 1987 (Reaffirmed 2003)- Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures:

A building has to perform many functions satisfactorily. Amongst these functions are the utility of the building for the intended use and occupancy, structural safety, fire safety; and compliance with hygienic, sanitation, ventilation and daylight standards. The design of the building is dependent upon the minimum requirements prescribed for each of the above functions. The minimum requirements pertaining to the structural safety of the building are being covered in this code by way of laying down minimum designed load which have to be assumed for dead loads, imposed load, snow load and other external loads, the structure is required to bear. Strict conformity to loading standard recommended in this code claims to ensure the safety of the buildings and thereby reduced the hazards to life and property caused by unsafe structures as well as eliminates the wastage caused by the assumption of unnecessary heavy loading. This code is divided into five different parts for five different kinds of loadings. The different parts of the code are:

Part 1: Dead Loads- Unit Weight of Building Materials and Stored Materials:

This part deals with the dead load to be assumed in the design of the building. These loads are given in the form of unit weight of materials. The unit weight of the materials that are likely to be stored in the building are also given in the code for the purpose of the load calculation due to stored materials. This code covers the unit weight or mass of the materials and parts and components in the building that apply to the determination of the dead load in the design of building. Table 1 of this code covers unit weight of the building materials and Table 2 of the code covers the unit weight of the building parts or the components.

Part 2: Imposed Loads:

Imposed load is the load assumed to be produced by the intended use or occupancy of a building including the weight of moveable partitions, distributed, concentrated loads, loads due to impact and vibrations and dust loads (Excluding wind, seismic, snow, load due to temperature change, creep, shrinkage, differential settlements etc.) This part of the code deals with imposed load of the building produced by the intended occupancy or use. Minimum imposed load that should be taken into consideration for the purpose of structural safety of the buildings are given in the code but it does not cover the incidental to construction and special cases of vibration, such as moving machinery, heavy acceleration from cranes hoist etc.

Part 3: Wind Loads

This part deals with the wind load to be considered when designing the building, structure and component thereof. This code gives the wind force and their effect (Static and Dynamic) that should be taken into account when designing buildings, structures and components thereof. In the code wind load estimation is done by taking into account the random variation of the wind speed with time.

Part 4: Snow Loads

This part of the code deals with snow loads on roofs of buildings. Roofs should be designed for the actual load due to snow or the imposed load specified in Part 2 whichever is more sever. Since location of the building is within Pokhara Valley, there is no possibility of snowfall. Hence the snow load is not considered in the design.

Part 5: Special Loads and Load Combinations

This code loads and loads effects (Except the loads covered in Part 1 to 4 and seismic load) due to temperature changes, internally generated stress due to creep shrinkage, differential settlement etc. in the building and its components, soil and hydrostatic pressures, accidental loads etc. This part also covers the guidance for the load combinations.

B. IS 1893 (Part 1): 2002 Criteria for Earthquake Resistant Design of Structures (General Provision and Building):

This code deals with the assessment of seismic loads on various structures and earthquake resistant design of buildings. Its basic provisions are applicable to buildings; elevated structures; industrial and stack like structures; bridges; concrete masonry and earth dams; embankment and retaining structures and other structures. Temporary supporting structures like scaffoldings etc. need not be considered for the seismic loads. It is concerned with the methods of determining seismic loads and the effects of various irregularities in a building can have upon its seismic response. This standard does not deal with the construction features relating to earthquake resistant design in building and other structures.

C. IS 13920: 2016 (Reaffirmed 2003) Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Force- Code of Practice:

This standard covers the requirements for designing and detailing of monolithic reinforced concrete buildings so as to give them adequate toughness and ductility to resist sever earthquake shock without collapse. The provision for the reinforced concrete construction given in the code are specifically to the monolithic reinforced concrete construction. For precast and prestressed concrete members, its use is limited only if they can provide the same level of ductility as that of monolithic reinforced concrete construction during or after earthquake. The code includes the detailing rules for flexural members, column and frame member subjected to bending and axial loads and shear walls.

D. IS 456: 2000 (Reaffirmed 2005) Plain and Reinforced Concrete – Code of Practice:

This Indian Standard code of practice deals with the general structural use of plain and reinforced concrete based on Limit State Design Method. According to the code, plain concrete structures referred to those structures where reinforcement if provided is ignored for Introduction 14 determination of the strength of the structure. This code does not cover special requirements for the structures like bridges, chimneys, hydraulic structures, earthquake resistance buildings etc. but allows the use of separate code for those structures in conjunction with this code.

E. IS 4326: 1993 (Reaffirmed 2003) Earthquake Resistant Design and Construction of Buildings – Code of Practice:

This standard deal with the selection of materials, special features of design and construction for earthquake resistant buildings including masonry construction using rectangular masonry units, timber construction and buildings with prefabricated flooring or roofing elements.

F. IS 5525: 1969 (Reaffirmed 1990) Recommendations for Detailing of Reinforcement in Reinforced Concrete Works:

This standard deal with the general requirements of detailing of reinforcement in reinforced concrete structures with some suitable modifications whenever necessary. This code includes the common method of detailing of reinforcement based on good practice with deviations made in special cases to comply with IS 456.

G. IS 1642: 1989 (Reaffirmed 1994) Fire Safety of Buildings (General): Details of Construction – Code of Practice:

This standard lays down the essential requirements of fire safety of buildings with respect to details of construction.

H. IS 2950 (Part I): 1981 (Reaffirmed 1998) Code of practice for design and construction of Raft Foundations:

Raft foundation is a substructure supporting an arrangement of columns or walls in a row or rows and transmitting the loads to the soil by means of a continuous slab with or without depressions or openings, useful where soil has low bearing capacity. This standard covers the design of raft foundation based on conventional method (for rigid foundation) and simplified methods (flexible foundation) for residential and industrial buildings, store-houses, silos, storage tanks, etc., which have mainly vertical and evenly distributed loads.

3.3 Indian Standard Special Publications (SP):

For the clarification and explanation for the clauses and equations mentioned in Indian Standard Codes, Bureau of Indian Standard has published some special publications including charts and tables for required values like material properties and explaining examples of designs. Following design aids will be used for the design of the structure:

3.4 SP 16: Design Aids for Reinforced Concrete to IS 456-2000:

This handbook explains the use of formulae mentioned in IS 456 and provides several design charts and interaction diagrams for flexure, deflection control criteria, axial compression, compression with bending and tension with bending for rectangular cross-sections (for Introduction 15 circular section in case of compression member) which can greatly expedite the design process if done manually. This aid is particularly useful for the preliminary design.

A. SP 22: Explanatory Handbook on Codes for Earthquake Engineering (IS 1893: 1975 and IS 4326: 1976):

The theoretical background behind many of the code provisions have been elaborated herein. Additionally, many worked out examples explaining the use of equations and charts in the code can also be found in this handbook.

B. SP 24: Explanatory Handbook on Indian Standard Code of Practice for Plain and Reinforced Concrete IS 456: 2000

SP 16 is meant to aid the calculation process, while SP 24 is meant to aid the conceptual understanding of the IS 456 code. It contains clause by clause explanation of the original code. The logic and justification behind the various equations and assumptions in the code are well explained here.

C. SP 34: Handbook on Concrete Reinforcement and Detailing:

The compilation of provisions and guidelines regarding reinforcement detailing scattered throughout IS codes 456, 4326, 5525 and 13920 can be found in this handbook. Searching for that information in the original codes can be very time consuming. This handbook presents all that information in a well-organized manner.

3.5 Nepal Health Infrastructure Development Standards 2074 B.S

Nepal Health Infrastructure Development Standards sets the standards for the construction of the hospital building in the various governmental levels of the country. This also explains about the minimum facilities and the equipment and machines required at different level hospitals. This standard forms the basis for rational planning for establishing and upgrading of health institutions on the basis of such factors as accessibility, catchment population, geography, availability of suitable land, condition of existing or nearby facilities and morbidity statistics and thereby, reducing haphazard construction of new or upgrading of existing facilities. Health institutions are classified into five levels based on a minimum set of health services: community level (Health Posts or Community Health Units); Primary Hospitals; Secondary Hospitals; Tertiary Hospitals; Academic or Super-specialty hospitals.

Catchment population and geography will form the basis for assigning number of beds and identifying the required number of health workers. Standard drawings for each type of health institution are developed that facilitate delivery of quality health services such as, attached bathroom in delivery room, an OPD room with privacy concerns of clients, while also mandating specifications for wiring, piping and flooring that minimize infections and reduce cost-of-ownership. These standards will improve the quality of buildings built, reduce times for completion of construction projects and promote the use of economical and locally available construction materials. As custodian of all construction of health institutions, effective implementation of this standard will require enhanced cooperation with Department of Urban Development and Building Construction (DUDBC).

3.6 Guidelines & Main Requirements in the Planning and Design of Hospitals

A. Planning General Facilities of Hospital

The basic consideration in planning should be to have:

a. The distances to be travelled by a nurse from bed areas to treatment room, pantry, etc., should be kept minimum.
b. A fully visible patient's area with adequate space all round for positioning of specialized equipment,
c. A central nurse's station with minimum possible walking distance,
d. Distinct clean and dirty utility area where movement of staff and supplies could be minimized,
e. The ward unit may be made of desired number of beds at the rate of 7 sq. bed and
f. The beds should be arranged with a minimum distance of 2.25 m between center of two beds and a clearance of 200 mm between the bed and wall.
g. In wards, the width of doors shall not be less than 1.2 m.
h. Wards should be relegated at the back to ensure quietness and freedom from unwanted visitors.

B. Functional Planning

Functional planning is an analytical process in hospital planning and development which includes definition of functional requirements, area requirements and workflow to meet the needs and priorities of the medical program.

In consideration of the medical program, the hospital is to have a balanced combination of the following functional areas and services:

- Entrance area,
- Ambulatory care area,
- Diagnostic services,
- Intermediate care area,
- Intensive care area,
- Critical care area,
- Therapeutic services,
- Hospital services,
- Administrative/Ancillary

Table 3-1: Number of Hospital Beds per Category of Wards

Category of Wards	**No. of Beds**
General ward 1 (Medical) including allied specialty	30
General ward 2 (Surgical) including allied specialty	30
Private ward (AC and Non-AC) (optional)	9
Maternity ward	15
Paediatrics ward	6
Intensive care services	4
Critical care services	6
Total	100

NOTE — the number of beds given may be suitably adjusted by hospital administration depending upon local requirements

C. Area Requirements

Area requirement for hospital is to be derived from carpet area of various services and functions. Land requirement depends on factors like, horizontal or vertical development, FAR (floor area ratio) regulations and ground coverage regulated by local self-government/municipal regulations correlated to availability of land. Area requirement can thus be calculated with the above parameters assumed as under:

Table 3-2: Calculation of Area Requirement for Hundred Bed Hospital

Total hospital beds	100
Number of storeys	3 (By placing 40 percent of area on ground floor and remaining in 2 upper floors)
Municipal regulations	(*can vary from Municipal to Municipal)
F.A. R	2.5
Ground coverage permitted	50%
Covered area per bed	92.5 sq.m
Total covered area	92.5×100=9250 sq.m

D. Site Planning

Hospital sites with high degree of sensitivity to outside noise should be avoided, but may be compatible with other considerations, such as, accessibility and availability of services. The buildings should be so planned that sensitive areas, like, wards, consulting and treatment rooms and operation theatres are placed away from the outdoor source of noise.

E. Building Requirements

i. Circulation Areas

Circulation areas, such, as, corridors, staircases, etc., in the hospital buildings should not be more than 40 percent of the total floor area of the building.

ii. Floor Height

The height of all the rooms in the hospital should not be less than 3.00 m measured at any point from the surface of the floor to the lowest point of the ceiling. The minimum head-room, such as, under the bottom of beams, fans and lights shall be 2.50 m measured vertical under such beam, fan or light.

Rooms shall have one or more apertures opening directly to the external air or into an open verandah for ventilation and lighting. The minimum aggregate areas of such openings excluding doors, inclusive of frames, shall be not less than 20 percent of the floor area, in case such apertures are located in one wall and not less than 15-percent of the floor area, in case such apertures are located in two opposite walls at the same sill level.

iii. Circulation

Normally there are three types of traffic flow, namely, (a) patients, (b) staff, and (c) supplies. All these should be properly channelized. Spaces shall be wide enough for free movement of patients, whether they are on beds, stretchers, or wheelchairs. Circulation routes for transferring patients from one area to another shall be available and free at all times.

- Corridors for access shall have a minimum width of 2.44 meters.
- Corridors in areas not commonly used for bed, stretcher and equipment transport may be reduced in width to 1.83 meters.
- A ramp or elevator shall be provided for ancillary, clinical and nursing areas located on the upper floor.
- A ramp shall be provided as access to the entrance of the hospital not on the same level of the site.
- Patients are brought from the ward and should not cross the transfer area in their ward clothing which is a great source of infection. Change-over of trolleys should be affected at a place which will link up both pre and post-operative rooms.

iv. Zoning

The different areas of a hospital shall be grouped according to zones as follows:

a. Outer Zone – areas that are immediately accessible to the public: emergency service, outpatient service, and administrative service shall be located near the entrance.

b. Second Zone – areas that receive workload from the outer zone: laboratory, pharmacy, and radiology. They shall be located near the outer zone.
c. Inner Zone – areas that provide nursing care and management of patients: nursing service. They shall be located in private areas but accessible to guests.
d. Deep Zone – areas that require asepsis to perform the prescribed services: surgical service, delivery service, nursery, and intensive care. They shall be segregated from the public areas but accessible to the outer, second and inner zones
e. Service Zone – areas that provide support to hospital activities: dietary service, housekeeping service, maintenance and motor pool service, and mortuary. They shall be located in areas away from normal traffic.

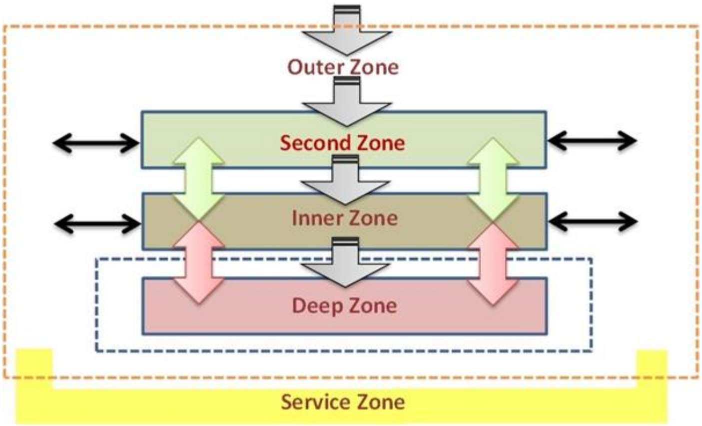


Figure 3-1: Zoning of Hospital Building

v. Function

The different areas of a hospital shall be functionally related with each other;

a. The emergency service shall be located in the ground floor to ensure immediate access. A separate entrance to the emergency room shall be provided.
b. The administrative service, particularly admitting office and business office, shall be located near the main entrance of the hospital. Offices for hospital management can be located in private areas.
c. The surgical service shall be located and arranged to prevent non-related traffic. The operating room shall be as remote as practicable from the entrance to provide asepsis. The dressing room shall be located to avoid exposure to dirty areas after changing to surgical garments.
d. The delivery service shall be located and arranged to prevent non-related traffic. The delivery room shall be as remote as practicable from the entrance to provide

asepsis. The dressing room shall be located to avoid exposure to dirty areas after changing to surgical garments. The nursery shall be separate but immediately accessible from the delivery room.

e. The nursing service shall be segregated from public areas. The nurse station shall be located to permit visual observation of patients. Nurse stations shall be provided in all inpatient units of the hospital.
f. In wards the rooms shall be of sufficient size to allow for work flow.
g. In wards the toilets shall be immediately accessible from rooms.
h. The dietary service shall be away from morgue.
i. Mortuary should be so located that the dead bodies can be transported unnoticed by the general public and patients. Relatives and mourners should have direct access to the mortuary.

4. METHODOLOGY

4.1 Planning Phase

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

4.1.1 Functional Planning

a. Client requirement is the main governing factor for the allocation of space required which is based upon its purposes. Thus, demand, economic status and taste of owner features the plan of building.
b. Building design should favor with the surrounding structures and weather so that it is relevant and is energy efficient.
c. Building is designed by following of building codes, municipal by laws and guidelines.

4.1.2 Structural Planning

The structural arrangement of building is chosen so as to make it efficient in resisting vertical and horizontal load. The material of the structure for construction should be chosen in such a way that the total weight of structure will be reduced so that the structure will gain less inertial force (caused during earthquake). The regular geometrical shape building is designed as an earthquake resistant structure based on NBC 105:2020.

4.2 Load Assessment

Once the detailed architectural drawing of building is drawn, next step is assessment of load on the building. The building is subjected to different kinds of loads. The types of the loads are found out and the calculation of the load is done. The loads on building are categorized as gravity load and lateral load.

4.2.1 Gravity Load

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

a. IS 875 (part 1):1987 Dead Loads
b. IS 875 (part II):1987 Imposed Loads

4.2.2 Lateral Load

Lateral load includes wind load and earthquake load. Wind load acts on roof truss while an earthquake act over the entire structure. Wind load calculation is based on IS 875 (part III):1987 and earthquake on NBC 105:2020. The dominant load is taken into consideration for design.

4.2.3 Load Combination

Combination of different loads is based on NBC 105:2020 Load combinations for Limit State Method.

4.3 Preliminary Design

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

Normally preliminary size will be decided considering following points:

a. Slab: The thickness of the slab is decided on the basis of span/d ratio assuming appropriate modification factor.
b. Beam: Generally, width is taken as that of wall i.e., 230 or 300 mm. The depth is 34 generally taken as 1/12-1/15 of the span.
c. Column: Size of column depends upon the moments from both direction and the axial load. Preliminary Column size may be finalized by approximately calculation of axial load and moments.

4.4 Idealization of Structure

4.4.1 Idealization of Support

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

4.4.2 Idealization of Load

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

4.4.3 Idealization of Structural System

Initially individual structural elements like beam, column, slab, staircase, footing, etc. are idealized. Once the individual members are idealized, the whole structural system is idealized to behave as theoretical approximation for first order linear analysis and corresponding design. The building is idealized as unbraced space frame. This 3D space frame-work is modelled in ETABS for analysis with several load case and load combination.

4.5 Modeling and Analysis of Structure

4.5.1 Salient Features of ETABS

ETABS is a powerful and user-friendly software offering range of features for analysis and design of building structures with comprehensive analysis capabilities, 3D modeling tools, intuitive interface, design features, advanced visualization, and collaboration capabilities.

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

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

The building is modelled as a 3D bare frame. Results from analysis are used in design of beams and columns only (i.e., linear elements). Joints are defined with constraints to serve as rigid floor diaphragm and hence slabs are designed manually as effect of seismic load is not seen on slab. The linear elements are also designed primarily by hand calculation to familiarize with hand computation and exude confidence where we are unable to trust fully on design results of ETABS. This has been done as we are quite unfamiliar with fundamentals of FEM analysis techniques based on which the software package performs analysis and gives results. As we are working with a computer-based system, the importance of data input is as important as the result of output derived from analysis. Hence with possibility of garbage in-garbage-out, we check our input parameters in explicit detail.

Material properties are defined for elements in terms of their characteristic strength i.e., M25 for slabs, beams and columns. Also, section properties are defined as obtained from preliminary design. Loading values are input as obtained from IS 875:1987. Loading combination based on NBC 105:2020 for limit state method is prepared. An envelope load case of all load combinations was used to provide us with the envelope of stresses for design.

4.6 Design Philosophy

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

4.7 Detailing Principle for Reinforced Concrete and Steel Structures

4.7.1 Ductile Detailing of Reinforced Concrete Structure

Ductile detailing of reinforced concrete structure is done based on IS 13920:2016 for the 37 provisions of compliance with earthquake resistant design philosophy. Special consideration is taken in detailing of linear frame elements (BEAMS & COLUMNS) to achieve ductility in the concrete to localize the formation of plastic hinge in beams and not columns to assure the capacity theory of STRONG COLUMN WEAK BEAM. Detailing provisions of IS 13920:2016 and IS 456:2000 is used extensively for these members.

4.7.2 Ordinary Detailing of Reinforced Concrete Structure

SP 34 detailing handbook for IS 456 is used extensively for reinforcement detailing of area elements (slabs and staircase). Defining the slabs to function as rigid floor diaphragm limits the necessity of special reinforcement provision for slabs eliminating the possibility of out-of-plane bending. Hence same follows for staircase slabs and detailing is done with the help of SP34. Detailing of substructures (isolated footing) is also done based on SP34 to comply with the design requirement of IS 456:2000. Reinforcement Detail drawings for typical representative elements are shown in detail on structural drawings.

4.7.3 Codal References

The project report has been prepared in complete conformity with various stipulations in Nepal Building Code NBC 105:2020, Indian Standards, Code of Practice for Plain and Reinforced Concrete IS 456:2000, Design Aids for Reinforced Concrete to IS 456:2000 (SP-16), Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces- Code of Practice IS 13920:2016, Handbook on Concrete Reinforcement and Detailing SP-34. Use of these codes have emphasized on providing sufficient safety, economy, strength and ductility besides satisfactory serviceability requirements of cracking and deflection in concrete structures. These codes are based on principles of Limit State of Design.

4.8 Drawings

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

a. Architectural Plan of Typical floors, Elevation and Cross Section of the building.
b. Detailed Structural drawing of full-size beam, column, slab, staircase, shear wall and isolated footing.

5. FUNCTIONAL AND STRUCTURAL PLANNING OF BUILDING

5.1 Functional Planning

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

5.1.1 Planning of Space and Facilities

Planning of space and approximate sizing of rooms for different departments of the hospital was done as per ANNEX A of IS 12433 (Part 2) :2001 Basic Requirements for Hospital Planning. For vertical mobility, two open well staircase and one ramp are provided. Lifts were provided in the middle part of the building. In terms of space management for elevator, slabs with openings were provided as shown in the building plan diagram.

Each floor of the building has separate washrooms for men and women, including dedicated staff washrooms. Male and female patients are accommodated in separate general wards. The building has a central open hall with a large truss, for natural lighting and ventilation.

OPD and pharmacy are placed nearby the entrance. Cafeteria is placed adjacent to waiting room to facilitate patient and visitors. And the functional amenities are used for load assessment and ignoring their aesthetic which is beyond the scope of this project.

5.1.2 Architectural Planning of 3D Framework of Building

The building to be designed is a multistory R.C.C hospital building. For reinforced concrete frames, a grid layout of beams is made considering the above functional variables. Columns are placed in most of grid intersection points and beams are arranged interconnecting the columns. This framework for each floor is then utilized with positioning of masonry wall between the columns. A total of 111 numbers of columns are provided in ground floor. There is provision of expansion joint, the justification for which is presented in detail in following subheadings. With this framework of beam and column having R.C.C slab in the floor and curved truss on the roof, architectural planning of the building is complete and 3D framework is thus complete.

5.1.3 Compliance to Municipal By-Laws

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

a. Type of Building
b. Land Area Available
c. Maximum height of the building, etc.
d. Maximum Ground Coverage (GCR)
e. Floor Area Ratio (FAR)

These variables are also dictated by specific location of site in different wards. A more comprehensive knowledge about such provisions can be referred in detail at the referenced publication. This completes the overall functional planning of the building with coverage of maximum number of variables in preliminary stage planning.

5.2 Structural Planning

5.2.1 Structural System

Our focus in the current section is the structural orientation of the building in vertical and horizontal plane avoiding vertical and plan irregularities mentioned in NBC 105:2020. The following types of irregularities mentioned in 5.5 Structural Irregularity of NBC 105:2020. should be avoided as far as practicable during functional planning.

Vertical Irregularities:

1. Weak Story
2. Soft Story
3. Vertical Geometric Irregularity
4. In-Plane Discontinuity in Vertical Lateral Force Resisting Element Irregularity
5. Mass Irregularity

Plan Irregularities:

1. Torsion Irregularity
2. Re-entrant Corners Irregularity
3. Diaphragm Discontinuity Irregularity
4. Out of plane offset Irregularity

To prevent the irregularities and to preserve the structural integrity of the entire building following measures were implemented in planning of the building:

a. **Regularity**: Buildings is planned with regular shapes as they are generally easier to design for seismic resistance than irregular structures. Regularity in building layout and floor plan can help to ensure that seismic forces are evenly distributed throughout the structure.

b. **Separation of blocks**: Buildings are divided into three separate blocks or wings as they are better in withstanding earthquakes than those with large, continuous spans. This is because the blocks can move independently of each other, which can help to reduce the overall stresses placed on the structure.
c. **Simplicity:** Complex architectural features, such as cantilevers, arches, or unsupported corners, can create stress concentrations that can increase the risk of damage during earthquake. So, the building planning is kept simple.
d. **Enclosed Area**: A small building enclosure with properly interconnected walls acts like a rigid box. Since, the earthquake strength which long walls derive from transverse walls increases as their length decreases. So, separately enclosed rooms rather than one long room are planned. Walls are provided along the perimeter of the building rather than concentrating them in the center of the building for enhanced torsion resistance and additional earthquake protection.
e. **Resisting Element**: Shear walls, are provided evenly throughout the building in both directions side to side, as well as top to bottom to prevent torsion.
f. **Structural Ductility:** Structural ductility refers to the ability of a building to undergo large deformations without collapsing during an earthquake by absorbing energy from the earthquake. In order to achieve structural ductility, buildings must be designed with flexible and resilient structural systems that can absorb and distribute seismic energy. So building is designed with reinforced concrete, which can undergo significant deformation without collapsing.

Structural planning is done over the proposed architectural plan for providing and preserving structural integrity of the entire building. Finalized structural plan is then employed for preliminary design of structural members and load assessment for modeling in ETABS.

5.2.2 Planning of Beam-Column Frame

When creating a frame building, structural member in regard to their stiffness are to be uniformly distributed and these should be well framed up in both orthogonal directions with nearly uniform spans. So, the orientation of beams and column grid in plan is in rectangular shape. This is done to avoid irregularities in plan. Lateral load resisting elements are oriented along mutually orthogonal horizontal directions i.e in parallel system.

6. PRELIMINARY DESIGN AND LOAD ASSESSMENT

6.1 Need of Preliminary Design

It is necessary to know the approximate section of the structure for the detail analysis as the section should be provided initially while analyzing in almost all software. Only dead loads and live loads are considered during preliminary design. Preliminary design is carried out to estimate approximate size of the structural members before analysis of structure. The preliminary sizing of structural elements was carried out based on deflection control criteria and approximate loads obtained using the tributary area method.

6.2 Preliminary Design

6.2.1 Preliminary Design of Slab

The preliminary design of R.C.C slab for the floor and roof of the proposed building is based on fulfillment of deflection control criteria of IS 456:2000.

i. Determination of type:

Designing for the biggest slab section in all blocks

Length along x-axis of slab (L_x)=6096 mm

Length along y-axis of slab (L_y)=6096 mm

L_y/L_x =1<2

Hence, we design for two-way slab.

ii. Calculating preliminary depth using deflection criteria

$d > L_x / (\alpha \times \beta \times \lambda \times \delta \times \gamma)$ (IS 456:2000 Cl 23.2.1)

For α =26 (for continuous slab) (Ref. IS 456:2000 Cl. 23.2.1 a.)

β = span factor = 1 since L_x <10 (Ref. IS 456:2000 Cl. 23.2.1 b.)

δ= 1 for no compression reinforcement in the slab (Ref. IS 456:2000 Cl. 23.2.1 d.)

λ = 1 for slab being rectangular without flange (Ref. IS 456:2000 Cl. 23.2.1 e.)

γ = 1.28 (approximate)

Therefore, $d = 6096/ (26×1.28×1×1×1)$

$= 183.17$ mm >150 mm

(It becomes difficult to cast in field, so secondary beam is provided.)

iii. Calculation for Secondary Beam:

Dividing the longest span into two parts,

$L_x = 6096/2 = 3048$ mm

Here, $L_y/ L_x = 6096/3048 = 2 < 2$

So, this is a two-way slab.

Again, By deflection,

$d = 3048/ (26×1.28) = 91.58 < 150$ mm ok.

Also, let provide clear cover of 15 mm

Total depth (D)= d+(dia./2)+clear cover =92+(10/2)+15= 112 mm

Hence provide overall depth of slab (D)=120 mm

6.2.2 Preliminary Design of Beam

The preliminary design of beam of the proposed building is based on fulfillment of deflection control criteria of IS 456:2000

A. For Main Beam

Taking longest beam along Horizontal grid and vertical grid:

Span length = 6096 mm

So, effective depth (d)= l/12 to l/15

Taking d =l/15=6096/15= 406.4 mm

Total depth (D)=d + d'=406.4+50= 456.4 mm Adopting D= 500 mm

Width of beam (b)=D/2 to 2×D/3

Taking b =2D/3 = 2×500/3= 333.33 mm, Adopt b = 350 mm

Hence, b×D= 350 mm × 500 mm

B. For Secondary Beam

Taking maximum span of beam = 6096 mm

Therefore, Lx =6096/2=3048 mm

Ly =6096 mm

We have from deflection criteria,

$d > Lx / (\alpha \times \beta \times \lambda \times \delta \times \gamma)$

For α =26 (for continuous beam) (Ref. IS 456:2000 Cl. 23.2.1 a.)

β = span factor = 1 since Lx <10 (Ref. IS 456:2000 Cl. 23.2.1 b.)

λ =1 for beam being rectangular without flange (Ref. IS 456:2000 Cl. 23.2.1 e.)

γ =1.28(approximate)

δ =1 for no compression reinforcement in the slab (Ref. IS 456:2000 Cl. 23.2.1 d.)

Therefore, d =3048/ (26×1.28×1×1×1) = 91.58 mm

So, adopt effective depth (d) = 100 mm

Assuming clear cover = 40 mm

And using 20 mm dia. bar,

Overall depth (D)= d+(dia./2)+clear cover

=100+(20/2)+40=150 mm

Therefore, adopt D=300 mm

And b = 2D/3 = 2×300/3 mm = 200 mm

Hence, adopt secondary beam size of 250×300 mm.

6.2.3 Preliminary Design of Column

Location of columns are chosen so that they best represent the best approximation of maximum stressed columns that have the maximum area of loading with respect to number of beams connected into them and the loaded slab area.

Floor height = 12′ = 3.658 m

A. For Intermediate Column I8

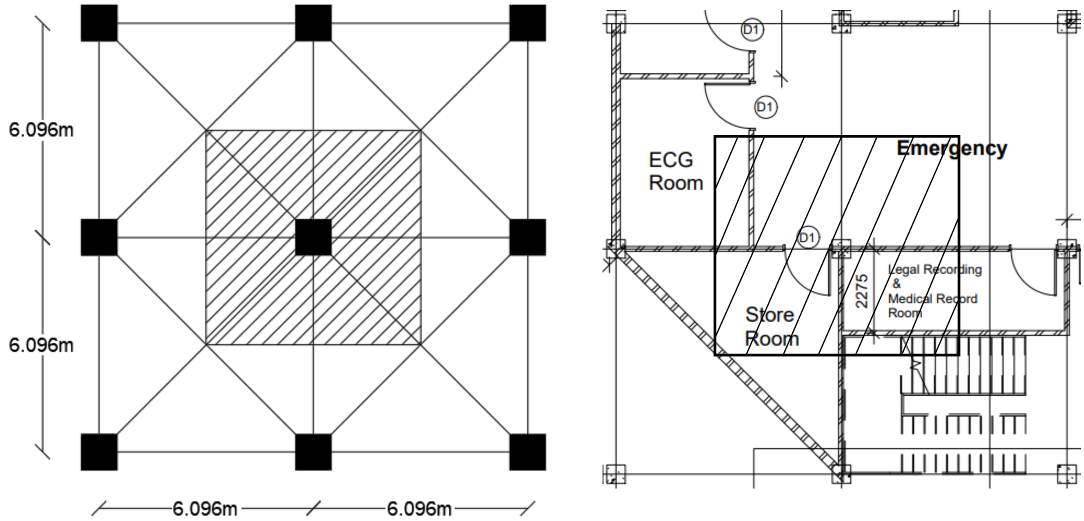


Figure 6-1: Tributary Area for Intermediate Column I8

Area of each shaded triangle = 1/2× (6.096/2)× (6.096/2)=4.645 m^2

Influence Area = 8×4.645 = 37.16 m^2

i. Dead load calculation:

Dead load (DL)= γ x D x Area of the section

DL of Slab (120 mm thick) = 25×0.12×37.16 = 111.48 kN

(γ of Concrete =25 kN/m^3)

DL of Ceiling Plaster (12.5 mm thick) = 20.4×0.0125×37.16 = 9.476 kN

(γ of Cement Plaster =20.4 kN/m^3 , Ref: IS 875 Part I Page 8)

DL of Screed with punning (25 mm thick) = 21×0.025×37.16 = 19.509 kN

(γ of Screed =20.4 kN/m^3 but for Screed with punning taking γ =21 kN/m^3 (assumed), Ref: IS 875 Part I Page 8)

DL of Floor Finish (Marble 25 m thick) = 26.5×0.025×37.16 = 24.619 kN

(γ of Marble dressing = 26.5 kN/m^3 Ref: IS 875 Part I Page 10)

DL of Main beam = 25× (6.096+6.096) ×0.35× (0.5-0.12) = 40.538 kN

DL of Secondary beam = 25× (6.096+6.096) ×0.25× (0.3-0.12) = 13.716 kN

DL of Partition wall (115 mm thick) = 18.85×(6.096+6.096)×0.115×(3.658-0.12)

= 93.507 kN

(γ of Common Brunt Brick =18.85 kN/m^3 Ref: IS 875 Part I Page 10)

Assuming deduction for opening = 30%

Total wall load = 0.7×93.507 = 65.455 kN

Total Dead load on one intermediate column = 111.48 + 9.476 + 19.509 + 24.619 + 40.538 + 13.716 + 65.455 = 284.783 kN

For three Storey building,

Total Dead Load = 3×2384.783 = 854.349 kN

ii. Live load calculation

For Institutional (Hospital) Building = 3.0 kN/m^2

Live load (LL) =Area × Load per unit area = 37.16×3 = 111.48 kN

LL on Ground Floor = 111.48 kN

LL on 1st Floor = 0.9×111.48 = 100.332 kN

LL on 2nd Floor = 0.8×111.48 = 89.184 kN

(For reduction in Live Load for Floors, Ref: IS 857-Part II Cl.:3.2.1, Page 12)

For curved roof with slope of line obtained by joining springing point to the crown with the horizontal, greater than 10-degree, Ref IS 857-Part II, Page 14)

Rise of Truss (r) = 1.841 m and Span of truss (w) = 12.192 m

Live load = $0.75-0.52\times (r/w)^2 = 0.75-0.52\times (r/w)^2 = 0.738$ kN/m^2

LL on 3th Floor (Roof Floor) = 37.16× 0.738 = 27.424 kN

Total Live load on Intermediate Column = 111.48+100.332+89.184+27.424 = 328.42 kN

Total load on Intermediate Column = 854.349 + 328.42 = 1182.769 kN

Total factored load on Intermediate Column = 1.5×1182.769 = 1774.154 kN

iii. Calculation of column section

Taking 3% steel reinforcement, M25 grade concrete and Fe500 grade rebar. Providing increment of 10% as allowance for bending due to effect of fixity.

From IS 456:2000, Clause 39.3, Page 71 we have,

$P_u = 0.4f_{ck}A_c + 0.67f_yA_s$

$1.1\times1774.154\times10^3 = 0.4\times25\times(1\text{-}0.03)\times A_g + 0.67\times500\times0.03\times A_g$

$A_g = 98813.641\ mm^2$

Choosing a square section,

Adopt Size of column = 350 mm ×350 mm

B. For Face Column F13

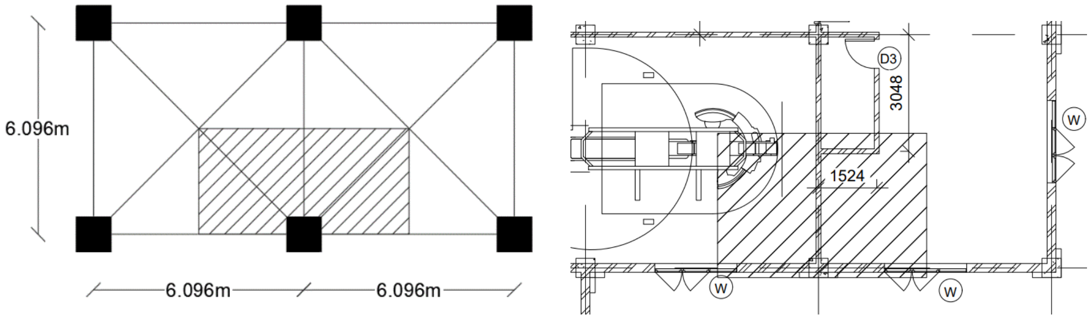


Figure 6-2: Tributary Area for Face Column F13

Area of each shaded triangle = 1/2 × (6.096/2) × (6.096/2)= 4.645 m^2

Influence Area = 4×4.645 = 18.58 m^2

i. Dead load calculation

Total Dead load on one face column = 284.783/2= 142.392 kN

For three Storey building,

Total Dead Load = 3×142.392 = 427.176 kN

ii. Live load calculation

Total Live load on Intermediate Column = 328.42/2=164.21 kN

Total load on Intermediate Column = 427.176+ 164.21 = 591.386 kN

Total factored load on Intermediate Column = 1.5×591.386 = 887.079 kN

iii. Calculation of column section

Taking 3% steel reinforcement, M25 grade concrete and Fe500 grade rebar. Providing increment of 15% as allowance for bending due to effect of fixity. From IS 456:2000, Clause 39.3, Page 71 we have,

$P_u = 0.4f_{ck}A_c + 0.67f_yA_s$

$1.15 \times 887.079 \times 10^3 = 0.4 \times 25 \times (1-0.03) \times Ag + 0.67 \times 500 \times 0.03 \times A_g$

$A_g = 51652.701\ mm^2$

Choosing a square section,

Adopt Size of column = 250 mm ×250 mm

C. For Corner Column C12

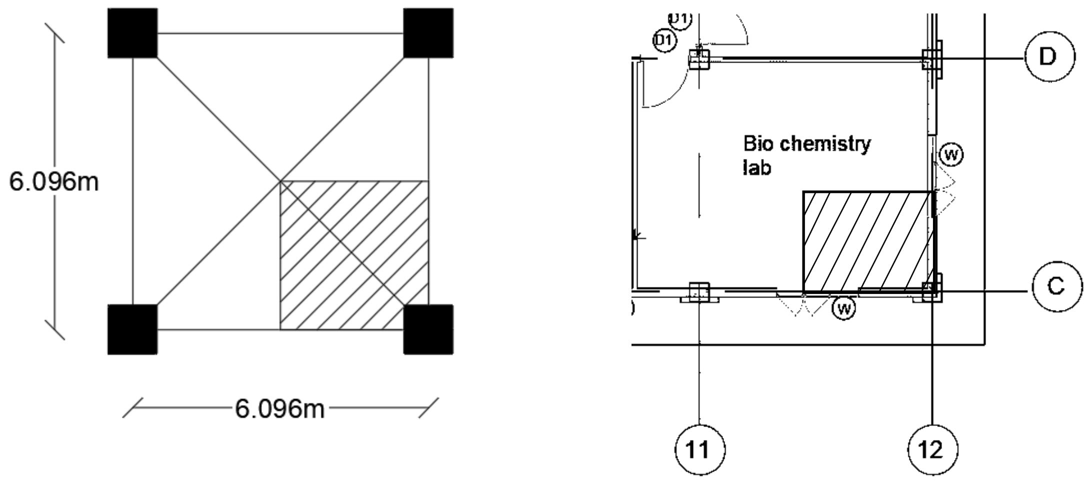


Figure 6-3: Tributary Area for Corner Column C12

Area of each shaded triangle = 1/2 × (6.096/2)× (6.096/2)= 4.645 m^2

Influence Area = 2×4.645 = 9.29 m^2

i. Dead load calculation

Total Dead load on one face column = 284.783/4= 71.196 kN

For three Storey building,

Total Dead Load = 3×71.196 = 213.588 kN

ii. Live load calculation

Total Live load on Intermediate Column = 328.42/4=82.105 kN

Total load on Intermediate Column = 213.588+ 82.105= 295.693 kN

Total factored load on Intermediate Column = 1.5×295.693 = 443.540 kN

iii. Calculation of column section

Taking 3% steel reinforcement, M25 grade concrete and Fe500 grade rebar. Providing increment of 30% as allowance for bending due to effect of fixity. From IS 456:2000, Clause 39.3, Page 71 we have,

$P_u = 0.4f_{ck}A_c + 0.67f_yA_s$

$1.3\times443.540\times10^3 = 0.4\times25\times(1-0.03)\times Ag + 0.67\times500\times0.03\times A_g$

$A_g = 29195.038\ mm^2$

Choosing a square section,

Adopt Size of column = 200 mm ×200 mm

6.3 Material Properties

A. Concrete

M25 grade concrete in used except in some places where the grade is specified as required for the cases and conditions involved.

Modulus of elasticity [Ec] $=5000\sqrt{f_{ck}}\ N/mm^2$ (Cl. 6.2.3.1, IS 456:2000)

$=5000\sqrt{25}\ N/mm^2 = 25000\ N/mm^2$

Characteristics strength [f_{ck}]= 25 N/mm²

The partial factor for concrete in flexure and axial loading is 1.5.

B. Reinforcement Steel

Characteristics strength of high yield steel is taken as 500 MPa for main rebar and shear rebar and material partial safety factor is to be 1.15.

Modulus of Elasticity [Es]=2×10^5 N/mm².

6.4 Load Assessment

The unit weight of different structural and non-structural elements are derived from IS 875 Part I and presented in the load calculations are based on actual measured drawings. The self-weight of beams, columns and slabs are calculated by the program itself. The values of imposed loads are tabulated as below:

- The weight of infill walls is calculated as applied on beams as line weight in kN/m.
- Partition wall load are applied on null beam element
- Floor finishing load are assigned as area load in slabs.
- Live load is assigned in each panel of slab corresponding of their area type.
- A frame load is applied as parapet loading on the exterior frame of the roof level.
- The curved roof is assumed inaccessible and the roof live load is applied as per Indian Standard, IS 875-1987(Part-2) but this load is not considered during seismic load. Dead load of roof cover Corrugated GI sheet is applied on the roof truss.
- Wind load is applied on the roof as per IS 875 (Part-3) :1987

6.4.1 Gravity Load Assessment on the Building

6.4.1.1 Unit Weight (Dead Load)

Table 6-1: Unit Weight of Materials

Material Used	**Unit Weight**	**Type of Member**	**Reference IS: 875 (Part 1)-1987**
Cement Concrete for RCC	25 kN/m^3	Beams, Columns, Slabs.	Table-1 Item no. 22
Common Burnt Clay Bricks	19.2 kN/m^3	Infill & Partition Walls	Table-1 Item no. 36
Screed on floor	20 kN/m^3	All flooring spaces	Table-1 Item no. 42
Flooring	0.22 kN/m^3	All flooring spaces	Table-2 Item no. 7

6.4.1.2 Imposed Load (Live Load)

The magnitude of live load depends upon the type of occupancy of the building. These are to be chosen from code Table 1 of IS 875:1987 (part 2) for various occupancies, for this project live loads are taken for institutional building. The live load distribution varies with time. Hence each member is designed for worst combination of dead load and live loads.

Table 6-2: Live Load for Hospital Building

S.N	Area Type (Occupancy Classification)	Uniformly Distributed Load
1.	Bed rooms, wards, dressing rooms, dormitories and lounges	2 kN/m^2
2.	Kitchen and laundry	3 kN/m^2
3.	Dining rooms, restaurants and cafeterias	3 kN/m^2
4.	Toilets and bathrooms	2 kN/m^2
5.	X-ray rooms, operating rooms, general storage	3 kN/m^2
6.	Office rooms and OPD rooms	2.5 kN/m^2
7.	Corridors, passages, lobbies	4 kN/m^2
8.	Ramp	5 kN/m^2
9.	Boiler rooms and plant rooms	5 kN/m^2

6.4.1.3 Wall Load Calculation

Full brick wall thickness = 0.23 m

Half brick wall thickness = 0.115 m

Floor to floor height = 3.658 m

Beam depth = 0.5 m

Slab thickness = 0.120 m

Height of wall resting on beam = 3.658-0.5= 3.158 m

Height of wall resting on slab = 3.658-0.12= 3.538 m

Unit weight of brick masonry = 19.2 kN/m^3 IS: 875 (part 1)-1987, Table-1 item no. 36

Parapet wall height = 1 m

Table 6-3: Wall Load Calculation

<table>
<tr><td rowspan="2">Wall</td><td colspan="2">WL (without opening)
Unit: kN/m</td><td colspan="2">With 25% opening
Unit: kN/m</td><td>Parapet wall load
Unit: kN/m</td></tr>
<tr><td>230 mm</td><td>115 mm</td><td>230 mm</td><td>115 mm</td><td>115 mm</td></tr>
<tr><td>Resting on beam</td><td>=0.23×3.158×19.2
=13.95</td><td>=0.115×3.158×19.2
=6.97</td><td>=0.75×13.95
=10.463</td><td>=0.75×6.97
=5.228</td><td rowspan="2">=1×0.115×19.2
2.208</td></tr>
<tr><td>Resting on slab</td><td>=0.23×3.538×19.2
=15.62</td><td>=0.115×3.538×19.2
=7.81</td><td>=0.75×15.62
=11.715</td><td>=0.75×7.81
=5.858</td></tr>
</table>

6.4.1.4 Area Load Calculation

A. Floor finish

Thickness of screeding =50 mm

Unit weight of screeding/plaster =20 kN/m^3 IS: 875 (Part 1)-1987, Table-1 Item no. 42

Unit weight of Marble dressing = 26.5 kN/m^3 IS: 875 (Part 1)-1987, Table-1 Item no.37

Thickness of plaster on ceiling =12.5 mm

Area considered =1 m^2

Floor finish load

Screeding = 20 × (50/1000) = 1 kN/m^2

Ceiling plaster = 20 × (12.5/1000) = 0.25 kN/m^2

Flooring (Marble 10 m thick) = 26.5 × (10/1000) = 0.265 kN/m^2

Total floor finish load = 1+0.25+0.265 = 1.515 kN/m^2

B. Water Tank Load

Slab length =6.069 m

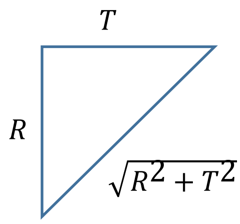
Slab breadth =3.048 m

Slab area =6.069×3.048= 18.498 m^2

Water tank capacity = 2000 ltr = 20 kN

Water tank load =20/18.498 = 1.081 kN/m^2

C. Waist slab (due to steps)



Riser (R)=0.178 m

Tread (T)=0.254 m

Width of stair =1.118 m

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

Load on Waist Slab= $\frac{R \times T}{2 \times \sqrt{R^2+T^2}}$ x unit weight of concrete = $\frac{0.178 \times 0.254}{2 \times \sqrt{0.178^2+0.254^2}} \times 25 = 1.822\ kN/m^2$

6.4.2 Lateral Force Calculation

6.4.2.1 Horizontal Base Shear Coefficient

For hospital building in Pokhara;

Table 6-4: Base Shear Coefficient Calculation for Block A and Block B

Zone Factor	Z=	0.3	Clause 4.1.4
Importance Factor	I=	1.5	Clause 4.1.5
	Soil type:	C	Clause 4.1.3.3
Height of Building	H=	14.632 m	
For Moment Resisting Concrete Frame	K_t =	0.075	Clause 5.1.2

Time Period	$T= 1.25\times K_t\times H^{0.75}$	0.701 sec	Clause 5.1.2 & 5.1.3
For Equivalent Static Method	Ta =	0	Table 4-1
	Tc =	1	Table 4-1
	α=	2.5	Table 4-1
	K=	1.8	Table 4-1
Spectral Shape Factor	Ch (T)	2.5	Clause 4.1.2
Elastic Site Spectra	C(T)=Ch (T)×Z×I	1.125	Clause 4.1.1
Elastic Site Spectra for serviceability limit state	Cs (T)=0.2×C(T)	0.225	Clause 4.2
Horizontal Base Shear Coefficient For Moment Resisting Frame Systems (Reinforced Concrete Moment Resisting Frame)	(i) For SLS		Clause 5.4.2
	Rs =	1	Clause 5.3.2
	Ωu =	1.5	Table 5-2
	Ωs =	1.25	Table 5-2
	Cd (T1)= Cs (T1)/Ωs =	0.1800	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)	0.1875	Clause 6.1.1

Building Height exponent	k =	1.10	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/Rs	0.006	Clause 8.1.3.1
Allowable story displacement			
(i)	For ULS: 0.025×(H/Ru)	91.450 mm	Clause 5.6.1.1
(ii)	For SLS: 0.006×(H/Rs)	87.792 mm	Clause 5.6.1.2

Table 6-5: Base Shear Coefficient Calculation for Block C

Zone Factor	Z=	0.3	Clause 4.1.4
Importance Factor	I=	1.5	Clause 4.1.5
	Soil type:	C	Clause 4.1.3.3
Height of Building	H=	10.974 m	
For Moment Resisting Concrete Frame	K_t =	0.075	Clause 5.1.2
Time Period	$T = 1.25 \times K_t \times H^{0.75}$	0.565 sec	Clause 5.1.2 & 5.1.3
For Equivalent Static Method	Ta =	0	Table 4-1
	Tc =	1	Table 4-1

	α=	2.5	Table 4-1
	K=	1.8	Table 4-1
Spectral Shape Factor	Ch (T)	2.5	Clause 4.1.2
Elastic Site Spectra	C(T)=Ch (T)×Z×I	1.125	Clause 4.1.1
Elastic Site Spectra for serviceability limit state	Cs (T)=0.2×C(T)	0.225	Clause 4.2
Horizontal Base Shear Coefficient For Moment Resisting Frame Systems (Reinforced Concrete Moment Resisting Frame)	(i) For SLS		Clause 5.4.2
	Rs =	1	Clause 5.3.2
	Ωu =	1.5	Table 5-2
	Ωs =	1.25	Table 5-2
	Cd (T1)= Cs (T1)/Ωs =	0.1800	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)	0.1875	Clause 6.1.1
Building Height exponent	k =	1.03	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/Rs	0.006	Clause 8.1.3.1
Allowable story displacement			
(i)	For ULS: 0.025×(H/Ru)	68.588 mm	Clause 5.6.1.1
(ii)	For SLS: 0.006×(H/Rs)	65.844 mm	Clause 5.6.1.2

6.4.2.2 Earthquake Load

Seismic weight is the total dead load plus appropriate amount of specified imposed load. The seismic weight at each level, Wi, shall be taken as the sum of the dead loads and the factored seismic live loads between the mid-heights of adjacent stories. The seismic weight of the whole building is the sum of the seismic weights of all the floors. It has been calculated according to NBC 105:2020 Cl 5.2. The seismic live load shall be determined as given in table below:

Table 6-6: Live Load Categories and Factors

Live Load Category	**Factor (λ)**
Storage	0.6
For Other Purpose	0.3
Roof	Nil

Linear load combination with dead load and factored live load is used to calculate seismic weight from ETABS.

Table 6-7: Lateral Seismic Force Calculation of Block A

Story	Wi	h	V = Cd (T1)x W		Whik	Fi (Lateral Seismic Force)	
			SLS	ULS		SLS	ULS
Story4	321.303	14.632	3304.4	3442.1	6159.54	139.93	145.76
Story3	3759.76	10.974			52514	1192.99	1242.7
Story2	5716.52	7.316			51100.6	1160.88	1209.25
Story1	8560.04	3.658			35680.5	810.572	844.346
Total	18357.6			Total	145455		

Where,

$$F_i = \frac{W_i h_i^k}{\sum_i^n W_i h_i^k} \text{ x } V$$

Table 6-8: Lateral Seismic Force Calculation of Block B

Story	W_i	h	V = Cd (T1)x W		Wh_i^k	F_i (Lateral Seismic Force)	
			SLS	ULS		SLS	ULS
Story4	228.558	14.632	7235.2	7536.6	4381.57	97.4512	101.512
Story3	8641.09	10.974			120694	2684.37	2796.22
Story2	14600.3	7.316			130514	2902.78	3023.73
Story1	16725.4	3.658			69715.7	1550.56	1615.17
Total	40195.3			Total	325305		

Table 6-9: Lateral Seismic Force Calculation of Block C

Story	W_i	h	V = Cd (T1)x W		Wh_i^k	F_i (Lateral Seismic Force)	
			SLS	ULS		SLS	ULS
Story3	1171.94	10.974			13905.5	395.519	411.999
Story2	1375.58	7.316	1069.13	1113.67	10738.3	305.434	318.161
Story1	3392.07	3.658			12944.1	368.173	383.514
Total	5939.59			Total	37587.8		

6.4.3 Scale Factor for Design Values of the Combined Response

In response spectrum analysis, first the individual modal responses are obtained and the responses are combined to approximate the maximum response of the structure. As per NBC 105:2020 Clause 7.4 we could use SRSS or the Complete Quadratic Combination (CQC) method. For this project we have used SRSS method of modal combination.

As per NBC 105:2020 Clause 7.5, when the design base shear (V_R) obtained by combining the modal base shear forces is less than the base shear (V) calculated using Equivalent Static Method; the member forces, story shear forces & base reactions obtained from the modal response spectrum method shall be multiplied by V/V_R. After obtaining design base shear (V) for modal response spectrum method from ETABS, scale factors V/V_R were obtained.

Sample calculation for scale factor for RSx of Block A:

Base reaction obtained for modal response spectrum from ETABS (V_R)= 2.422

Scale factor (V/V_R) = 3442.1 /2.422 =1421.16

Table 6-10: Scale Factor for Design Values of The Combined Response

Block	Scale Factor for RSx	Scale Factor for RSy
Block A	1421.16	1425.28
Block B	3098.56	3051.27
Block C	1009.95	1079.56

6.4.4 Roof Truss Load Assessment

Dead load due to roof covering, live load and wind load acting on the roof were calculated. The calculated loads were applied on purlins of the roof as shown in figure below to obtain support reactions. The reactions were then assigned as joint loads in columns of the building supporting the roof truss.

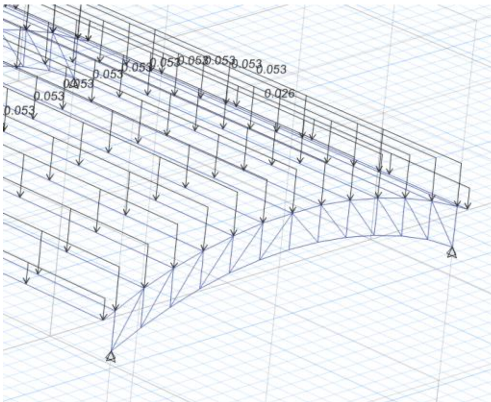


Figure 6-4: Dead Load due to Roof Covering

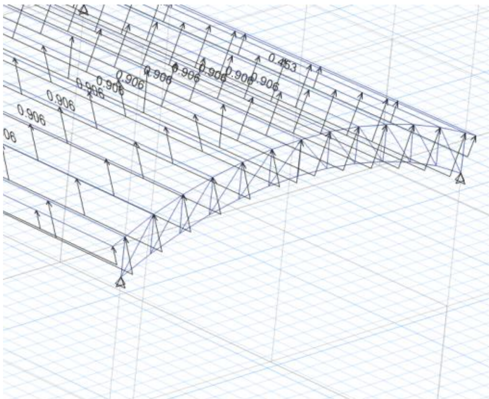


Figure 6-5: Wind Load Acting on Purlin

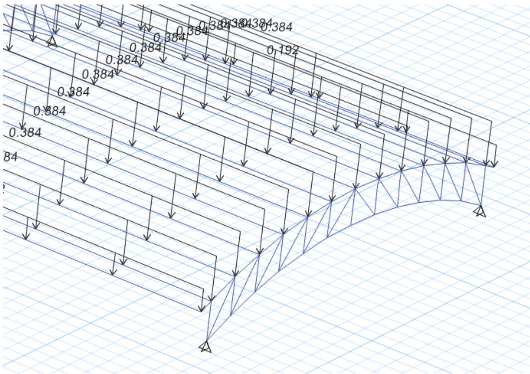


Figure 6-6: Live Load Acting on Roof

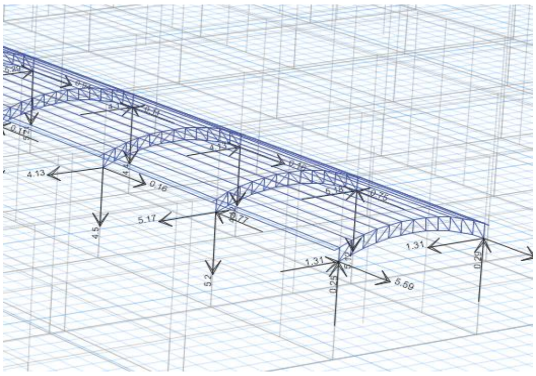


Figure 6-7: Support Reactions Due to Truss Load

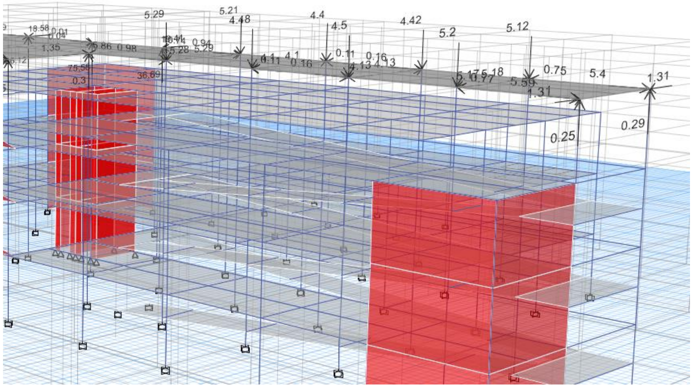


Figure 6-8: Truss Reactions Applied on Joints in Building

Table 6-11: Truss Load Calculation

S.N	Calculation	Reference
1.	**Calculation of slope** We have, Rise of Truss (r) = 1.841 m Span of truss (w) = 12.192 m Height of structure (h) = 14.632 m Spacing of purlin (s) = 0.9 m Slope = $\tan^{-1}(2r/w) = 16.80^{o}$	
2.	**Weight of roof covering (Corrugated GI Sheet)** Roofing load (l) = 0.100 kN/m^2 (Class 4, 1.25mm) Roofing load per m for peripheral purlins = l x s/2 = 0.045 kN/m Roofing load per m for other purlins = l x s = 0.090 kN/m	IS:875 Part 1 Table 1
3.	**Live Load** For curved roof with slope more than 10^{o}, Live load (l) = $0.75-0.52\times (r/w)^2 = 0.738$ kN/m^2 Imposed load for peripheral purlins = l x s/2 = 0.332 kN/m Imposed load for other purlins = l x s = 0.664 kN/m	IS:875 Part 2 Table 2

S.N	Calculation	Reference
4.	**Wind load**	IS:875 Part 3
	(Assumed) Basic Speed (V_b) = 47 m/s	
	Probability factor (k_1) = 1.07	Table 1
	For terrain category 2, building class A and height of 14.632 m	
	Terrain, height and structure size factor (k_2)= 1.0463	Table 2
	Also,	
	Topography factor (k_3)= 1	Annex C
	Design wind speed (V_z) = $V_p \times k_1 \times k_2 \times k_3$ = 52.618 m/s	Clause 5.3
	Design wind Pressure (P_z) = $0.6 \times V_z^2$ = 1.661 kN/m^2	Clause 5.4
	Force per unit area = $(C_{pe}-C_{pi}) \times P_d$	
	For roof on elevated structure,	Table 15
	Rise by span ratio (r/w) = 1.841/12.192 = 0.151	
	C_{pe} = External Pressure Coefficient = -0.9	
	Assuming that cladding permits the flow of air less than 5 percent of the wall area.	Cl 6.2.3.1
	C_{pi} = Internal Pressure Coefficient = ± 0.2	
	F = (-0.9-0.2)×1.661 = -1.827 kN/m^2	
	F = (-0.9+0.2)×1.661 = -1.163 kN/m^2	
	Uplift Pressure = max of above = -1.827 kN/m^2	
	Uplift Pressure per m (l) = -1.645 kN/m	
	Wind load for peripheral purlins = l x s/2 = -0.822 kN/m	
	Wind load for other purlins = lxs = -1.645 kN/m	

7. MODELING AND STRUCTURAL ANALYSIS

7.1 Modeling

From: ETABS

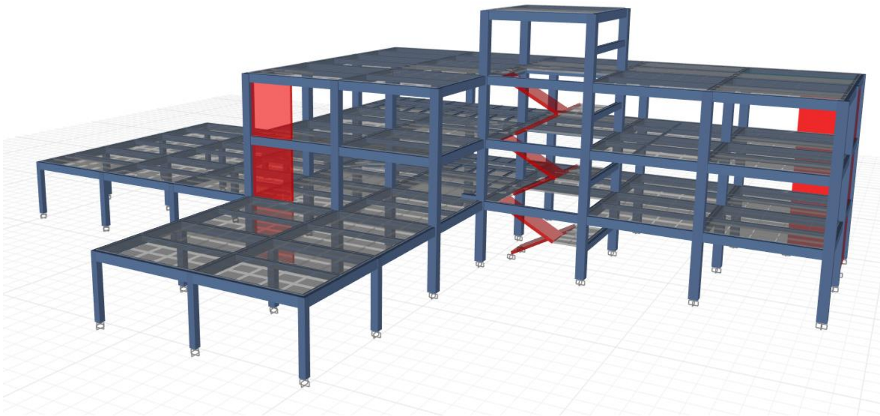


Figure 7-1: 3D Frame Model of Block A

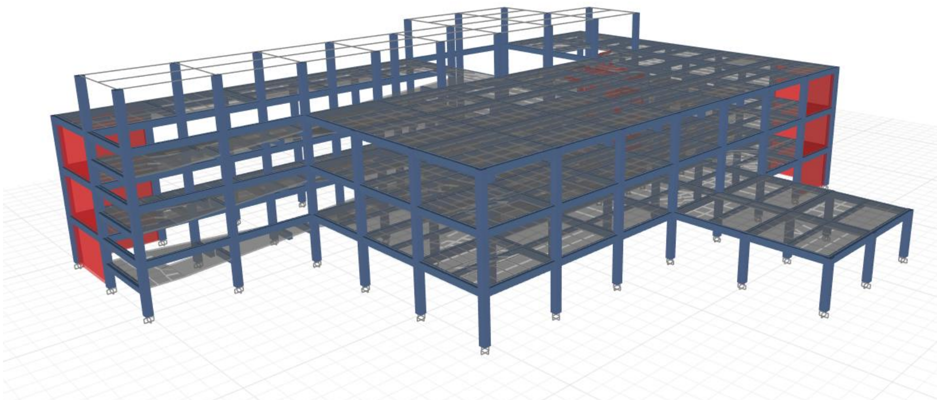


Figure 7-2: 3D Frame Model of Block B

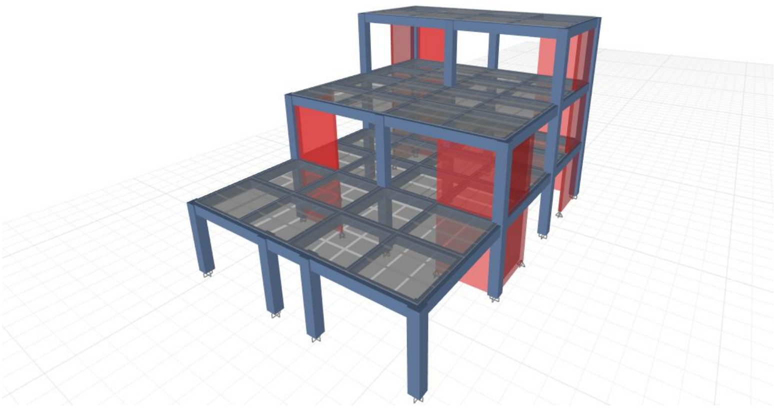


Figure 7-3: 3D Frame Model of Block C

7.2 Load Combination

Load combinations for limit state method for parallel system are based on NBC 105:2020.

1.2DL + 1.5LL

DL + λLL + E

Where, $\lambda = 0.6$ for storage facilities and 0.3 for other usage.

Following are the load combinations:

Table 7-1: Load Combinations

S.N.	Name of Combination in ETABS	Type
1	1.2DL + 1.5LL	1.2DL + 1.5LL
2	DL + 0.3LL + 0.6LL Storage + EQx	DL + 0.3LL + 0.6LL Storage + EQx
3	DL + 0.3LL + 0.6LL Storage + EQy	DL + 0.3LL + 0.6LL Storage + EQy
4	DL + 0.3LL + 0.6LL Storage + RSx	DL + 0.3LL + 0.6LL Storage + RSx
5	DL + 0.3LL + 0.6LL Storage + RSy	DL + 0.3LL + 0.6LL Storage + RSy
6	DL + 0.3LL + 0.6LL Storage - EQx	DL + 0.3LL + 0.6LL Storage - EQx
7	DL + 0.3LL + 0.6LL Storage - EQy	DL + 0.3LL + 0.6LL Storage - EQy
8	DL + LL	DL + LL
9	Envelope	Envelope

7.3 Load Cases

The following load cases are used for the loading during analysis.

Table 7-2: Load Cases

Load Case Name	**Load Case Type**	**Load Type**
modal	Modal-Eigen	-
Dead	Linear Static	Dead
Live	Linear Static	Live
Main Wall load	Linear Static	Dead
Partition Wall load	Linear Static	Dead
Floor Finish	Linear Static	Dead
Water Tank	Linear Static	Dead
Stairs Dead	Linear Static	Dead
Stairs Live	Linear Static	Live
EQx ULS	Linear Static	Seismic
EQx SLS	Linear Static	Seismic
EQy ULS	Linear Static	Seismic
EQy SLS	Linear Static	Seismic
Wards, toilet	Linear Static	Live
office, opd, staff room	Linear Static	Live
kitchen, x-ray, operation, laundries	Linear Static	Live
pasage	Linear Static	Live

Load Case Name	Load Case Type	Load Type
truss	Linear Static	Live
RSx	Response Spectrum	Seismic
RSy	Response Spectrum	Seismic

In ETABS, the seismic loads are applied to building with auto lateral load. Calculated lateral forces on each storey are applied on corresponding diaphragms.

7.4 Applicability of Analysis Methods

The Equivalent Static Method may be used for all serviceability limit state (SLS) calculations regardless of the building characteristics. For ultimate limit state (ULS), the Equivalent Static Method may be used when at least one of the following criteria is satisfied:

i. The height of the structure is less than or equal to 15
ii. The natural time period of the structure is less than 0.5 secs.
iii. The structure is not categorized as irregular as per 5.5 and height is less than 40 m

For this hospital project, Block A and Block B satisfied criteria (i) and (iii), the block was height of 14.632 m but their natural time period was 0.701 sec which exceeds the criteria. Block C only satisfied criteria (i) as its height was 10.974 but its natural time period was 0.565 sec which exceeds the value mentioned in the criteria. For this reason, The Modal Response Spectrum Method (MRSM) was also used for analysis in addition to The Equivalent Static Method.

7.5 Analysis of Structure

Analysis of structure was carried out using ETABS. Analysis was done to check against irregularities, to calculate forces and moment acting on structural members for their design purpose and to check size and reinforcement of beams and columns. Several checks done with the Modal Response Spectrum Method of analysis are as below:

7.5.1 Modal Mass Participation Factor

Modal Mass Participation Factor is amount by which a specific vibration mode contributes to the overall response of the structure. As per NBC 105:2020 Cl 7.3, during Modal Participation Analysis, the seismic mass participation should be at least 90% of total seismic mass in the direction under consideration.

Table 7-3: Modal Mass Participation of Block A

Case	Mode	Period (sec)	UX	UY	RZ	SumUX	SumUY	SumRZ
modal	1	0.424	0.3886	0.2919	0.0033	0.3886	0.2919	0.0033
modal	2	0.324	0.2821	0.3805	0.0247	0.6707	0.6724	0.028
modal	3	0.212	0.0323	2E-06	0.4878	0.7029	0.6724	0.5159
modal	4	0.198	0.0301	0.0273	0.0113	0.733	0.6997	0.5271
modal	5	0.196	0.0018	0.0277	0.1695	0.7348	0.7274	0.6966
modal	6	0.143	0.0007	1E-05	0.0101	0.7355	0.7274	0.7068
modal	7	0.105	0.1618	0.078	0.0163	0.8973	0.8054	0.723
modal	8	0.091	0.0623	0.1647	0.0311	0.9596	0.9701	0.7542
modal	9	0.07	0.0095	0.0035	0.2198	0.9691	0.9736	0.974
modal	10	0.052	0.0087	0.0233	0.0028	0.9778	0.9969	0.9768
modal	11	0.046	0.0193	0.0023	0.0141	0.9971	0.9991	0.9909
modal	12	0.041	2E-05	2E-06	1.4E-6	0.9971	0.9991	0.9909

Table 7-4: Modal Mass Participation of Block B

Case	Mode	Period (sec)	UX	UY	RZ	SumUX	SumUY	SumRZ
modal	1	0.236	0.2807	0.4855	0.0004	0.2807	0.4855	0.0004
modal	2	0.194	0.4896	0.3102	0.0009	0.7703	0.7957	0.0013
modal	3	0.134	0.001	0.0001	0.7655	0.7712	0.7958	0.7668
modal	4	0.112	0.0199	1.5E-06	0.0004	0.7911	0.7958	0.7672
modal	5	0.109	4.1E-05	0.0175	0.0061	0.7911	0.8133	0.7733
modal	6	0.106	0.0003	0.0002	0.0607	0.7914	0.8134	0.834
modal	7	0.07	0.0248	0.1524	0.0013	0.8162	0.9658	0.8352
modal	8	0.062	0.1596	0.0118	0.0004	0.9757	0.9776	0.8356
modal	9	0.046	0.0012	4.6E-05	0.1424	0.9769	0.9776	0.978

Case	Mode	Period (sec)	UX	UY	RZ	SumUX	SumUY	SumRZ
modal	10	0.041	0.0006	0.0172	0.0053	0.9775	0.9948	0.9833
modal	11	0.037	0.0174	3.1E-05	0.0002	0.9949	0.9949	0.9835
modal	12	0.034	1.1E-05	3.6E-05	0.0001	0.9949	0.9949	0.9836

Table 7-5: Modal Mass Participation of Block C

Case	Mode	Period	UX	UY	RZ	SumUX	SumUY	SumRZ
modal	1	0.185	0.0004	0.6835	0.0001	0.0004	0.6835	0.0001
modal	2	0.182	0.6272	0.0004	0.0729	0.6275	0.6839	0.073
modal	3	0.109	0.0291	0.0005	0.6163	0.6566	0.6844	0.6894
modal	4	0.072	0.2268	0.001	0.0623	0.8835	0.6854	0.7516
modal	5	0.058	0.0007	0.2438	5.60E-06	0.8842	0.9292	0.7517
modal	6	0.037	0.0051	0.0001	0.2237	0.8893	0.9293	0.9754
modal	7	0.032	0.1038	0.0002	0.0001	0.9931	0.9294	0.9755
modal	8	0.03	0.0001	0.0705	0.0001	0.9932	0.9999	0.9756
modal	9	0.02	0.0067	0	0.0242	0.9999	0.9999	0.9998
modal	10	0.009	3.39E-05	0	2.99E-05	0.9999	0.9999	0.9999
modal	11	0.008	0.0001	0	0.0001	1	0.9999	0.9999
modal	12	0.008	2.04E-06	0	1.67E-06	1	0.9999	0.9999

7.5.2 Eccentricity

Eccentricity is said to occur when there is offset between the centre of mass and the centre of stiffness of the structure.

Centre of mass: The point in a body or system of bodies at which the whole mass may be considered as concentrated is known as centre of mass. It indicates the point through which earthquake force acts in the building.

Centre of stiffness: The point through which a horizontal force applied result in the translation of floor without any rotation. It indicates the point through which building tends to resist any forces.

As per NBC 105:2020 clause 5.7, the applied torsion at each level shall use either the forces calculated by the Equivalent Static Method or the combined story inertial forces found in a Modal Response Spectrum Method. The accidental eccentricity is taken as ±0.1b.

Table 7-6: Eccentricity Calculation of Block A

Story	Mass X	Mass Y	XCM	YCM	Cum Mass X	Cum Mass Y	XCCM
	kg	kg	m	m	kg	kg	m
Story1	440206.22	440206.22	21.221	33.366	440206.22	440206.22	21.2209
Story2	380667.77	380667.77	23.164	30.124	380667.77	380667.77	23.1639
Story3	164755.2	164755.2	22.085	29.168	164755.2	164755.2	22.0848
Story4	16939.88	16939.88	21.336	27.432	16939.88	16939.88	21.336

Story	YCCM	XCR	YCR	ex	ey	Check<10%	
	m	m	m			x=ex/Ly	y=ey/Lx
Story1	33.3656	21.2098	29.644	0.0111	3.7212	0.02%	12.21%
Story2	30.1236	21.4408	29.759	1.7231	0.3648	3.53%	1.20%
Story3	29.1684	21.5166	29.677	0.5682	-0.5083	1.17%	1.67%
Story4	27.432	21.3548	27.516	-0.0188	-0.0835	0.04%	0.27%
where,	Lx = 30.48 m	Ly = 48.768 m			Average	1.19%	3.84%
						<10% OK	<10% OK

Table 7-7: Eccentricity Calculation of Block B

Story	Mass X	Mass Y	XCM	YCM	Cum Mass X	Cum Mass Y	XCCM
	kg	kg	m	m	kg	kg	m
Story1	1092541	1092541	47.3276	31.145	1092541.03	1092541	47.3276
Story2	973911.84	973911.84	45.5545	30.032	973911.84	973911.84	45.5545
Story3	425319.05	425319.05	44.9189	30.067	425319.05	425319.05	44.9189
Story4	23306.38	23306.38	38.4048	25.603	23306.38	23306.38	38.4048

Story	YCCM	XCR	YCR	ex	ey	Check<10%	
	m	m	m			x=ex/Ly	y=ey/Lx
Story1	31.1449	45.5454	32.6612	1.7822	-1.5163	3.25%	3.55%
Story2	30.0321	44.8205	31.8107	0.734	-1.7786	1.34%	4.17%
Story3	30.0673	44.2253	31.1261	0.6936	-1.0588	1.26%	2.48%
Story4	25.6032	38.6092	25.5651	-0.2044	0.0381	0.37%	0.09%
where,	Lx = 42.672 m	Ly = 54.864 m			Average	1.56%	2.575%
						<10% OK	<10% OK

Table 7-8: Eccentricity Calculation of Block C

Story	Mass X	Mass Y	XCM	YCM	Cum Mass X	Cum Mass Y	XCCM
	kg	kg	m	m	kg	kg	m
Story1	253688.26	253688.26	36.509	64.653	531257.85	531257.85	36.4746
Story2	188261.07	188261.07	36.428	60.839	277569.59	277569.59	36.4435
Story3	89308.52	89308.52	36.477	58.305	89308.52	89308.52	36.4767

Story	YCCM	XCR	YCR	ex	Ey	Check<10%	
	m	m	m			x=ex/Ly	y=ey/Lx
Story1	62.2341	36.5684	61.429	-0.094	0.805	0.51%	6.60%
Story2	60.0234	36.6505	61.367	-0.207	-1.344	1.13%	11.02%
Story3	58.3046	36.8942	57.665	-0.418	0.640	2.28%	5.25%
where, Lx = 12.192 m Ly = 18.288 m					Avg.	1.31%	7.62%
						<10% OK	<10% OK

7.5.3 Weak Story and Soft Story

These vertical irregularities are discussed on clause 7.3 of NBC 105: 2020.

i. Weak Story:

A story is considered as weak story if the strength of the lateral force resisting system in that story is less than 80% of the strength of the story above.

ii. Soft Story:

A soft story is the one whose stiffness of the lateral-force-resisting system is less than 70% of the lateral-force-resisting system stiffness in an adjacent story above or below, or less than 80% of the average lateral-force-resisting system stiffness of the three stories above or below. Check for soft story automatically checks for weak story.

Table 7-9: Soft Storey in X-Direction of Block A

Story	Output Case	Stiff X kN/m	K_i/K_{i+1}	Check Ki/Ki+1 >0.7	K_{avg}= Avg(K_{i+1},K_{i+2},K_{i+3})	Check K_i/K_{avg} >0.8
Story4	RSx	22914.92	-	-	-	-
Story3	RSx	276680.5	12.074	OK	-	-
Story2	RSx	526894.2	1.9043	OK	275496.5447	OK
Story1	RSx	1187663	2.2541	OK	663745.811	OK

Table 7-10: Soft Storey in Y-Direction of Block A

Story	Output Case	Stiff Y kN/m	K_i/K_{i+1}	Check Ki/Ki+1 >0.7	K_{avg}= Avg(K_{i+1},K_{i+2},K_{i+3})	Check K_i/K_{avg} >0.8
Story4	RSy	25047.96	-	-	-	-
Story3	RSy	315845.1	12.61	OK	-	-
Story2	RSy	627179.3	1.9857	OK	322690.783	OK
Story1	RSy	1422312	2.2678	OK	788445.4193	OK

Table 7-11: Soft Storey in X-Direction of Block B

Story	Output Case	Stiff X kN/m	K_i/K_{i+1}	Check Ki/Ki+1 >0.7	K_{avg}= Avg(K_{i+1},K_{i+2},K_{i+3})	Check K_i/K_{avg} >0.8
Story4	RSx	49342.49	-	-	-	-
Story3	RSx	2346582	47.557	OK	-	-
Story2	RSx	4742405	2.0209	OK	2379442.915	OK
Story1	RSx	7729309	1.6298	OK	4939431.591	OK

Table 7-12: Soft Storey in Y-Direction of Block B

Story	Output Case	Stiff Y kN/m	K_i/K_{i+1}	Check Ki/Ki+1 >0.7	K_{avg}= Avg(K_{i+1},K_{i+2},K_{i+3})	Check K_i/K_{avg} >0.8
Story4	RSy	58415.54	-	-	-	-
Story3	RSy	2132543	36.50644	OK	-	-
Story2	RSy	4206099	1.972339	OK	2132352.733	OK
Story1	RSy	6549651	1.557179	OK	4296098.039	OK

Table 7-13: Soft Storey in X-Direction of Block C

Story	Output Case	Stiff X kN/m	K_i/K_{i+1}	Check Ki/Ki+1 >0.7	K_{avg}= Avg(K_{i+1},K_{i+2},K_{i+3})	Check K_i/K_{avg} >0.8
Story3	RSx	315974.212	-	-	-	-
Story2	RSx	971284.863	3.0739	OK	-	-
Story1	RSx	2406719.64	2.4779	OK	1231326.2	OK

Table 7-14: Soft Storey in Y-Direction of Block C

Story	Output Case	Stiff Y kN/m	K_i/K_{i+1}	Check Ki/Ki+1 >0.7	K_{avg}= Avg(K_{i+1},K_{i+2},K_{i+3})	Check K_i/K_{avg} >0.8
Story3	RSy	317276.031	-	-	-	-
Story2	RSy	890804.149	2.8077	OK	-	-
Story1	RSy	2199251.52	2.4688	OK	1135777.2	OK

7.5.4 Torsion Irregularity

As per NBC 105: 2020 clause 5.5.2.1, Torsion irregularity is considered to exist where the maximum horizontal displacement of any floor in the direction of the lateral force (applied at the center of mass) at one end of the story is more than 1.5 times its minimum horizontal displacement at the far end of the same story in that direction.

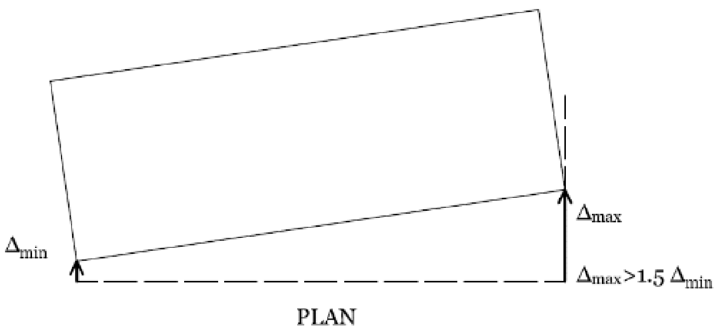


Figure 7-4: Torsion Irregularity

Table 7-15: Torsion Irregularity in Block A

Axis	Displacement	Story1	Story2	Story3	Story4
RSx	Minimum displacement (Δ_{min})	2.642	7.884	13.688	22.691
	Maximum displacement (Δ_{max})	3.124	8.42	14.595	23.201
	$\Delta_{max}<1.5\Delta_{min}$	OK	OK	OK	OK
RSy	Minimum displacement (Δ_{min})	2.312	6.826	11.781	20.219
	Maximum displacement (Δ_{max})	2.476	7.302	12.573	20.707
	$\Delta_{max}<1.5\Delta_{min}$	OK	OK	OK	OK

Table 7-16: Torsion Irregularity in Block B

Axis	Displacement	Story1	Story2	Story3	Story4
RSx	Minimum displacement (Δ_{min})	0.878	2.217	3.539	0.878
	Maximum displacement (Δ_{max})	1.08	2.569	3.981	1.08
	$\Delta_{max}<1.5\Delta_{min}$	OK	OK	OK	OK
RSy	Minimum displacement (Δ_{min})	1.077	2.597	4.017	1.077
	Maximum displacement (Δ_{max})	1.237	2.81	4.345	1.237
	$\Delta_{max}<1.5\Delta_{min}$	OK	OK	OK	OK

Table 7-17: Torsion Irregularity in Block C

Axis	Displacement	Story1	Story2	Story3	Story4
RSx	Minimum displacement (Δ_{min})	0.467	1.421	3.383	0.467
	Maximum displacement (Δ_{max})	0.625	1.827	3.413	0.625
	$\Delta_{max}<1.5\Delta_{min}$	OK	OK	OK	OK
RSy	Minimum displacement (Δ_{min})	0.686	1.997	3.75	0.686
	Maximum displacement (Δ_{max})	0.71	2.063	4.047	0.71
	$\Delta_{max}<1.5\Delta_{min}$	OK	OK	OK	OK

7.5.5 Maximum Story displacement

Story displacement is the deflection of a single story relative to the base or ground level of the structure. Intuitively, we can expect higher total displacement values as we move up the structure. So, a graph showing the story displacement vs. the height of the structure looks exactly like the deflected shape. As calculated in Table 5-4 for Block A and B allowable story displacement is 91.450 mm for ULS and 87.792 mm for SLS.

7.5.5.1 Story Response - Maximum Story Displacement for Block A

Summary Description

This is story response output for a specified range of stories and a selected load cases.

Name	StoryResp3		
Display Type:	Max story displ	Story Range:	All Stories
Load Case:	RSx	Top Story:	Story4
Output Type:	Not Applicable	Bottom Story:	Base

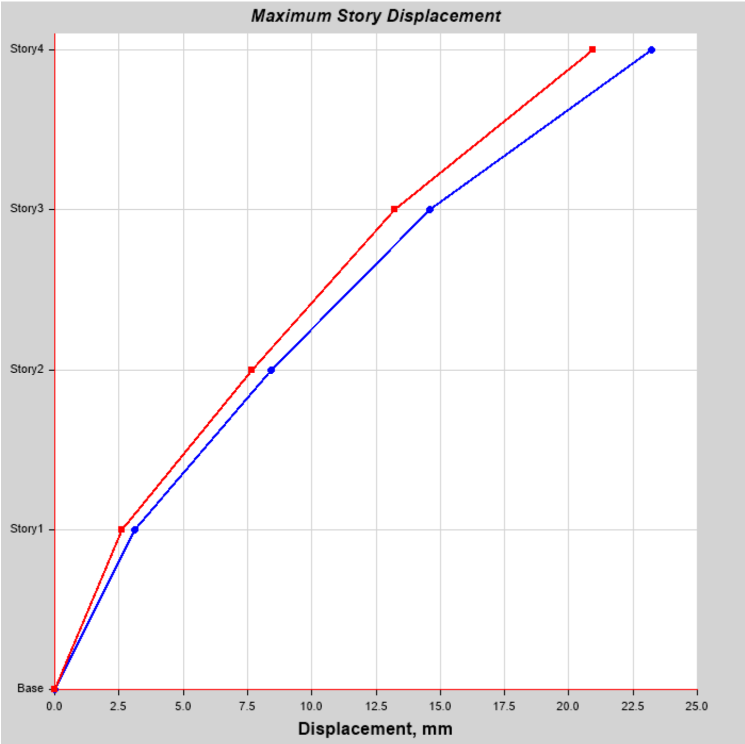


Figure 7-5: RSx Maximum Story Displacement Plot for Block A

Table 7-18: RSx Maximum Story Displacement Plot Coordinates for Block A

Story	Elevation (m)	Location	X-Dir (mm)	Y-Dir (mm)
Story4	14.632	Top	23.201	20.946
Story3	10.974	Top	14.595	13.244
Story2	7.316	Top	8.42	7.673
Story1	3.658	Top	3.124	2.617
Base	0	Top	0	0

Input Data

Name	StoryResp3		
Display Type	Max story displ	Story Range	All Stories
Load Case	RSy	Top Story	Story4
Output Type	Not Applicable	Bottom Story	Base

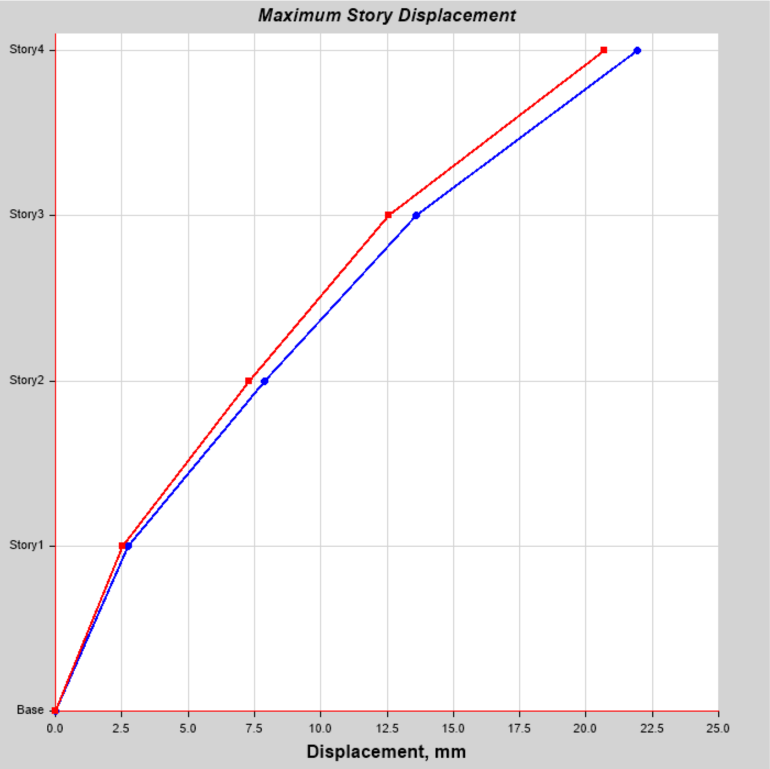


Figure 7-6: RSy Maximum Story Displacement Plot for Block A

Table 7-19: RSy Maximum Story Displacement Plot Coordinates for Block A

Story	Elevation (m)	Location	X-Dir (mm)	Y-Dir (mm)
Story4	14.632	Top	21.933	20.707
Story3	10.974	Top	13.615	12.573
Story2	7.316	Top	7.882	7.302
Story1	3.658	Top	2.757	2.515
Base	0	Top	0	0

7.5.5.2 Story Response - Maximum Story Displacement for Block B

Input Data

Name: StoryResp1

Display Type: Max story displ — Story Range; All Stories

Load Case: RSx — Top Story: Story4

Output Type: Not Applicable — Bottom Story: Base

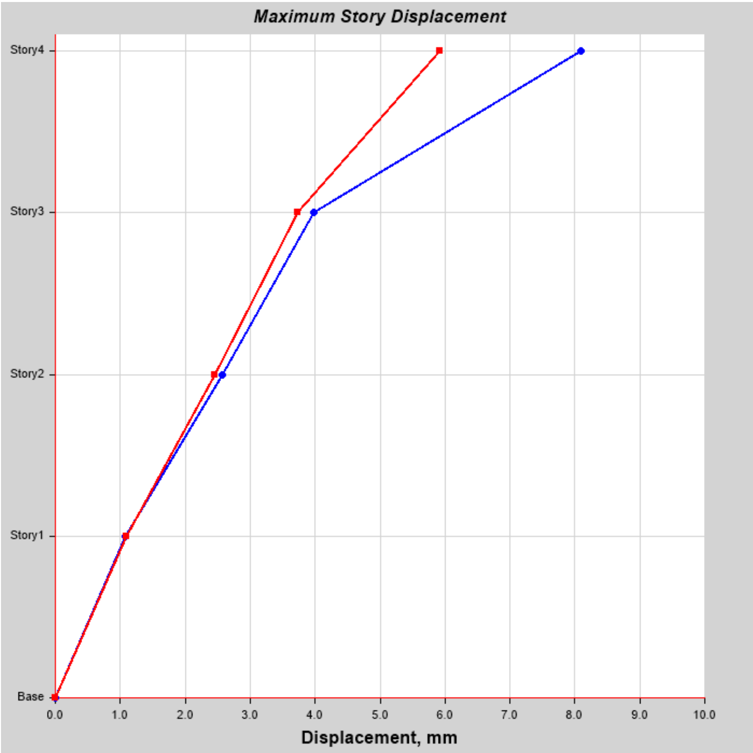


Figure 7-7: RSx Maximum Story Displacement Plot for Block B

Table 7-20: RSx Maximum Story Displacement Plot Coordinates for Block B

Story	Elevation (m)	Location	X-Dir (mm)	Y-Dir (mm)
Story4	14.632	Top	8.085	5.915
Story3	10.974	Top	3.981	3.737
Story2	7.316	Top	2.569	2.45
Story1	3.658	Top	1.08	1.088
Base	0	Top	0	0

Input Data

Name:	StoryResp1		
Display Type:	Max story displ	Story Range:	All Stories
Load Case:	RSy	Top Story:	Story4
Output Type:	Not Applicable	Bottom Story:	Base

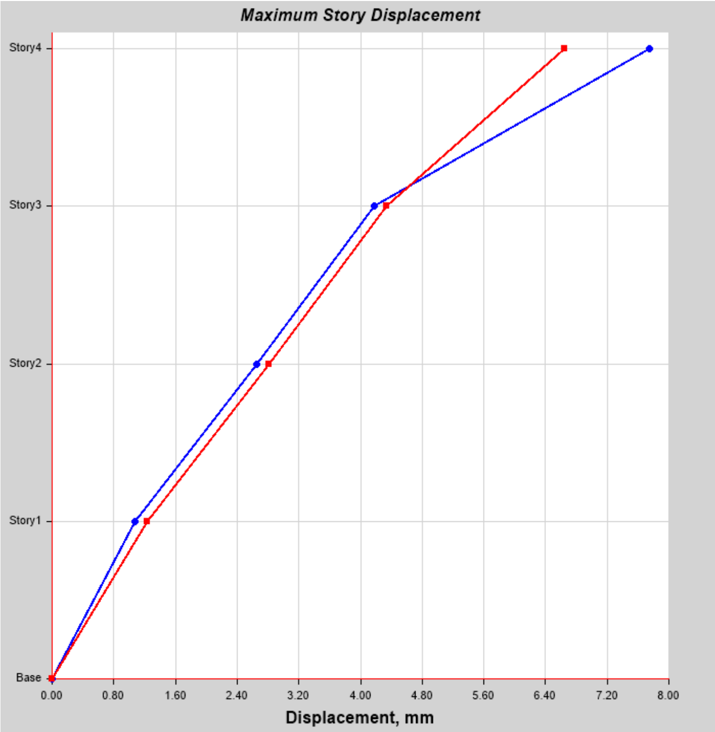


Figure 7-8: RSy Maximum Story Displacement Plot for Block B

Table 7-21: RSy Maximum Story Displacement Plot Coordinates for Block B

Story	Elevation (m)	Location	X-Dir (mm)	Y-Dir (mm)
Story4	14.632	Top	7.754	6.651
Story3	10.974	Top	4.181	4.345
Story2	7.316	Top	2.652	2.81
Story1	3.658	Top	1.075	1.237
Base	0	Top	0	0

7.5.5.3 Story Response - Maximum Story Displacement for Block C

As calculated in Table 5-4 for Block A and B allowable story displacement is 68.588 mm for ULS and 65.844 mm for SLS.

Input Data

Name:	StoryResp1		
Display Type:	Max story displ	Story Range:	All Stories
Load Case:	RSx	Top Story:	Story3
Output Type:	Not Applicable	Bottom Story:	Base

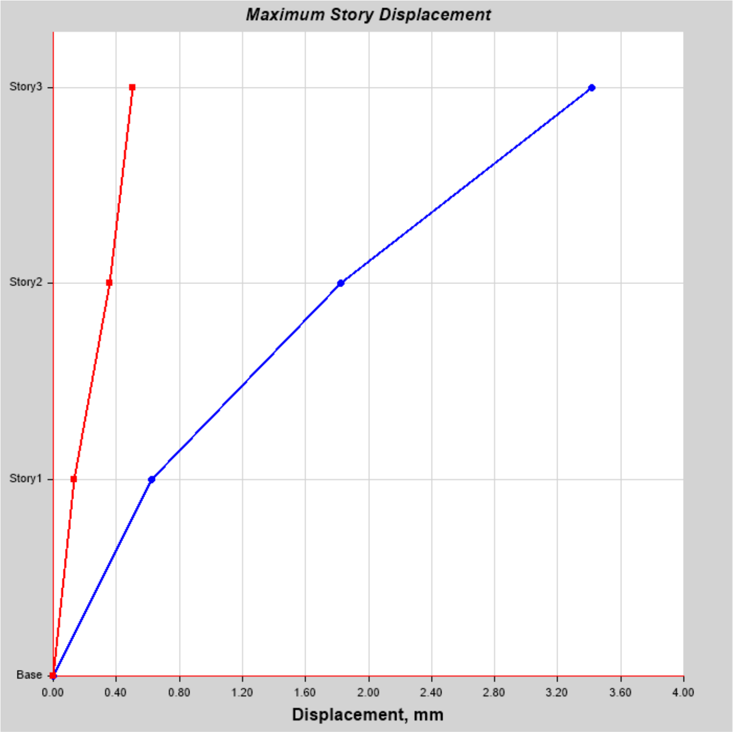


Figure 7-9: RSx Maximum Story Displacement Plot for Block C

Table 7-22: RSx Maximum Story Displacement Plot Coordinates for Block C

Story	Elevation (m)	Location	X-Dir (mm)	Y-Dir (mm)
Story3	10.974	Top	3.413	0.507
Story2	7.316	Top	1.827	0.361
Story1	3.658	Top	0.625	0.13
Base	0	Top	0	0

Input Data

Name:	StoryResp1		
Display Type:	Max story displ	Story Range:	All Stories
Load Case:	RSy	Top Story:	Story3
Output Type:	Not Applicable	Bottom Story:	Base

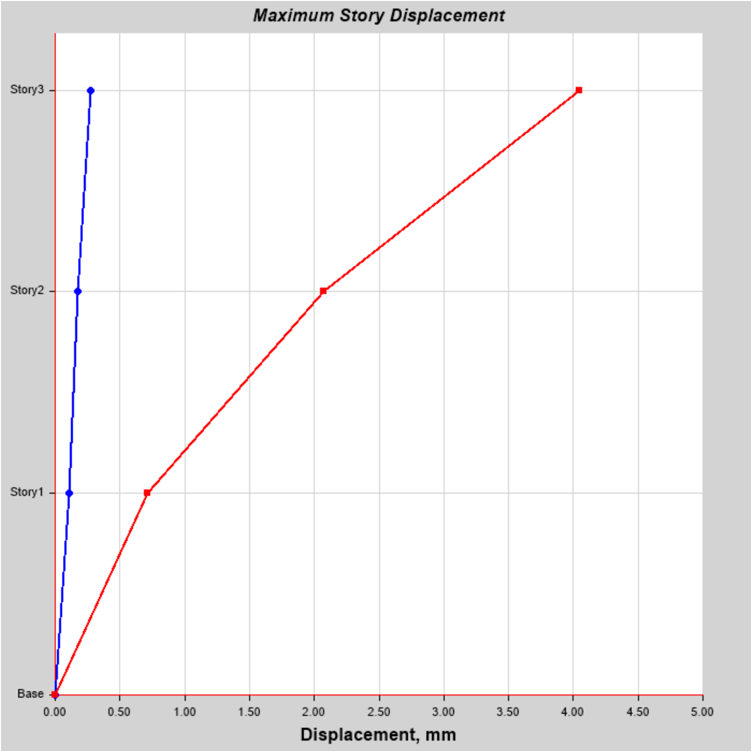


Figure 7-10: RSy Maximum Story Displacement Plot for Block C

Table 7-23: RSy Maximum Story Displacement Plot Coordinates for Block C

Story	Elevation (m)	Location	X-Dir (mm)	Y-Dir (mm)
Story3	10.974	Top	3.413	0.507
Story2	7.316	Top	1.827	0.361
Story1	3.658	Top	0.625	0.13
Base	0	Top	0	0

7.5.6 Maximum Story Drifts

Storey drift is the lateral displacement of a floor relative to the floor below, and the storey drift ratio is the storey drift divided by the storey height. As in Table 5-4 and Table 5-5 for Block A and B and C allowable story drift is 0.00625 mm for ULS and 0.006 mm for SLS.

7.5.6.1 Story Response - Maximum Story Drift for Block A

Input

Name	StoryResp3		
Display Type:	Max story drifts	Story Range:	All Stories
Load Case:	RSx	Top Story:	Story4
Output Type:	Not Applicable	Bottom Story:	Base

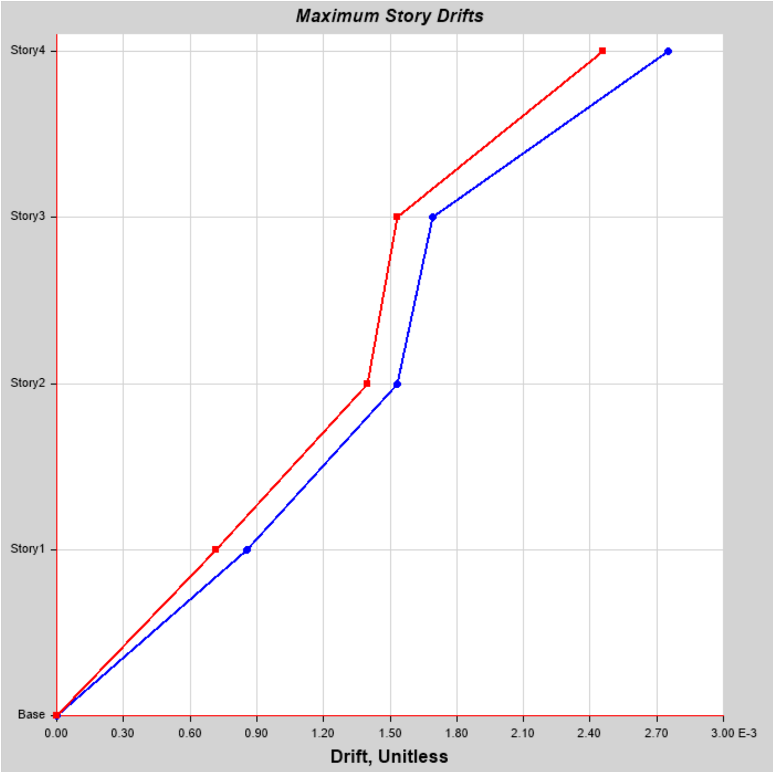


Figure 7-11: RSx Maximum Story Drift Plot for Block A

Table 7-24: RSx Maximum Story Drift Plot Coordinates for Block A

Story	Elevation (m)	Location	X-Dir (mm)	Y-Dir (mm)
Story4	14.632	Top	0.002749	0.002457
Story3	10.974	Top	0.001693	0.001531
Story2	7.316	Top	0.001531	0.001397
Story1	3.658	Top	0.000854	0.000715
Base	0	Top	0	0

Input Data

Name	StoryResp3		
Display Type	Max story drifts	Story Range	All Stories
Load Case	RSy	Top Story	Story4
Output Type	Not Applicable	Bottom Story	Base

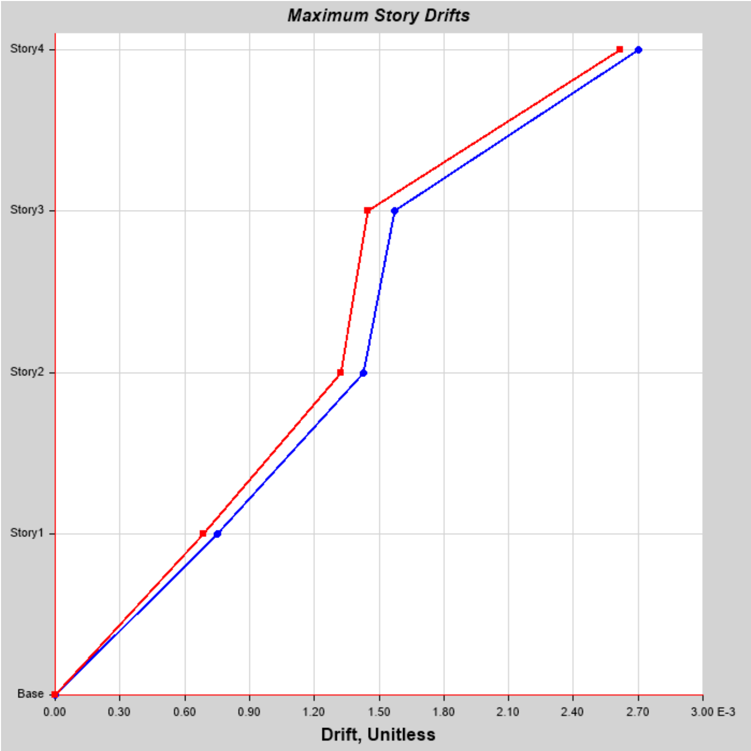


Figure 7-12: RSy Maximum Story Drift Plot for Block A

Table 7-25: RSy Maximum Story Drift Plot Coordinates for Block A

Story	Elevation (m)	Location	X-Dir (mm)	Y-Dir (mm)
Story4	14.632	Top	0.002704	0.002618
Story3	10.974	Top	0.00157	0.001448
Story2	7.316	Top	0.001427	0.001324
Story1	3.658	Top	0.000754	0.000688
Base	0	Top	0	0

7.5.6.2 Story Response - Maximum Story Drift for Block B

Input Data

Name: StoryResp1

Display Type: Max story drifts | Story Range; All Stories

Load Case: RSx | Top Story: Story4

Output Type: Not Applicable | Bottom Story: Base

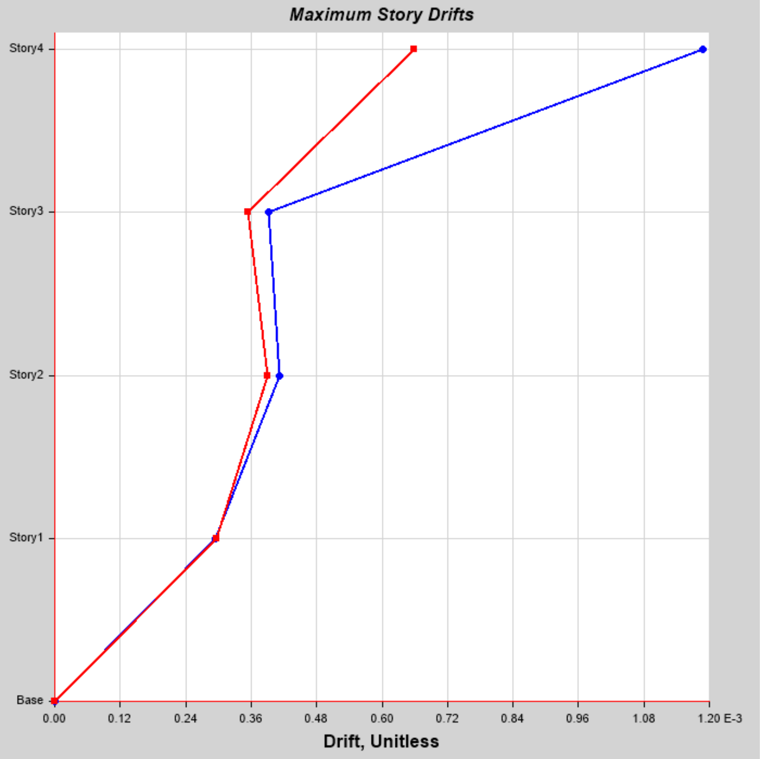


Figure 7-13: RSx Maximum Story Drift Plot for Block B

Table 7-26: RSx Maximum Story Drift Plot Coordinates for Block B

Story	Elevation (m)	Location	X-Dir (mm)	Y-Dir (mm)
Story4	14.632	Top	0.001189	0.000659
Story3	10.974	Top	0.000392	0.000355
Story2	7.316	Top	0.000412	0.00039
Story1	3.658	Top	0.000295	0.000297
Base	0	Top	0	0

Input Data

Name: StoryResp1

Display Type: Max story drifts | Story Range: All Stories

Load Case: RSy | Top Story: Story4

Output Type: Not Applicable | Bottom Story: Base

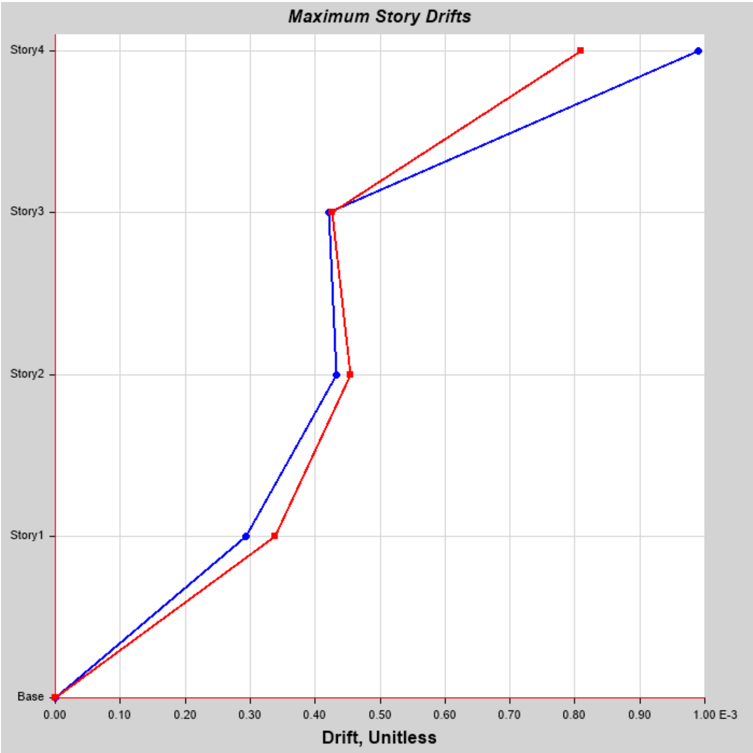


Figure 7-14: RSy Maximum Story Drift Plot for Block B

Table 7-27: RSy Maximum Story Drift Plot Coordinates for Block B

Story	Elevation (m)	Location	X-Dir (mm)	Y-Dir (mm)
Story4	14.632	Top	0.00099	0.00081
Story3	10.974	Top	0.000421	0.000426
Story2	7.316	Top	0.000433	0.000454
Story1	3.658	Top	0.000294	0.000338
Base	0	Top	0	0

7.5.6.3 Story Response - Maximum Story Drift for Block C

Input Data

Name:	StoryResp1		
Display Type:	Max story drifts	Story Range:	All Stories
Load Case:	RSx	Top Story:	Story3
Output Type:	Not Applicable	Bottom Story:	Base

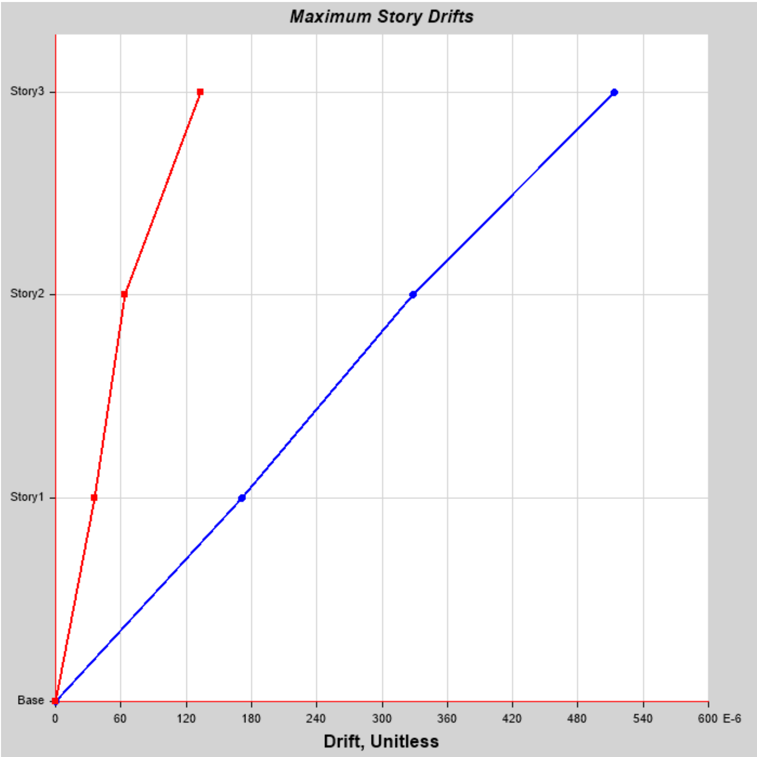


Figure 7-15: RSx Maximum Story Drift Plot for Block C

Table 7-28: RSx Maximum Story Drift Plot Coordinates for Block C

Story	Elevation (m)	Location	X-Dir (mm)	Y-Dir (mm)
Story3	10.974	Top	0.000514	0.000133
Story2	7.316	Top	0.000329	0.000064
Story1	3.658	Top	0.000171	0.000036
Base	0	Top	0	0

Input Data

Name:	StoryResp1		
Display Type:	Max story drifts	Story Range:	All Stories
Load Case:	RSy	Top Story:	Story3
Output Type:	Not Applicable	Bottom Story:	Base

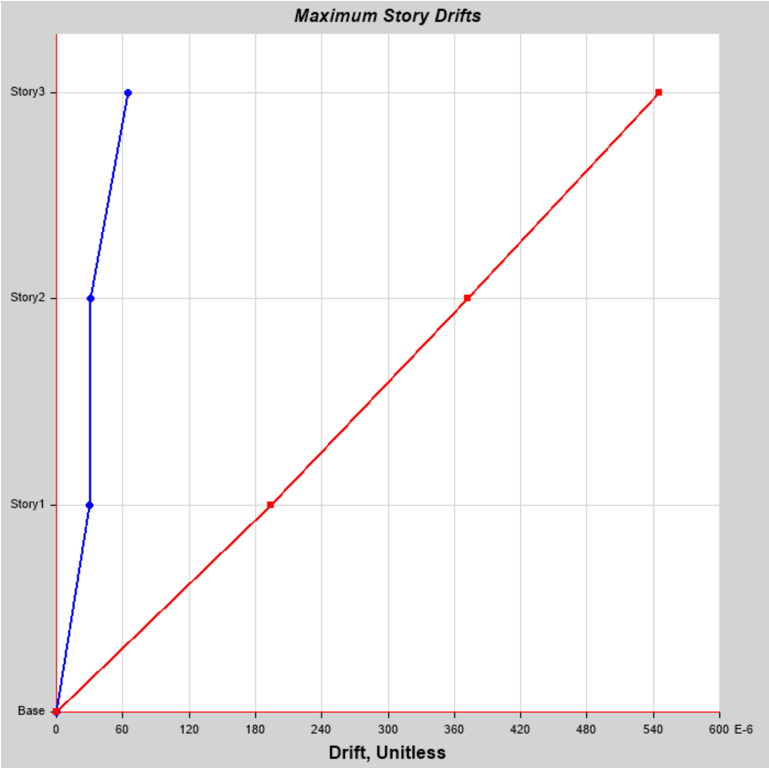


Figure 7-16: RSy Maximum Story Drift Plot for Block C

Table 7-29: RSy Maximum Story Drift Plot Coordinates for Block C

Story	Elevation (m)	Location	X-Dir (mm)	Y-Dir (mm)
Story3	10.974	Top	0.000064	0.000545
Story2	7.316	Top	0.000031	0.000372
Story1	3.658	Top	0.000029	0.000194
Base	0	Top	0	0

7.6 Seismic Gap Calculation

According to NBC 105:2020 clause 5.6.2 parts of buildings or buildings on the same site which are not designed to act as an integral unit shall be separated from each other by a distance of not less than the sum of the design horizontal deflections. As per clause 5.6.1 of the same code the design horizontal deflections shall be determined by multiplying the horizontal deflection found from Equivalent Static Method or Modal Response Spectrum Method by the Ductility factor ($R\mu$). The sum of the horizontal deflection will result in unfunctional gap in-between blocks of high buildings. So, generally square root of sum of squares of the horizontal deflection of two blocks is used to calculate building separations.

Table 7-30: Calculation of Seismic Gap

S.N	Block	Maximum displacement in X-direction	Maximum displacement in Y-direction
1	A	23.201	20.707
2	B	3.981	4.345
3	C	3.413	4.047

In X-direction

S.N	Block Separation	Horizontal deflection×Ductility factor(mm) X-direction	Seismic Gap required $(\sqrt{}\Delta_{12}+\Delta_{22})\times R_u$
1	AB	108.728 mm	94.160 mm
2	AC	106.456 mm	93.803 mm
3	BC	29.576 mm	20.975 mm

In Y-direction

S.N	Block Separation	Horizontal deflection×Ductility factor(mm) Y-direction	Seismic Gap required $(\sqrt{}\Delta_{12}+\Delta_{22})\times R_u$
1	AB	100.208 mm	84.632 mm
2	AC	99.016 mm	84.395 mm
3	BC	33.568 mm	23.751 mm

Seismic gap of mm was provided for all block separation.

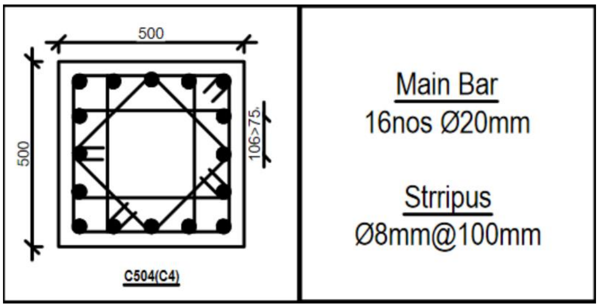
7.7 Column Beam Moment Capacity Ratio

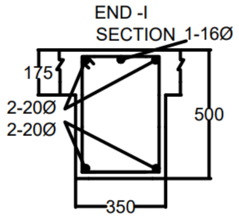
According to NBC 105: 2020 clause 4.4.4, at every beam column junction in a frame, the summation of the moment capacities of the column end sections shall be greater than 1.2 times the summation of the beam end moment capacities.

As we all know that if a beam of any high-rise building fails it will only affect the that particular story but, if a column of structure fails then it will lead to a failure of the whole building. So, this demands a strong column weak beam design.

Sample calculation of Column Beam Moment Capacity Ratio for column located in grid H-3 is shown below:

Table 7-31: Column Beam Moment Capacity Ratio

SN	Calculation	Reference
	Grid = H-3 Column Type = C504 Beam Type = PB2 f_{ck} = 25 N/mm^2 f_y = 500 N/mm^2	
1.	**Dimensions** Beam Width (b) = 350 mm Beam Depth = 500 mm Column width (b_c)= 500 mm Column Depth = 500 mm Clear Cover = 25 mm Bar Size used in Beam = 20 mm	
2. i.	**Reinforcement Provided** Column reinforcement  Percentage of steel provided in column = $\{\pi\times20^2/4\times16\}/(500\times500))\times100$ = 2.01% Effective depth of column (d)= 500-25-8-(20/2) = 457 mm	

SN	Calculation	Reference
ii.	Beam reinforcement  Percentage of steel provided in beam at top = $[\{(\pi\times20^2)/4\}\times2+\{(\pi\times16^2)/4\}\times1]/ (350\times500))\times100$ = 0.47393% Percentage of steel provided in beam at bottom = $\{(\pi\times20^2)/4\}\times2/ (350\times500))\times100$ = 0.36% Diameter of stirrup = 8 mm Effective depth of beam (d)= 500-25-8-(20/2) = 457 mm Area of Steel provided in beam at top ($A_{st,Top}$) = 0.47393×350×457 /100 = 758.054 mm^2 Area of Steel provided in beam at bottom ($A_{st,Bottom}$) = 0.36×350×457 /100= 575.82 mm^2	
3.	**Moment Capacity of Column**	
i.	Sagging Moment Capacity $X_u = (0.87\times f_y\times A_{st,Bottom})/ (0.36\times f_{ck}\times b_c)$ = (0.87×500×575.82)/ (0.36×25×350) = 79.52 mm $(M_B)_{Left} = 0.87\times f_y\times A_{st,Bottom}\times(d-0.42\times X_u)$ = 0.87×500×575.82×{457-(0.42×79.52)} = 106104448.891 N-mm = 106.104 kN-m	IS 456:2000 G-1.1 (a) IS 456:2000 G-1.1 (b)
ii.	Hogging Moment Capacity M_{u1} (Moment due to balanced section) = $0.133\times f_{ck}\times b\times d^2$ = $0.133\times25\times350\times457^2$	

SN	Calculation	Reference
	= 243048023.75 N-mm = 243.048 kN-m $A_{st1} = M_{u1}/[0.87 \times f_y \times \{d-(0.42 \times X_{u,lim})\}]$ = $(243.048 \times 10^6)/[0.87 \times 500 \times \{457-(0.42 \times 0.46 \times 457)\}])$ = 1515.377 mm^2 $A_{sc} = A_{st,Top} - A_{st1}$ = 758.054-1515.377 = -757.323 mm^2 Mu_2 (Moment due to compression member) = $f_{sc} \times A_{sc} \times (d-d')$ = (0.87×500)×-757.32×{457-(25+8+20/2)} = -136386299.07 N-mm = -136.386 kN-m $(M_B)_{Right} = M_{u1} + M_{u2}$ = 106.662 kN-m Moment Capacity of beam (M_B) = $(M_B)_{Left} + (M_B)_{Right}$ = 106.104+106.662 =212.77 kN-m	IS 456:2000 G-1.1 (b) IS 456:2000 G-1.2
4.	**Moment Capacity of Column** P% = 2.01% P/f_{ck} = 0.0804 For zero axial force, P_u = 0 Eff. cover/effective depth = 0.0941 $M_u/ f_{ck}bd^2$ = 0.125 $M_u = 0.125 \times 25 \times 500 \times 500^2$ = 390.625 kN-m Adding moment capacity for upper and lower column, Moment Capacity of Column (M_C) = $2 \times M_u$ = 781.25 kN-m	Sp-16 Chart 48
5.	**Column Beam Moment Capacity Ratio** Column Beam Moment Capacity Ratio = M_C/M_B = 3.672 Which is greater than 1.2 OK	NBC 105:2020 Clause 4.4.4

7.8 Axial Force Diagram

Load Combination: Envelope

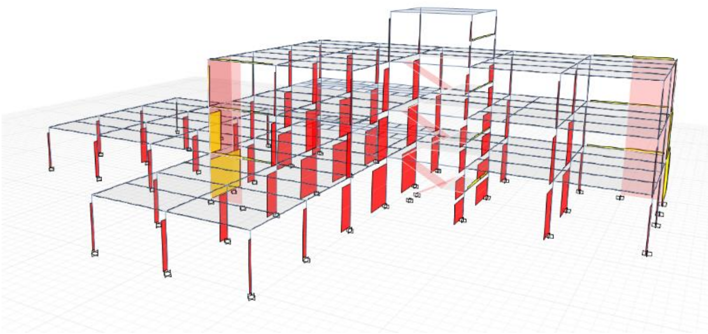


Figure 7-17: Axial Force Diagram of Block A

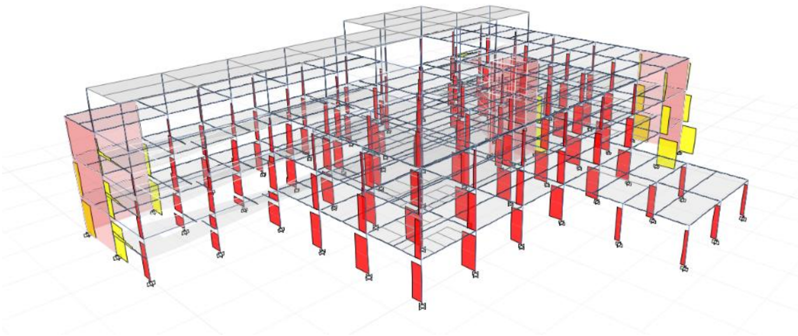


Figure 7-18: Axial Force Diagram of Block B

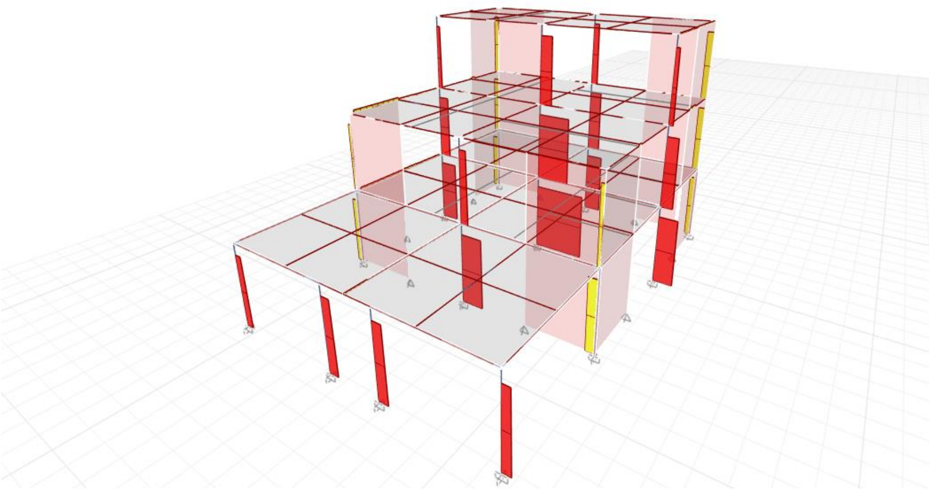


Figure 7-19: Axial Force Diagram of Block C

7.9 Shear Force Diagram

Load Combination: Envelope

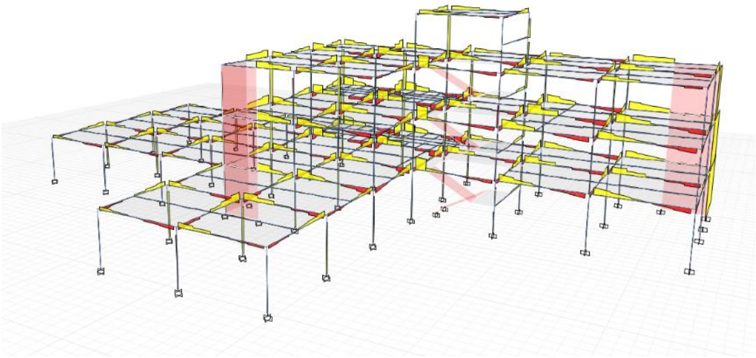


Figure 7-20: Shear Force Diagram of Block A

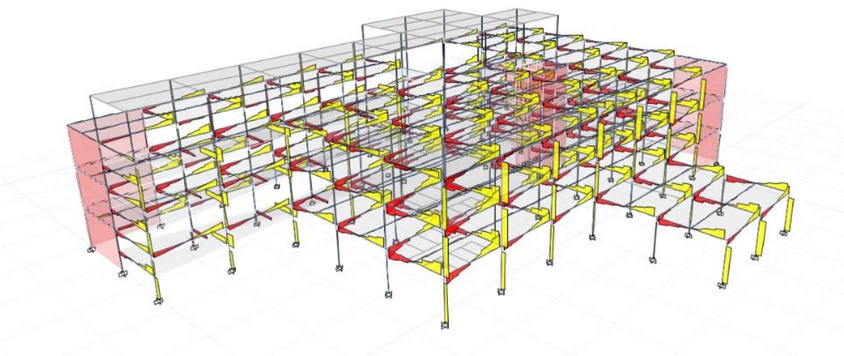


Figure 7-21: Shear Force Diagram of Block B

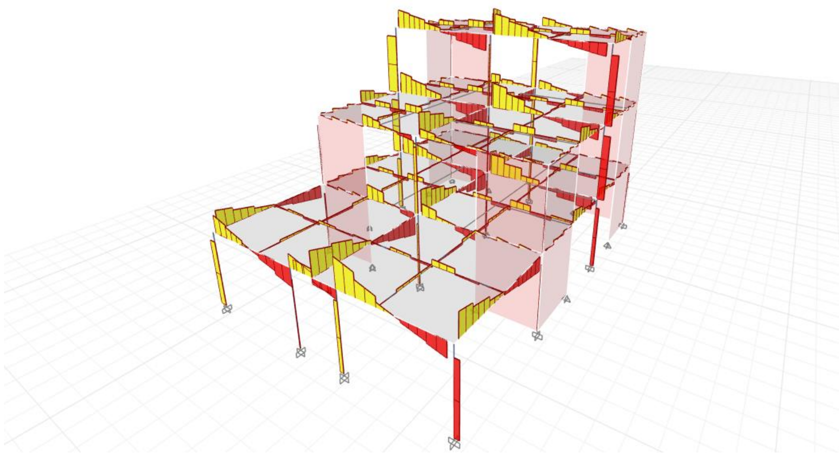


Figure 7-22: Shear Force Diagram of Block C

7.10 Bending Moment Diagram

Load Combination: Envelope

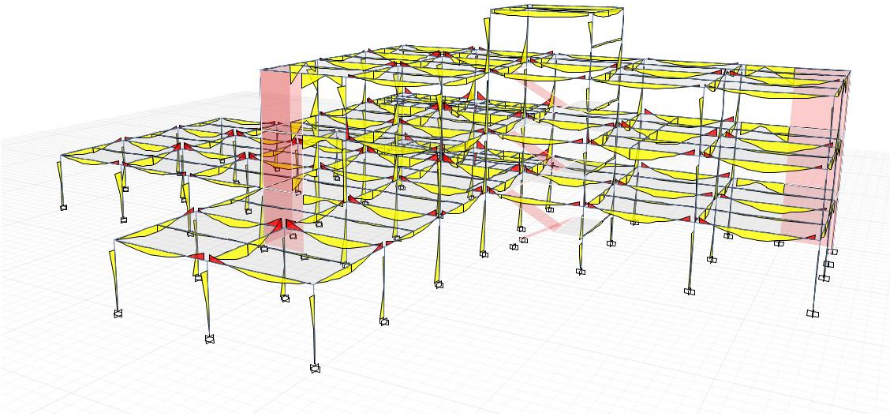


Figure 7-23: Bending Moment Diagram of Block A

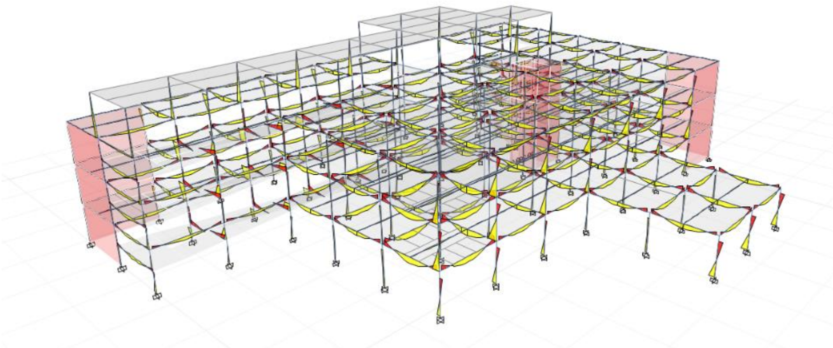


Figure 7-24: Bending Moment Diagram of Block B

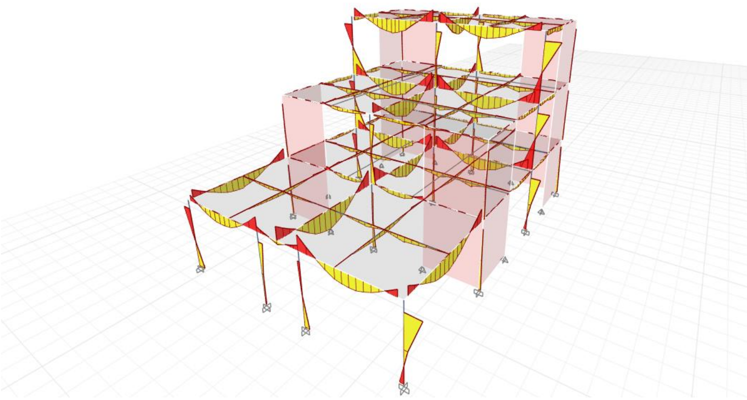


Figure 7-25: Bending Moment Diagram of Block C

8. DETAIL DESIGN

8.1 Design of Column

A compression member is defined as a structural member subjected to compressive force in a directional parallel to its longitudinal axis. The most common type of compression member is column (vertical element building). Columns are predominantly subjected to axial forces. Columns support the beams and slabs and transfer the load to the foundation.

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

Based on the nature of loading, columns may be classified as either axially loaded, uniaxially loaded or biaxially loaded columns. Axially loaded columns are under pure axial compression whereas uniaxially and biaxially loaded columns have eccentric loading in one or both directions respectively. Columns in framed structures under seismic loads are generally biaxially loaded and hence designed accordingly. Here we design for biaxially loaded column. The end moments and end shear are available from computer analysis. The design moment should include the following:

a. The additional moment if any, due to long column effect as per C1.39.7 of IS 456:2000
b. The moments due to minimum eccentricity as per C1.25.4 of IS 456:2000

Design Steps:

i. Calculate the factored load and the factored moment along both axes on the column.
ii. Calculate the eccentricity along both axes. Each eccentricity is multiplied by the design axial load to find the moment due to eccentricity in the respective axis.
iii. The greater of imposed moment and moment due to eccentricity is taken for design.
iv. Size of reinforcement, distribution of reinforcement, cover and percentage of steel needs to be assumed to calculate $\left(\frac{P}{f_{ck}}\right)$ and $\left(\frac{P_u}{f_{ck}bD}\right)$.
v. Calculate M_{ux1} and M_{uy1} using related charts of Sp-16.

vi. Calculate value of α_n from the relation:

$$\alpha_n = 0.667 + 1.661 \text{ x } \left(\frac{p_u}{p_z}\right)$$

vii. All the values are supplied to compute the interaction relation:

$$\left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1$$

If the above relation is true, the design is safe otherwise revise the percentage of steel and repeat all steps after step (iv)

viii. From the percentage of steel area of steel (A_{st}) is calculated which in turn is used to find number of longitudinal bars.

ix. The diameter of the ties shall not be lesser than the greatest of the following values:

a. 6 mm

b. 1/4th of the Diameter of the Largest Diameter Bar

The Spacing of Ties shall not exceed the least of the followings three values:

a. Least Lateral Dimension

b. 16 Times of the Diameter of the Smallest Diameter Longitudinal Bar

c. 300 mm

Sample design calculation for column of unique name "176" in Story 1 of Block B:

Table 8-1: Design of Column

Step	Calculation	Reference
1.	Governing Combo = DL + 0.3LL + 0.6LL Storage + RSx Storey = Story 1 of Block B Grid position = A8 Column Type = C502 Unique Name = 176 **Material Properties** Grade of Concrete Used (f_{ck}) = 25 N/mm^2 Grade of Steel Used (f_y) = 500 N/mm^2	
2.	**Member Properties** Length Of Column (L) = 3.658 m Depth of Beam (D)= 500 mm	

Step	Calculation	Reference
	Effective Length Factor (K_x) = 0.65 Effective Length Factor (K_y) = 0.65 Unsupported Length of Column = 3.658-500/1000 = 3.158 m Effective Length of Column (L_{ex}) = 3.158×0.65 = 2.053 m Effective Length of Column (L_{ey}) = 3.158×0.65 = 2.053 m Width of Column (D_x) = 500 mm Depth of Column (D_y) = 500 mm Clear Cover = 40 mm Confinement Bar (Ties) = 8 mm Effective Cover (d') = 40+8+20/2 = 58 mm	Table 28 Table 28
3.	**Load Data** Load combination: Axial Load of Column (P_u) = 1059.749 kN Moment About X-axis $M_{x,1}$ = 30.710 kN-m $M_{x,2}$ = 0 kN-m Moment About Y-axis $M_{y,1}$ = 29.937 kN-m $M_{y,2}$ = 0 kN-m	
4.	**Flexural Design of Column** Slenderness Check L_{ex}/D_x = 2.053/ (500/1000) = 4.11 Since, L_{ex}/D_x <12, Design as Short Column L_{ey}/D_y = 2.053/ (500/1000) = 4.11 Since, L_{ey}/D_y <12, Design as Short Column Minimum Eccentricities $e_{min} = \frac{L}{500} + \frac{D}{30} \geq 20mm$ $e_{x,min}$ = 3.658×1000/500+500/30 = 23.983 mm $e_{y,min}$ = 3.658×1000/500+500/30 = 23.983 mm	IS 456:2000 CL 25.1.2 IS 456:2000 CL 25.4

Step	Calculation	Reference
	Moment Due to Eccentricities $M_{ex} = e_{x,min} \times P_u$ = 23.982×1059.749 /1000 = 25.416 kN-m $M_{ey} = e_{y,min} \times P_u$ = 23.982×1059.749 /1000 = 25.416 kN-m Hence, Design Moment M_{ux1} = 30.710 kN-m M_{uy1} = 29.937 kN-m For Biaxially Loaded Column Assume Percentage of Steel (p_t) = 1.2% Gross Area (As) = 500×500 = 250000 mm^2 Moment Carrying Capacity ($M_{ux,y}$) Along X-axis d'/D = 58/500 = 0.116 P_t/f_{ck} = 1.2/25 = 0.048 $P_u/f_{ck}bD$ = 1059.749×1000/ (25×500×500) = 0.170 $M_u/f_{ck}bD^2$ = 0.10 $M_{ux} = 0.1 \times 25 \times 500 \times 500^2/10^6$ = 312.500 kN-m Along Y-axis d'/D = 58/500 = 0.116 P_t/f_{ck} = 1.5/25 = 0.048 $P_u/f_{ck}bD$ = 1059.749×1000/ (25×500×500) = 0.170 $M_u/f_{ck}bD^2$ = 0.10 $M_{uy} = 0.1 \times 25 \times 500 \times 500^2/10^6$ = 312.500 kN-m	SP-16 Chart 49 SP-16 Chart 49
5.	**Check For Interaction Formula** Axial Load Carrying Capacity (P_{uz}) = $= 0.45 f_{ck} \times A_c + 0.75 \times f_y \times A_{sc}$ = [0.45×25×{1-(1.2/100)}×500×500+0.75×500×(1.2/100)×500×500]/1000 = 3903.750 kN P_u/P_{uz} = 0.271 and by interpolation α_n = 1.119 $\left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1$	IS 456:2000 CL 39.6

Step	Calculation	Reference
	$= (30.710/312.5)^{1.119}+(29.937/312.5)^{1.119} <= 1$ $0.147 <= 1$, Ok So, the design is safe with 1.2% steel. Area of Steel Required (A_{st}) = 1.2/100×500×500 = 3000 mm^2 Let's provide 4-20 mm Φ bar and 12-16 mm Φ bar Area of Steel Provided (A_{st}) $= 4\times\pi\times20^2/4+12\times\pi\times16^2/4 = 3669.380$ mm^2 Percentage of Steel Provided (P_t) = 100×3669.380/ (500×500) = 1.468%	
6.	**Check For Shear** Design Shear Reinforcement from ETABS = 554.22 mm^2 No. of Legs Provided = 4 nos Diameter of tie bar is greater of: i. 1/4×Longitudinal bar diameter = 1/4×25 mm = 6.25 mm ii. 6 mm Thus provide 8 mm diameter bar. Area of shear Reinforcement = Area/No. of legs $= \pi\times8^2/4\times4 = 201.062$ mm^2 Spacing of Tie Required = 201.062/554.22×1000 = 362.784 mm Spacing should be less than: i. Least lateral dimension = 500 mm ii. 16×Longitudinal bar diameter = 16×16 = 256 iii. 300 mm Let's provide Spacing of lateral bars = 150 mm	CL 26.5.3.2 c CL 26.5.3.2 c
7.	**Design comparison with ETABS** From ETABS analysis for column with unique name "176" has rebar area of 3135 mm^2 which is within 10% range of $A_{st,required}$ 3000 mm^2. So, rebar values from ETABS can be used for design purpose. Both of the values are less than provided rebar area of 3669.380 mm^2.	

Step	Calculation	Reference
8.	**Design Summary** Size of column = 500×500 mm Longitudinal reinforcement = 4-20 mm Φ bar and 12-16 mm Φ bar distributed equally on four sides. Transverse reinforcement = 8 mm diameter lateral ties at spacing of 150 mm Clear cover = 40 mm	

8.2 Design of Beams

Beams are structural members (generally horizontal) of moment resisting frames which are subjected to flexural and shear actions. Beam distribute the vertical load to the column and resists the bending moment. The design of the beam deals with the determination of the beam section and the steel required.

For each story of each block maximum reinforcement required in either ends and middle of beams are obtained from ETABS, these beams are taken as critical beams. Value of design moment, factored torsion moment and shear forced acting on the critical beams are obtained from ETABS, these values form basis for design calculation. Here we design for singly reinforced beam subjected to torsion as per IS 456:2000. Design of shear reinforcement is done as per IS 456:2000 Cl 41.3.

Sample design calculation for beams of Story 3 of Block A is shown below:

Table 8-2: ETABS Result for Critical Beams of Story3 of Block A

Design combination: DL + 0.3LL + 0.6LL Storage + RSx

Design Moment M3 (kN-m)	Factored Torsion T(kN-m)	Shear Force V2 (kN)	Ast Req (Etabs)	Location	grid
-93.766	22.52	75.09	651	end-i	F5-G5
48.8466	14.55	69.3	353	mid	G5-H5
-108.5	-18.08	87.6	682	end-j	E4-E5

8.2.1 Design of Beam Considering Negative End-I Moment

Table 8-3: Design of Beam Considering Negative End-I Moment

Step	Calculation	Reference
	Grid of Beam: F5-G5 Unique name: 123 Story 3 of Block A Governing Combo = DL + 0.3LL + 0.6LL Storage + RSx	
1.	**Material Properties** Grade Of Concrete (f_{ck}) = 25 MPa Grade Of Steel (f_y) = 500 MPa Modulus of Elasticity of Steel (E) = 200000MPa	
2.	**Section Properties** Width of Beam (b) = 350 mm Overall Depth of Beam (D) = 500 mm Clear Cover (CC) = 25 mm Effective Cover (d') = CC+ dia of shear bar+ dia of main bar/2 = 25+8+20/2= 43 mm Effective Depth of Beam (d) = D - d' = 500-43 =457 mm Effective Length of Beam (L) = 6096 mm	
3.	**Flexural design of Beam** Governing Combo = DL + 0.3LL + 0.6LL Storage + RSx Design Moment (M_u)= 93.766 kN-m Factored Torsion Moment (T_u)= 22.52 kN-m $M = M_u + T_u\left(\frac{1 + D/b}{1.7}\right)$ Ultimate Design Moment = 93.766+22.52×[{1+(500/350)}/1.7] = 125.937 kN-m	IS456:2000 Cl.41.4.2

Step	Calculation	Reference
	$M_{u,lim} = \frac{0.36 x_{u,max}}{d}\left(1 - 0.42\frac{x_{u,max}}{d}\right) b\, d^2 f_{ck}$ $=(0.36×0.46×(1-0.42×0.46)×350×457^2×25)/10^6$ = 244.156 kNm $M_{u,lim}$/Ultimate Design Moment = 244.156/125.937 = 1.939 >1 So, we design Singly Reinforced Section.	IS 456:2000 G-1.1 (c)
	$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot d \cdot \left(\frac{1 - Ast \cdot fy}{b \cdot d f_{ck}}\right)$ On solving, P_t= 50 (f_{ck}/f_y)×[1-√{1- 4.6×M_u / $(f_{ck}×b×d^2)$}] P_t= 50 (25/500)×[1-√{1- 4.6×125.937 ×10^6/$(25×350×457^2)$}] =0.434 % Area of Tension Steel Corresponding to the Limiting Moment = 0.434/100×350×457 = 694.053 mm^2 $A_{st,required}$ = 694.053 mm^2 Let's provide, 4 numbers of 16 mm dia bars Total Reinforcement Provided = $4×\pi×16^2/4$= 804.25 mm^2 Max bar size adopted = 16 mm	IS 456:2000 Cl,G-1.1.b
4.	**Shear Design of Beam** From ETABS Analysis, Ultimate Shear = 100 kN Shear Force Due to Formation of Plastic Hinge at the End of the Beam (V_b) = 28 kN Max Design Shear Force at Ends (V_{du}) = 28+100 = 128 kN	
	Equivalent Design Shear (V_u) = V_{du} +1.6×T_u/b = 128+1.6×22.52/ (350/1000) = 230.949 kN	IS 456:2000 Cl 41.3.1
	For Required Shear reinforcement (A_{sv}) A_{sv} = V_u x S_v / (0.87f_y x d) = (230.949×1000)/ (0.87×500×457)×1000 = 1161.742 mm^2/m	IS 456:2000, Cl 40.4
	Percentage of Tension Reinforcement % = 100×804.25/ (350×457) = 0.503%	
	Design Shear Strength of Concrete (T_u) = 0.491 N/mm^2	Table 19
	$T_{uc,max}$ = 3.1 N/mm^2	Table 20

Step	Calculation	Reference
	V_{uc} = 0.491×350×457/1000 = 78.535 kN $V_{uc,max}$ = 3.1×350×457/1000 = 495.845 kN Since, $V_{uc,max}$>V_u , Safe in shear. Since, V_{du} >V_{uc} , Shear Reinforcement Should Be Provided Diameter of Shear Reinforcement = 8 mm Area of Shear reinforcement provided = π×8^2/4 = 50.266 mm^2 Number of legs = 2 As = 2×50.266 = 100.531 mm^2 Spacing of Shear Reinforcement (sv) = (0.87×500×100.531×457/ ((230.949-78.535)×1000)) =131.124 mm This spacing should be less than: i. 0.75d = 0.75×457 = 342.75 mm ii. 300 mm iii. 2.5$f_y A_{sv}$/b = 2.5×500×100.531/350 = 359.039 mm For ductility spacing should be less than: i. d/4 = 457/4 = 114.25 mm ii. 8 dia of smallest longitudinal bar = 8×16 = 128 mm Provide 2-legged stirrups of 8mmΦ @100mm c/c distance.	 IS 456:2000 Cl 40.4 IS 456:2000 Cl 26.5.1.5 Cl 26.5.1.6 IS 13920:2016 CL 6.3.5
5.	**Check For Deflection** Required Tension Reinforcement P_t% ={694.053/(350×457)}×100 = 0.434 Provided Tension Reinforcement P_t% = 0.503 Basic Value of Span to Effective Depth Ratio (α) = 26 Modification Factor For span>10 m (β) = 1 fs = 0.58×f_y ($A_{st,required}$/$A_{st,Provided}$) = 0.58×500×(694.053/804.25)= 250.265 Modification Factor for Tension Reinforcement (γ)=1.27 Modification Factor for Compression Reinforcement (δ) = 1 Reduction Factor (λ) = 1	Cl 23.2.1 CL 23.2.1 a CL 23.2.1 b Fig 4 Fig 5 Fig 6

Step	Calculation	Reference
	Allowable span to effective Depth ratio = $\alpha\times\beta\times\gamma\times\delta\times\lambda$ = $26\times1\times1.27\times1\times1 = 33.03$ Calculated Span to Effective Depth Ratio = 6096/457 = 13.339 Since, Calculated ratio is less than Allowable ratio, design is safe in deflection.	
6.	**Design summary** Provide 4-16 mm Φ longitudinal reinforcement and provide 2-legged stirrups of 8 mm Φ @100 mm c/c distance.	
7.	**Comparison with ETABS's design** This beam is designed for maximum End-I moment of story 3 of block A. Reinforcement area of 652 mm^2 was obtained from ETABS analysis which is within 10% range of $A_{st,required}$ 694.053 mm^2 so ETABS values can be used for further design purpose.	

8.2.2 Design of Beam Considering Positive Mid Moment

Table 8-4: Design of Beam Considering Positive Mid Moment

Step	Calculation	Reference
	Grid of Beam: G5-H5 Unique name: 127 Story 3 of Block A Governing Combo = DL + 0.3LL + 0.6LL Storage + RSx	
1.	**Material Properties** Grade Of Concrete (f_{ck}) = 25 MPa Grade Of Steel (f_y) = 500 MPa Modulus of Elasticity of Steel (E) = 200000 MPa	

Step	Calculation	Reference
2.	**Section Properties** Width of Beam (b) = 350 mm Overall Depth of Beam (D) = 500 mm Clear Cover (CC) = 25 mm Effective Cover (d') = CC+dia of shear bar+ dia/2 of main bar = 25+8+20/2 =43 mm Effective Depth of Beam (d) == D- d'=500-43= 457 mm Effective Length of Beam (L) = 6096 mm	
3.	**Flexural design of Beam** Governing Combo = DL + 0.3LL + 0.6LL Storage + RSx Design Moment (M_u)= 48.847 kN-m Factored Torsion Moment (T_u)= 14.55 kN-m $\mathrm{M} = \mathrm{M_u} + \mathrm{T_u}\left(\frac{1 + {}^{\mathrm{D}}/_{\mathrm{b}}}{1.7}\right)$ Ultimate Design Moment (M_u) = 48.847+14.55×{(1+500/350)/1.7} = 69.632 kN-m For Limiting Moment ($M_{u,lim}$) $M_{u,lim} = \frac{0.36 x_{u,max}}{d}\left(1 - 0.42\frac{x_{u,max}}{d}\right) b\, d^2 f_{ck}$ = (0.36×0.46×(1-0.42×0.46)×350×457²×25)/10⁶ = 244.156 kN-m $M_{u,lim}$/Ultimate Design Moment = 244.156/69.632 = 3.506 >1 So, we design Singly Reinforced Section. $M_u = 0.87 \cdot f_y \cdot A_{st} \cdot d \cdot \left(\frac{1 - Ast \cdot fy}{b \cdot d f_{ck}}\right)$ On solving, P_t= 50 (f_{ck}/f_y)×[1-√{1-4.6×M_u /(f_{ck}×b×d^2)}] P_t= 50 (25/500)×[1-√{1-4.6×69.632×10^6/(25×350×457^2)}]=0.23 % Area of Tension Steel corresponding to The Limiting Moment = 0.23/100×350×457 = 367.318 mm^2 $A_{st,required}$ = 367.318 mm^2 Let's provide, 3 numbers of 16 mm dia bars Total Reinforcement Provided = 3×π×16^2/4= 603.186 mm^2	IS456:2000, Cl.41.4.2 IS456:2000, Cl.41.4.2 IS456:2000 G-1.1 (c) IS 456:2000 G-1.1.b

Step	Calculation	Reference
4.	**Shear Design of Beam**	
	From ETABS Analysis, Ultimate Shear = 100 kN	
	Shear Force Due to Formation of Plastic Hinge at the End of the Beam (V_b) = 28 kN	
	Max Design Shear Force at Ends (V_{du}) = 28+100 = 128 kN	
	Equivalent Design Shear (Vu) = V_{du} +1.6×Tu/b = 128+1.6×14.55/ (350/1000) = 194.514 kN	IS 456:2000 Cl 41.3.1
	For Required Shear reinforcement (A_{sv})	
	$A_{sv} = V_u$ x S_v / ($0.87f_y$ x d)	IS 456:2000
	= (194.514×1000)/ (0.87×500×457)×1000 = 978.467 mm^2/m	Cl 40.4
	Percentage of Tension Reinforcement% = 603.187/ (350×457)×100 = 0.377%	
	Design Shear Strength of Concrete (T_u) = 0.451 N/mm^2	Table 19
	$T_{uc,max}$ = 3.1 N/mm^2	Table 20
	V_{uc} = 0.451×350×457/1000 = 72.137 kN	
	$V_{uc,max}$ = 3.1×350×457/1000 = 495.845 kN	
	Since, $V_{uc,max}$ >Vu, Safe in shear.	
	Since, V_{du} >V_{uc} , Shear reinforcement should be provided	
	Diameter of Shear Reinforcement = 8 mm	
	Area of Shear reinforcement provided = π×8^2/4 = 50.266 mm^2	
	Number of legs = 2	
	As = 2×50.266 = 100.531 mm^2	
	Spacing of Shear Reinforcement (sv) = (0.87×500×100.531×457/ ((194.514-72.137)×1000)) = 163.308 mm	IS 456:2000 Cl 40.4
	This spacing should be less than:	IS 456:2000
	i. 0.75d = 0.75×457 = 342.75 mm	Cl 26.5.1.5
	ii. 300 mm	
	iii. $2.5f_y$Asv/b = 2.5×500×100.531/350 = 359.04 mm	Cl 26.5.1.6
	Therefore, provide 2-legged stirrups of 8 mm Φ @150 mm c/c distance.	

Step	Calculation	Reference
5.	**Check For Deflection** Required Tension Reinforcement P_t = 367.318/ (350×457)×100 = 0.23% Provided Tension Reinforcement P_t = 0.377% Basic Value of Span to Effective Depth Ratio (α) = 26 Modification Factor For span>10 m (β) = 1 fs = 0.58×f_y ($A_{st,required}$/$A_{st,Provided}$) = 0.58×500×(367.318/603.187) = 176.599 Modification Factor for Tension Reinforcement (γ) = 1.901 Modification Factor for Compression Reinforcement (δ) = 1 Reduction Factor (λ) = 1 Allowable span to effective Depth ratio = α×β×γ×δ×λ = 26×1×1.901×1×1 = 49.433 Calculated Span to Effective Depth Ratio = 6096/457 = 13.339 Since, Calculated ratio is less than Allowable ratio, design is safe in deflection.	Cl 23.2.1 CL 23.2.1 a CL 23.2.1 b Fig 4 Fig 5 Fig 6
6.	**Design summary** Provide 3-16 mm Φ longitudinal reinforcement and provide 2-legged stirrups of 8 mm Φ @150 mm c/c distance.	
7.	**Comparison with ETABS's design** This beam is designed for maximum Mid moment of story 3 of block A. Reinforcement area of 353 mm^2 was obtained from ETABS analysis which is within 10% range of $A_{st,required}$ 367.318 mm^2 so ETABS values can be used for further design purpose.	

8.2.3 Design of Beam Considering Negative End-J Moment

Table 8-5: Design of Beam Considering Negative End-J Moment

Step	Calculation	Reference
1.	Grid of Beam: E4-E5 Unique name: 449 Story 3 of Block A Governing Combo = DL + 0.3LL + 0.6LL Storage + RSx **Material Properties** Grade Of Concrete (f_{ck}) = 25 MPa Grade Of Steel (f_y) = 500 MPa Modulus of Elasticity of Steel (E) = 200000 MPa	
2.	**Section Properties** Width of Beam (b) = 350 mm Overall Depth of Beam (D) = 500 mm Clear Cover (CC) = 25 mm Effective Cover (d') = CC+dia of shear bar+ dia/2 of main bar =25+8+20/2 = 43 mm Effective Depth of Beam (d) = D- d'=500-43 = 457 mm Effective Length of Beam (L) = 6096 mm	
3.	**Flexural design of Beam** Governing Combo = DL + 0.3LL + 0.6LL Storage + RSx Design Moment = 108.5 kN-m Factored Torsion Moment = 18.1 kN-m $\mathrm{M} = \mathrm{M_u} + \mathrm{T_u}\left(\frac{1 + {}^{\mathrm{D}}/_{\mathrm{b}}}{1.7}\right)$ Ultimate Design Moment (M_u)= 108.5+18.1×{(1+(500/350)/1.7} = 134.357 kN-m For Limiting Moment ($M_{u,lim}$) $M_{u,lim} = \frac{0.36x_{u,max}}{d}\left(1 - 0.42\frac{x_{u,max}}{d}\right) b\, d^2 f_{ck}$	IS 456:2000 Cl.41.4.2 IS 456:2000 G-1.1 (c)

Step	Calculation	Reference
	$= (0.36\times0.46\times(1-0.42\times0.46)\times350\times457^{2\times}25)/ 10^6$ $= 244.156$ kN-m $M_{u,lim}$/Ultimate Design Moment = 244.156/134.357 = 1.817 >1 So, we design Singly Reinforced Section. $M_u = 0.87 \cdot f_y \cdot A_{st} \cdot d \cdot \left(\frac{1 - Ast \cdot fy}{b \cdot df_{ck}}\right)$ On solving, $P_t = 50\ (f_{ck}/f_y)\times[1-\sqrt{\{1-4.6\times M_u / (f_{ck}\times b\times d^2)\}}]$ $P_t=50(25/500)\times[1-\sqrt{\{1-4.6\times134.357\times10^6/(25\times350\times457^2)\}}]=0.466\%$ Area of Tension Steel corresponding to the Limiting Moment = 0.466/100×350×457 = 745.732 mm^2 $A_{st,required}$ = 745.732 mm^2 Let's provide, 4 numbers of 16 mm dia bars Total Reinforcement Provided = $4\times\pi\times16^2/4$= 804.25 mm^2 Max bar size adopted = 16 mm	IS 456:2000 G-1.1 (b)
4.	**Shear Design of Beam** Given Ultimate Shear = 100 kN Shear Force Due to Formation of Plastic Hinge at the End of the Beam (V_b) = 28 kN Max Design Shear Force at Ends (V_{du}) = 28+100 = 128 kN Equivalent Design Shear (Vu) = V_{du} +1.6×T_u/b = 128+1.6×18.1/ (350/1000) = 210.743 kN For Required Shear reinforcement (A_{sv}) $A_{sv} = V_u$ x S_v / (0.87f_y x d) = (210.743×1000)/ (0.87×500×457)×1000 = 1060.101 mm^2/m Percentage of Tension Reinforcement% = 804.25/ (350×457)×100 = 0.503% Design Shear Strength of Concrete (Tu) = 0.491 N/mm^2 $T_{uc,max}$ = 3.1 N/mm^2 V_{uc} = 0.491×350×457/1000 = 78.535 kN $V_{uc,max}$ = 3.1×350×457/1000 = 495.845 kN Since, $V_{uc,max}$ >V_u ,SAFE	IS 456:2000 Cl 41.3.1 IS 456:2000 Cl 40.4 Table 19 Table 20

Step	Calculation	Reference
	Since, $V_{du}>V_{uc}$, Shear Reinforcement Should Be Provided Diameter of Shear Reinforcement = 8 mm Area of Shear reinforcement provided = $\pi\times8^2/4$ = 50.266 mm^2 Number of legs = 2 As = 2×50.266 = 100.531 mm^2 Spacing of Shear Reinforcement (sv) = (0.87×500×100.531×457/ ((210.743-78.535)×1000)) = 151.165 mm This spacing should be less than: i. 0.75d =0.75×457 = 342.75 mm ii. 300 mm iii. $2.5f_y$Asv/b = 2.5×500×100.531/350 = 359.04 mm For Ductility spacing should be less than: i. d/4 = 457/4 = 114.25 mm ii. 8 dia of Smallest longitudinal Bar = 8×16 = 128 mm Thus, provide 2-legged stirrups of 8 mm Φ @100mm c/c distance.	Cl 40.4 IS 456:2000 Cl 26.5.1.5 Cl 26.5.1.6 IS13920:2016 CL 6.3.5
5.	**Check for Deflection** Required Tension Reinforcement Pt% = 745.732/ (350×457)×100 = 0.466 Provided Tension Reinforcement Pt% = 0.503 Basic Value of Span to Effective Depth Ratio (α) = 26 Modification Factor For span>10 m (β) = 1 fs = $0.58\times f_y$ ($A_{st,required}/A_{st,Provided}$) = 0.58×500×(745.732/804.25) = 268.899 Actual Modification Factor for Tension Reinforcement (γ) = 1.183 Modification Factor for Compression Reinforcement (δ) = 1 Reduction Factor (λ) = 1 Allowable span to effective Depth ratio = α×β×γ×δ×λ = 26×1×1.183×1×1 = 30.767 Calculated Span to Effective Depth Ratio = 6096/457 = 13.339 Since, Calculated ratio is less than Allowable ratio, design is safe in deflection.	Cl 23.2.1 CL 23.2.1 a CL 23.2.1 b Fig 4 Fig 5 Fig 6

Step	Calculation	Reference
6.	**Design summary** Provide 4-16 mmΦ longitudinal reinforcement and provide 2-legged stirrups of 8 mmΦ @100 mm c/c distance.	
7.	**Comparison with ETABS's design** This beam is designed for maximum End-J moment of story 3 of block A. Reinforcement area of 688 mm^2 was obtained from ETABS analysis which is within 10% of $A_{st,required}$ 745.732 mm^2 so ETABS values can be used for further design purpose.	

8.3 Design of Cantilever Beam

A cantilever is a rigid structural element that extends horizontally and is supported at only one end. Typically, it extends from a flat vertical surface such as a wall, to which it must be firmly attached. In this project, ramp is supported by cantilever beams extending from walls.

Table 8-6: Design of Cantilever Beam

S.N	Calculation	Reference
1.	**Material Properties** Grade of Concrete (f_{ck}) = 25 N/mm^2 Grade of Steel (f_y) = 500 N/mm^2	
2.	**Load Calculation** Span of slab for one beam = 6938 mm = 6.938 m Thickness of Slab = 120 mm Load due to slab span = $120 \times 25/10^3$ = 3 kN/m^2 Live Load on slab = 5 kN/m^2 Floor finish Load = 1.5 kN/m^2 Total Area Load On Slab = 3+5+1.5 = 9.5 kN/m^2 Load on cantilever beam due to live load = 9.5× 6.938 = 65.911 kN/m	

S.N	Calculation	Reference
3.	**Preliminary Depth Calculation** Clear span of Cantilever beam (L)= 2560 mm Let, P_t = 1.5% Basic value of Span to Effective Depth Ratio (α) = 7 (for cantilever) Modification factor for span (β) = 1 (for span less than 10m) Modification factor for tension reinforcement (γ) = 1 Modification factor for compression reinforcement (δ) = 1 Reduction factor (λ) = 0.9 Minimum depth of beam = 2560/ (7×1×1×1×0.9) = 406.349 mm Let's adopt overall depth (D) = 500 mm Width (b) = 350 mm Effective cover (d') = 25+8+16+25/2 = 61.5 mm Effective depth (d) = D- d' = 500-61.5 = 438.5 mm $X_{u,max}/d$ = 0.46 for f_y = 500 MPa $X_{u,max}$ = 0.46×438.5 = 201.71 mm	 Cl 23.2.1 a Cl 23.2.1 b Fig 4 Fig 5 Fig 6
4.	**Depth Check** Dead load of beam =Vol.×Unit.wt =500×350×2560×25/10^9 = 11.2 kN Centre of gravity of dead load of the beam from fixed end = (2560/2) ×1000 = 1.28 m Factored BM(M_u)=1.5(11.2×1.28+65.911×2.560^2/2) = 345.470 kN-m Then, Limiting Moment $M_{u,lim} = \frac{0.36x_{u,max}}{d}\left(1-0.42\frac{x_{u,max}}{d}\right) b\, d^2 f_{ck}$ $M_{u,lim}$=0.36×0.46×(1-0.42×0.46) ×350×438.5^2×25 =224.788 kN-m Since $M_{u,lim} < M_u$, Design Doubly Reinforced Section	ANNEX G
5.	**Flexural Design** Ultimate Design Moment (M_u) = 345.470 kN-m Limiting Moment ($M_{u,lim}$)= 224.788 kN-m $M_u = 0.87 \cdot f_y \cdot A_{st} \cdot d \cdot \left(\frac{1 - Ast \cdot fy}{b \cdot df_{ck}}\right)$	Cl. 41.4.2 IS456:2000 G-1.1.b

S.N	Calculation	Reference
	On solving, P_t= 50 $(f_{ck}/f_y)\times[1-\sqrt{\{1-4.6\times M_u /(f_{ck}\times b\times d^2)\}}]$ Percentage Tension Reinforcement Required P_t $= 50\times(25/500)\times[1-\sqrt{\{1-4.6\times224.788\times10^6/(25\times350\times438.5^2)\}}]=0.948\%$ Area of tension steel(A_{st1})=0.948×350×438.5/100 =1454.943 Now, $M_{u2} = M_u-M_{u,lim}$ = 345.47-224.788= 120.682 kN-m $0.0035(X_{u,max}-d')/X_{u,max}$ = 0.0035×(201.71- 61.5)/201.71 = 0.00243 From Sp-16 Table A, for strain 0.00243 and f_y=500 N/mm^2) f_{sc}= 398.533 kN-m For compression reinforcement Area of Compression Steel (A_{sc}) = $(M_u-M_{u,lim})/f_{sc}(d- d')$ $=120.682\times10^6/ (398.533\times(438.5-61.5))$ = 803.224 mm^2 Let's provide 2-20 mm and 1-16 mm diameter bars Total Reinforcement Provided = $2\times\pi\times20^2/4+\pi\times16^2/4$ = 829.380 mm^2 Percentage Compression Reinforcement Provided = 100×829.380/ (350×438.5) = 0.54% Also, for tension reinforcement $A_{st2} = A_{sc} \times f_{sc}/0.87f_y$ = 803.224×398.533/(0.87×500) = 735.890 mm^2 Total Tension Reinforcement (A_{st1}+A_{st2}) = 1454.943+735.890 = 2190.833 mm^2 Let's provide 7-20 mm diameter bars Total Reinforcement Provided = $7\times\pi\times20^2/4$ = 2199.11 mm^2 Percentage Tension Reinforcement Provided (P_t) = 100×2199.11/ (350×438.5) = 1.433%	G-1.2
6.	**Check For Maximum Shear** Factored maximum shear forces at fixed end, V_u = 1.5×11.2+1.5×65.911×2560/1000 = 269.898 kN Nominal Shear Stress, $\tau_v=\frac{V_u}{bd}=\frac{269.898}{350\times438.5}$ =1.759 N/mm^2	CL 40.1

S.N	Calculation	Reference
	Design shear strength of Concrete (τ_{uc})= 0.729 N/mm^2 ,for P_t=1.433% Maximum Shear Stress ($\tau_{uc,max}$) = 3.1 N/mm^2 Since, $\tau_v < \tau_{uc,max}$ Design is safe in shear. But since, $\tau_{uc} < \tau_v$ Shear reinforcement should be provided. Let's provide 2-legged 8 mm dia shear reinforcement Area of Shear Reinforcement (A_{sv}) = 2×π×8^2/4 = 100.531 mm^2 Spacing of Shear Reinforcement (sv) = 0.87×f_y×A_{sv}×d/ (V_u-V_{uc}) = 0.87×500×100.531×438.5/ ((1.759-0.729)×438.5×350) = 121.306 mm This spacing is less than following: i. 0.75d = 0.75×438.5 = 328.88 mm ii. 300 mm iii. Sv = 0.87f_y×A_{sv}/ (b×0.4) = 312.365 mm Thus provide 2-legged 8 mm diameter Fe500 steel as vertical stirrup @ 100 mm c/c spacing.	Table 19 Table20 Cl 26.5.1.5 Cl 26.5.1.6
7.	**Check For Deflection** Basic Value of Span to Effective Depth Ratio (α) = 7 Modification Factor for span>10 m (β) = 1 fs = 0.58×f_y ($A_{st,required}$/$A_{stprovided}$) = 0.58×500×(2190.833/2199.11) = 288.908 Modification factor for tension reinforcement(γ)=0.8 ,for P_t=1.433% Modification factor for comp. reinforcement(δ)= 1.152 ,for P_t=0.54% Reduction Factor (λ) = 1 Allowable span to Eff. Depth ratio = α×β×γ×δ×λ = 7×1×0.8×1.152×1 = 6.451 Calculated Span to Effective Depth Ratio = 2560/438.5 = 5.838 Since, allowable span ratio is greater than calculated span ratio. The design is safe in deflection.	 Cl 23.2.1 a Cl 23.2.1 b Fig 4 Fig 5 Fig 6

8.4 Design of Slab

A slab is a flat, two-dimensional planar structural component of building having a very small thickness compared to its other two dimensions. Slabs form floors and roofs of a building and carry distributed load primarily by flexure. A slab is generally designed as a flexural element considering a strip of 1 m width, even though it is cast in one piece and not in strips of unit width. Hence, for the purpose of design, a slab is equivalent to a rectangular beam, with b = 1000 mm. The moment of resistance and the required area of reinforcement are expressed per unit width. As slabs are thin compared to beams, the serviceability limit state of deflection is normally critical in slabs, rather than the ultimate limit state.

8.4.1 Design of Slab of Block A and Block B

In block A and block B, there are four different types of slabs, the type of slabs with their bending moment coefficient as per table-26 of IS 456:2000 is given in table below. Design for the slabs having maximum bending moment coefficient will be considered conservative design for other types of slabs. So, we design for two adjacent edges discontinuous slab and its design outcome will be adopted for rest of the slabs. And torsional reinforcement is provided only in discontinuous edges. For l_{ey}/l_{ex} = 5.819/2.798 = 2.0

Table 8-7: Bending Moment Coefficients of Slabs of Block A and B

Type of Slab	Short span coefficient		Long span coefficient		Remarks
	Negative moment at continuous edge (α_x^{-ve})	Positive moment at mid-span (α_x^{+ve})	Negative moment at continuous edge (α_y^{-ve})	Positive moment at mid-span (α_y^{+ve})	
Interior Panels	0.065	0.049	0.032	0.024	
One long edge discontinuous	0.085	0.065	0.037	0.028	
One short edge discontinuous	0.068	0.052	0.037	0.028	
Two adjacent edges discontinuous	0.091	0.069	0.047	0.035	**Critical**

Table 8-8: Design of Two Adjacent Edges Discontinuous Slab of Block A and B

Step	Calculation	Reference
1.	**Calculation of effective depth** c/c distance of shorter length (l_x) = 3.048 m c/c distance of longer length (l_y) = 6.069 m Width of Support = 0.35 m For two adjacent edge discontinuous, l_x/d = (35+40)/2=37.5 d = l_x/37.5= 1000×3.048/37.5 = 81.28 mm , let d=100 mm Adopt diameter of main bar (Φ) = 10 mm Adopt diameter of distribution bar (Φ) = 10 mm Clear cover (CC)= 15 mm Overall depth (D) =d+ Φ/2 + CC = 100+(10/2) +15=120 mm Dia. of reinforcing bars is chosen in such a way that it shall not exceed 1/8th of the total thickness of slab 1/8th of total thickness of slab = 120/8 =15 mm >10 mm, ok Provide Slab thickness of 120 mm.	IS 456:2000 CL 24.1 IS 456:2000 CL 26.5.2.2
2.	**Calculation of effective length** Along shorter length Clear span = l_x -support width =3.048-0.35= 2.698 mm clear span + eff. Depth (d_x) = 2.698+(100/1000) = 2.798 m l_{ex} = minimum of 3.048 and 2.798 = 2.798 m Along longer length Clear span = l_y -support width =6.069-0.35= 5.719 mm clear span + eff. Depth (d_y)= 5.719+(100/1000) = 5.819 m l_{ey} = minimum of 6.069 and 5.819 = 5.819 m l_{ey}/l_{ex} = 5.819/2.798 = 2 Since $l_{ey}/l_{ex} \leq 2$, it is two-way slab.	IS 456:2000 Cl 22.2 (a)
3.	**Load Calculation** Compressive Strength of Concrete (f_{ck}) = 25 N/mm^2 Minimum Yield Strength of Steel (f_y) = 500 N/mm^2	

Step	Calculation	Reference
	From Table-1 Item no. 22 of IS: 875 (Part 1)-1987 Unit weight of concrete (γ) = 25 kN/m^2 (For 1 m width of slab) Live Load (LL) = 5 kN/m^2= 5 kN/m^2×1 m= 5 kN/m Floor Finish (FF) = 1.5 kN/m Partition wall = 2 kN/m Self-weight = γ×D = 25×120 = 3000 N/m= 3 kN/m Total Load = 5+1.5+2+3=11.5 kN/m Factored Load = 1.5×11.5 = 17.25 kN/m	
4.	**Maximum bending moment calculation** Type of Panel = Two Adjacent Edges Discontinuous The value of l_{ey}/l_{ex} and α is taken from IS 456:2000 Table-26 $\alpha_x^{-ve} = 0.091$ $\alpha_x^{+ve} = 0.069$ $\alpha_y^{-ve} = 0.047$ $\alpha_y^{+ve} = 0.035$ Moments: $M_x^{-ve} = \alpha_x^{-ve} \times W \times l_x^2 = 0.091\times17.25\times2.798^2 = 12.289$ kN-m $M_x^{+ve} = \alpha_x^{+ve} \times W \times l_x^2 = 0.069\times17.25\times2.798^2 = 9.318$ kN-m $M_y^{-ve} = \alpha_y^{-ve} \times W \times l_x^2 = 0.047\times17.25\times2.798^2 = 6.347$ kN-m $M_y^{+ve} = \alpha_y^{+ve} \times W \times l_x^2 = 0.035\times17.25\times2.798^2 = 4.727$ kN-m	
5.	**Check for depth against bending** $M_{u,lim}=\frac{0.36x_{u,max}}{d}\left(1\text{-}0.42\frac{x_{u,max}}{d}\right)bd^2f_{ck}$ $M_{max} = 0.133\times f_{ck}\times b\times d_{req}^2$ $12.289\times10^6 = 0.133\times25\times1000\times d_{req}^2$ d_{req} = 60.794 mm d_{req}< effective depth provided (100 mm.) OK	ANNEX G

Step	Calculation	Reference
6.	**Area of Steel Calculation** $M_{x-ve} = 0.87 \cdot f_y \cdot A_{st-ve} \cdot d_x \cdot \left(1 - \frac{f_y \cdot A_{st-ve}}{b \cdot d_x \cdot f_{ck}}\right)$ $12.289\times10^6 = 0.87\times500\times(A_{st})_x^{+ve}\times100\times[1-\{(A_{stx}^{+ve}\times500)/(1000\times100\times25)\}]$ $Ast_x^{-ve} = 300.575\ mm^2$ Similarly, $Ast_x^{+ve} = 224.266\ mm^2$ $Ast_y^{-ve} = 150.434\ mm^2$ $Ast_y^{+ve} = 111.137\ mm^2$ $(A_{st})_{min}$ = 0.12% of bD = 0.12%×1000×120=144 mm^2 Taking maximum area between A_{st} and $(A_{st})_{min}$ for spacing: $(Spacing)_x^{-ve} = 1000/[300.575/\{(\pi\times10^2)/4\}] = 261.299\approx260$ $(Spacing)_x^{+ve} = 1000/[224.266/\{(\pi\times10^2)/4\}] = 350.208\approx350$ $(Spacing)_y^{-ve} = 1000/[150.434/\{(\pi\times10^2)/4\}] = 522.088\approx520$ $(Spacing)_y^{+ve} = 1000/[144/\{(\pi\times10^2)/4\}] = 545.415\approx545$ Maximum Spacing of main bar is minimum of (i) & (ii) (i) 3d = 3×100 = 300 mm (ii) 300 mm Maximum Spacing of distribution bar is minimum of (i) & (ii) (i) 5d = 5×100 = 500 mm (ii) 450 mm Maximum allowable spacing for main bar = 260 mm Maximum allowable spacing for distribution bar = 450 mm Provide 10 mm dia. bar @200 mm c/c at support of X-axis Provide 10 mm dia. bar @200 mm c/c at mid-span of X- axis Provide 10 mm dia. bar @200 mm c/c at support of Y-axis Provide 10 mm dia. bar @200 mm c/c at mid-span of Y-axis	ANNEX G

Step	Calculation	Reference
	$A_{st,\ provided}$ = {1000×(π×10^2)/4}/200 = 392.699 mm^2 Percentage steel provided = Area provided/ (b×D) = (392.699×100)/ (1000×120) = 0.327%	
7.	**Check for shear force** V_{max} = (17.25×2.798)/2 = 24.133 kN τ_v = (24.133×10^3)/ (1000×100) = 0.241 N/mm^2 For M25 concrete and (A_{st}×100)/bd= 392.699×100/(1000×100)= 0.393 Design shear strength of concrete (τ_c) = 0.434 N/mm^2 k = 1.3 But, for solid slab Design shear strength of concrete= k×τ_c = 1.3×0.434 = 0.564 N/mm^2 Maximum shear stress $(\tau_c)_{max}$ = 3.1 N/mm^2 $\tau_v < k\tau_c < (\tau_c)_{max}$,safe in shear.	 Table 19 Cl 40.2.1.1 Table 20
8.	**Check for deflection** Basic value: α =(20+26)/3= 23 Modification factor (β) = 1 For Calculating the factor γ Steel Stress of service load (f_s) = 0.58×500×[($A_{st.req.}$)/ ($A_{st.provided}$)] =0.58×500×[224.266/392.699] = 165.616 For Tension Reinforcement (γ) = 2 ,for P_t=0.327% For Compression Reinforcement (δ) = 1 Reduction factor for flanged beam (λ) = 1 α×β×γ×δ×λ = 23×1×2×1×1 = 46 l/d = 3.048/ (100/1000) = 30.48 Since, (l/d) < α×β×γ×δ×λ Ok, safe in deflection.	 Cl 23.2.1(a) Cl 23.2.1(b) Cl 23.2.1(c) fig 4 Cl 23.2.1(d) fig 5 Cl 23.2.1(e) fig 6

Step	Calculation	Reference
9.	**Check for development length** $L_d = \frac{\Phi * \sigma s}{4 * \tau bd}$ =(10×435)/ (4×1.4×1.6)) = 485.491 mm Where, Nominal dia. of bar (Φ) = 10 Stress in bar (σ_s) = 0.87×f_y = 0.87×500 = 435 Design bond stress (τ_{bd}) = 1.4 M_l = 0.87×500×392.699×100×[1-{392.699×500)/ (1000×100×25)}] = 15.741 kN-m let L_o = 0 $(1.3M_l)/V + L_o = (1.3\times15.741\times10^6)/(24.396\times10^3)+0$ = 838.797 mm Since, $L_d < [(1.3M_l)/V + L_o]$ Ok	IS 456:2000 Cl 26.2.1 Cl.2.1.1 ANNEX G G-1.1(b) Cl 26.2.3.3
10.	**Torsional Reinforcement at corners** Extreme corner Size of mesh = lx/5 = 609.6 mm Size of wall = 350 mm Provide mesh of size = 609.6+350 = 959.696≈ 960 mm Area of torsional reinforcement = 3/4×$A_{st\ req}$ = 3/4×300.575 = 225.431 mm^2 Use 8 mm diameter bar Spacing = 1000×(π×8^2/4)/ 225.431 = 222.975 mm Spacing provided = 200 mm Let's provide 8 mm diameter bar @ 200 c/c spacing in discontinuous edge.	
11.	**Reinforcement in edge strip** Provide edge strip of 0.12% of gross area. = 0.12%×1000×120 = 144 mm^2 Provide 8 mm diameter bar Spacing = (π×8^2/4)×1000/144 = 349.056 ≈340 mm	

Step	Calculation	Reference
12.	**Design summary** Effective Depth (d) = 100 mm Clear cover = 15 mm Overall depth (D) = 120 mm For middle strip (3l/8): Main bar Reinforcement (at short span) =10 mm dia. bar @ 200 c/c Distribution bar Reinforcement (at long span) = 10 mm dia. bar @200 c/c For edge strip: Provide 8 mm diameter bar @ 340 mm at length of l/8. For torsional reinforcement at corner: Provide 8 mm diameter bar @ 200 c/c spacing in discontinuous edge on mesh size of 960 mm	

8.4.2 Design of Slab of Block C

For Block C l_{ey}/l_{ex} = 2.798 /2.798 = 1

Table 8-9: Bending Moment Coefficients of Slabs of Block C

Type of Slab	Short span coefficient		Long span coefficient		Remarks
	Negative moment at continuous edge (α_x^{-ve})	Positive moment at mid-span (α_x^{+ve})	Negative moment at continuous edge (α_y^{-ve})	Positive moment at mid-span (α_y^{+ve})	
Interior Panels	0.032	0.024	0.032	0.024	
One long edge discontinuous	0.037	0.028	0.037	0.028	
One short edge discontinuous	0.037	0.028	0.037	0.028	
Two adjacent edges discontinuous	0.047	0.035	0.047	0.035	**Critical**

Table 8-10: Design of Two Adjacent Edges Discontinuous Slab of Block C

SN	Calculation	Reference
1.	**Calculation of effective depth** c/c distance of shorter length (l_x) = 3.048 m c/c distance of longer length (l_y) = 3.048 m Width of Support = 0.35 m For two adjacent edge discontinuous, l_x/d = (35+40)/2=37.5 d = l_x/37.5= 1000×3.048/37.5 = 81.28, let d=100 mm Adopt diameter of main bar (Φ) = 10 mm Adopt diameter of distribution bar (Φ) = 10 mm Clear cover (CC) = 15 mm Overall depth (D) =d+ Φ/2+ CC = 100+(10/2) +15=120 mm Dia. of reinforcing bars is chosen in such a way that it shall not exceed 1/8th of the total thickness of slab 1/8th of total thickness of slab = 15 mm > 10 mm, ok Provide Slab thickness of 120 mm.	IS 456:2000 CL 24.1 IS 456:2000 CL 26.5.2.2
2.	**Calculation of effective length** Along shorter length Clear span = l_x -support width =3.048-0.35= 2.698 mm clear span + eff. Depth (d_x) = 2.698+(100/1000) = 2.798 m l_{ex} = minimum of 3.048 and 2.798 = 2.798 m Along longer length Clear span = l_y -support width =3.048-0.35= 2.698 mm clear span + eff. Depth(d_y)= 2.698+(100/1000)= 2.798 m l_{ey} = minimum of 3.048 and 2.798 = 2.798 m l_{ey}/l_{ex} = 2.798/2.798 = 1 Since $l_{ey}/l_{ex} \leq 2$, it is two-way slab.	IS 456:2000 Cl 22.2 (a)
3.	**Load Calculation** Compressive Strength of Concrete (f_{ck}) = 25 N/mm^2 Minimum Yield Strength of Steel (f_y) = 500 N/mm^2	

SN	Calculation	Reference
	From Table-1 Item no. 22 of IS: 875 (Part 1)-1987 Unit weight of concrete (γ) = 25 kN/m^2 (For 1 m width of slab) Live Load (LL) = 5 kN/m^2 = 5 $kN/m^2 \times$1 m= 5 kN/m Floor Finish (FF) = 1.5 kN/m Partition wall = 2 kN/m Self-weight = $\gamma \times D$ = 25×120 = 3000 N/m =3 kN/m Total Load = 5+1.5+2+3=11.5 kN/m Factored Load = 1.5×11.5 = 17.25 kN/m	
4.	**Maximum bending moment calculation** Type of Panel = One Long Edge Discontinuous The value of l_{ey}/l_{ex} and α is taken from IS 456:2000 Table-26 α_x^{-ve} = 0.047 α_x^{+ve} = 0.035 α_y^{-ve} = 0.047 α_y^{+ve} = 0.035 Moments: $M_x^{-ve} = \alpha_x^{-ve} \times W \times l_x^2 = 0.047 \times 17.25 \times 2.798^2 = 6.347$ kN-m $M_x^{+ve} = \alpha_x^{+ve} \times W \times l_x^2 = 0.035 \times 17.25 \times 2.798^2 = 4.727$ kN-m $M_y^{-ve} = \alpha_y^{-ve} \times W \times l_x^2 = 0.047 \times 17.25 \times 2.798^2 = 6.347$ kN-m $M_y^{+ve} = \alpha_y^{+ve} \times W \times l_x^2 = 0.035 \times 17.25 \times 2.798^2 = 4.727$ kN-m	
5.	**Check for depth against bending** $M_{u,lim} = \frac{0.36x_{u,max}}{d}\left(1-0.42\frac{x_{u,max}}{d}\right) b\, d^2 f_{ck}$ $M_{max} = 0.133 \times f_{ck} \times b \times d_{req}^2$ $6.347 \times 10^6 = 0.133 \times 25 \times 1000 \times d_{req}^2$ d_{req} = 43.691 mm d_{req}< effective depth provided (100 mm.) OK	ANNEX G

SN	Calculation	Reference
6.	**Area of Steel Calculation** $M_{x-ve} = 0.87 \cdot f_y \cdot A_{st-ve} \cdot dx \cdot \left(\frac{1 - fy \cdot Ast}{b \cdot dx f_{C^k}}\right)$ $6.347 \times 10^6 = 0.87 \times 500 \times (A_{st})_x^{+ve} \times 100 \times [1-\{(Ast_x^{+ve} \times 500)/$ $(1000 \times 100 \times 25)\}]$ $Ast_x^{-ve} = 150.434\ mm^2$ Similarly, $Ast_x^{+ve} = 111.137\ mm^2$ $Ast_y^{-ve} = 150.434\ mm^2$ $Ast_y^{+ve} = 111.137\ mm^2$ $(A_{st})_{min} = 0.12\%$ of bD $= 0.12\% \times 1000 \times 120 = 144\ mm^2$ Taking maximum area between A_{st} and $(A_{st})_{min}$ for spacing: $(Spacing)_x^{-ve} = 1000/[150.434/\{(\pi \times 10^2)/4\}] = 522.088 \approx 520$ mm $(Spacing)_x^{+ve} = 1000/[144/\{(\pi \times 10^2)/4\}] = 545.451 \approx 545$ mm $(Spacing)_y^{-ve} = 1000/[150.434/\{(\pi \times 10^2)/4\}] = 522.088 \approx 520$ mm $(Spacing)_y^{+ve} = 1000/[144/\{(\pi \times 10^2)/4\}] = 545.451 \approx 545$ mm Maximum allowable spacing of main bar is minimum of: (i)3d = 3×100 = 300 mm (ii)300 mm Maximum allowable spacing of distribution bar is minimum of below two values: (i)5d = 5×100 = 500 mm (ii)450 mm Thus, Maximum allowable spacing for main bar = 300 mm Maximum allowable spacing for distribution bar = 450 mm Provide 10 mm dia. bar @200 mm c/c at support of X-axis Provide 10 mm dia. bar @200 mm c/c at mid-span of X-axis Provide 10 mm dia. bar @200 mm c/c at support of Y-axis	ANNEX G

SN	Calculation	Reference
	Provide 10 mm dia. bar @200 mm c/c at mid-span of Y-axis $(A_{st})_x^{+ve}$ provided = $\{1000\times(\pi\times10^2)/4\}/200 = 392.699$ mm^2 Percentage steel provided = Area provided/ (b×D) = (392.699×100)/ (1000×120) = 0.327%	
7.	**Check for shear force** V_{max} = (17.25×2.798)/2 = 24.133 kN $\tau_v = (24.133\times10^3)/(1000\times100) = 0.241$ N/mm^2 For M25 concrete and $(A_{st}\times100)/bd$= 392.699×100/(1000×100)= 0.393% Design shear strength of concrete (τ_c) = 0.434 N/mm^2 k = 1.3 For solid slab Design Shear strength of concrete=$k\times\tau_c$ = 1.3×0.428 = 0.557 N/mm^2 Maximum shear stress $(\tau_c)_{max}$ = 3.1 N/mm^2 $\tau_v < k\tau_c < (\tau_c)_{max}$, safe in shear	 Table 19 CL 40.2.1 Table 20
8.	**Check for deflection** Basic value: α = (20+26)/3= 23 Modification factor (β) = 1 For Calculating the factor γ Steel Stress of service load (f_s) = $0.58\times500\times[(A_{st.req.})/(A_{st.provided})]$ =0.58×500×[111.137/392.699]= 82.072 For Tension Reinforcement (γ) = 2 ,for P_t=0.327% For Compression Reinforcement (δ) = 1 Reduction factor for flanged beam (λ) = 1 $\alpha\times\beta\times\gamma\times\delta\times\lambda = 23\times1\times2\times1\times1 = 46$ l/d = 3.048/ (100/1000) = 30.48 Since, (l/d) < $\alpha\times\beta\times\gamma\times\delta\times\lambda$ Ok, safe in deflection.	 Cl 23.2.1(a) Cl 23.2.1(b) Cl 23.2.1(c) fig 4 Cl 23.2.1(d) fig 5 Cl 23.2.1(e) fig 6

SN	Calculation	Reference
9.	**Check for development length** $L_d = \frac{\Phi * \sigma s}{4 * \tau bd}$ = (10×435)/ (4×1.4×1.6)) = 485.491 mm Nominal dia. of bar (Φ) = 10 Stress in bar (σ_s) = 0.87×f_y = 0.87×500 = 435 Design bond stress (τ_{bd}) = 1.4 M_l = 0.87×500×392.699×100×[1-{392.699×500)/ (1000×100×25)}] = 15.741 kN-m let L_o = 0 (1.3M_l)/V +L_o = [(1.3×15.741×10^6)/ (24.396×10^3)]+0 = 838.797 mm Since, L_d < [(1.3M_l)/V +L_o] Ok	IS 456:2000 CL 26.2.1 ANNEX G G-1.1 (b) CL 26.2.3.3
10.	**Torsional Reinforcement at corners** Size of mesh = lx/5 = 609.6 mm Size of wall = 350 mm Provide mesh of size = 609.6+350 = 959.696 Area of torsional reinforcement = 3/4×$A_{st\ req}$ = 3/4×150.434 = 112.826 mm^2 Use 8 mm diameter bar Spacing = 1000×(π×8^2/4)/ 112.826 = 445.5 mm So, provide 8 mm diameter bar @ c/c spacing in discontinuous edge. And provide 10 mm diameter bar @ 200 c/c spacing in continuous edge.	
11.	**Reinforcement in edge strip** Provide edge strip of 0.12% of gross area. = 0.12%×1000×120 = 144 mm^2 Provide 8 mm diameter bar = (π×8^2/4)×1000/144= 349.056≈340 mm	

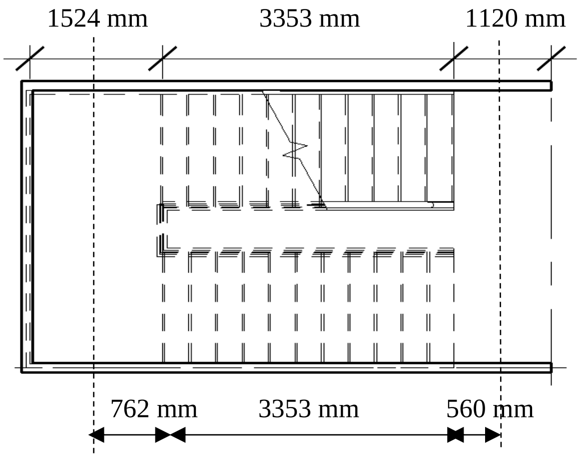
SN	Calculation	Reference
12.	**Design summary** Effective Depth (d)= 100 mm Clear cover = 15 mm Overall depth (D) = 120 mm Main bar Reinforcement (at short span) = 10 mm diameter bar @ 200 c/c Distribution bar Reinforcement (at long span) = 10 mm diameter bar @ 200 c/c For edge strip: Provide 8 mm diameter bar @ 340 mm at length of l/8.	

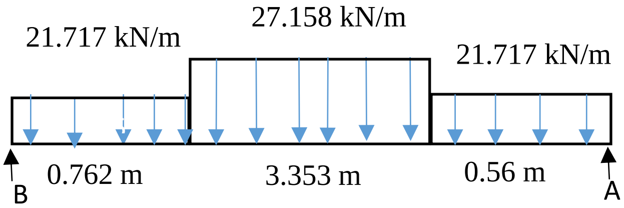
8.5 Design of Staircase

The staircase is designed as a longitudinal staircase and modelled as a slab supported simply at the first riser and the end of landing.

Table 8-11: Design of Staircase

SN	Calculation	Reference
1.	**Assumptions:** Slab thickness (D) = 160 mm Tread (T) = 12" = 304.8 mm Riser (R) = 6" = 152.4 mm Floor height = 12' = 3.6576 m Number of riser in a flight = (3.6576×1000/152.4)/2 = 12 Number of treads = No. of riser-1 = 12-1 = 11 nos Length of riser (L) = $\sqrt{(0.3048^2+0.1524^2)}$ = 0.341 m Width of landing (W) = 1.524 m Length of front landing (L_1) = 1.120 m Length of flight (L_2) = 3.353 m Length of end landing (L_3) = 1.524 m	

SN	Calculation	Reference
	Effective span (L) = $L_{2}+\{(L_1+L_3)/2\}$= 3.353+(1.120+1.524)/2 = 4.675 mm Let's provide 20 mm cover and 20⌀ bars.	Cl 33.1 (b)
		
2.	**Load Calculation** Specific weight of concrete (γ) = 25 kN/m^3 **Dead load:** Dead load of inclined fight per unit length = (0.5×R×T× W×γ + D×L×W× γ) /T = (0.5×0.1524×0.3048×1.524×25+0.16×0.341×1.524×25)/0.3048 = 9.723 kN/m Dead load of landing slab per unit length = D×γ×W = 0.160×25×1.524= 6.096 kN/m Floor finish per area (FF)= 1.5 kN/m^2	
	Floor finish = W×FF =1.524×1.5= 2.286 kN/m	IS:875 Part 1
	Live load: Live load per area (LL)= 4 kN/m^2	
	Live load =W× LL = 1.524×4= 6.096 kN/m	IS:875 Part 2
	Total load: Total load on landing = 6.096+2.286+6.096= 14.478 kN/m Factored load on landing = 1.5×14.478 = 21.717 kN/m Total load on flight = 9.723+2.286+6.096= 18.105 kN/m Factored load on flight =1.5×18.105 = 27.158 kN/m	

SN	Calculation	Reference
3.	**Analysis**	
		
	Reaction at support A (R_A). Taking moment about B	
	$\Sigma M_B = 0$	
	R_A=[21.717×0.56×{(0.5×0.56)+3.353+0.762}+27.158×3.353×{(0.5×3.353)+0.762}+21.717×0.762×(0.5×0.762)]/(0.56+3.353+0.762)	
	R_A = 60.279 kN	
	Now,	
	R_B = (21.717×0.56+27.158×3.353+21.717×0.762)-60.279	
	R_B = 59.491 kN	
	Shear force at 0.56 m from A=58.826-21.146×0.56= 46.984 kN	
	Shear force at 3.913 m from A=46.984-26.519×3.353= -41.934 kN	
	By interpolation, for max moment consider point of zero shear from A is at distance x = 2.332 m	
	Max Bending Moment at x (M_x) = (60.279×2.332-21.717×0.56×(2.332-0.56×0.5)-27.158×(2.332-0.56)×(2.332-0.56)/2)	
	= 72.978 kN-m	
	We know,	
	$M_{u,lim} = \frac{0.36x_{u,max}}{d}\left(1 - 0.42\frac{x_{u,max}}{d}\right) b\, d^2 f_{ck}$	ANNEX G.1
	Required effective depth (d)	
	$d=\sqrt{\frac{M_{max}}{0.133\times f_{ck}\times b}} = \sqrt{\frac{72.978\times 10^6}{0.133\times 25\times 1.524\times 1000}}$=120.007 mm	
	Required depth = d+20+Φ/2 = 120.007+20+20/2=150.007 mm	
	Provided depth (D) = 160 mm	
	Effective depth (d) = 160-20-20/2 = 130 mm	
	Design of Main reinforcement: $M_x=0.87\cdot f_y\cdot A_{st}\cdot d\cdot\left(1-\frac{f_y\cdot A_{st}}{b\cdot d\cdot f_{ck}}\right)$	IS 456:2000 ANNEX G.1

SN	Calculation	Reference
	$72.978\times10^6=0.87\times500\times A_{st}\times130\times\left(1-\frac{500\times A_{st}}{1524\times130\times25}\right)$ On solving, Area of rebar required (A_{st}) = 1525.395 mm^2 Size of bar provided (Φ) = 20 mm Area of 20 mm bar = $\pi\times20^2/4$ = 314.159 mm^2 Spacing required (s_{req}) = 314.159×1524/1481.469=323.178 mm Let's provide spacing of 130 mm. Area of rebar provided (A_{st}) = 1524×314.159/130= 3682.910 mm^2	
4.	**Check for Shear** Percentage of tensile steel(P_t)=$\frac{100\cdot A_{st_{Provided}}}{b\cdot d}$ $P_t=\frac{100\times3682.910}{1.524\times1000\times(160-20)}$=1.726% Shear strength of Concrete (τ_c) = 0.776 N/mm^2 Shear strength coefficient (k) = 1.28 , for overall depth of 160mm Shear strength of Concrete (τ_c') = 0.776×1.28 = 0.993 Maximum shear strength of concrete ($\tau_{c,max}$) = 3.1 Nominal shear stress (τ_v) = V_u/ (b×d) = 60.279×1000/(1.524×1000×130) = 0.304 N/mm^2 Since, $\tau_v < \tau_c' < \tau_{c,max}$ Design is safe in shear and no need for shear reinforcement.	 Table 19 CL 40.2.1.1 Table 20
5.	**Distribution bars:** For waist slab, provide nominal reinforcement $A_{st\ min}$= 0.12% of bD=0.0012×1.524×1000×160 = 292.608 mm^2 Provide 8 mm diameter bar of area = $\pi\times8^2/4$ = 50.265 mm Spacing required =50.265×1524/292.608 = 261.797 mm Spacing provided (s) = 250 mm Area of rebar provided (A_{st}) =50.265×1524/250= 306.415 mm^2	 CL 26.5.2.1
6.	**Check for deflection** $f_s = 0.58\cdot f_y\cdot A_{s_{t,re}}/A_{st}$	Cl.23.2

SN	Calculation	Reference
	Basis value of span to effective depth (L/d) ratio (α) = 26 Modification factor for span >10 m (β) = 1 Modification factor for tension reinforcement (γ) = 1.39 Allowable span to effective depth ratio (L/d) = 26×1×1.39 = 36.14 $(L/d)_{calculated}$ = 4.675×1000/ 130 = 35.596 Since calculated ratio is less than allowable ratio, the design is safe from deflection criteria.	Cl 23.2.1 (a) Cl 23.2.1 (b) From Fig.4
7.	**Development length** Design bond stress(τ_{bd}) = 1.4 Stress in bar (σ_s) = 0.87×f_y = 0.87×500 = 435 $L_d = \frac{\Phi * \sigma s}{4 * \tau bd}$ L_d =20×0.87×500/ (4×1.6×1.4) = 970.982 mm ≈970 mm	 CL 26.2.1.1
8.	**Summary** Provide 160 mm thick slab with 20 mm diameter rebar @ 130 mm c/c with development length of 970 mm and 8 mm diameter distribution rebar @ 250 mm c/c bar.	

8.6 Design of Shear Wall

The term shear wall refers to a wall that opposes lateral wind or earthquake loads acting parallel to the plane of the wall in addition to the gravity loads from the floors and roof adjacent to the wall. However, in this project we have dealt the lift wall as the isolated structure of shear wall so that it only bears its own load. They are designed as per the procedure given in IS-456:2000. According to IS-13920:2016, clause 9.1.2, the minimum thickness of the shear wall should be 150 mm. As per IS-456:2000, clause 32.2.2, the design of wall shall take account of the actual eccentricity of the vertical force subject to a minimum of 0.05t.

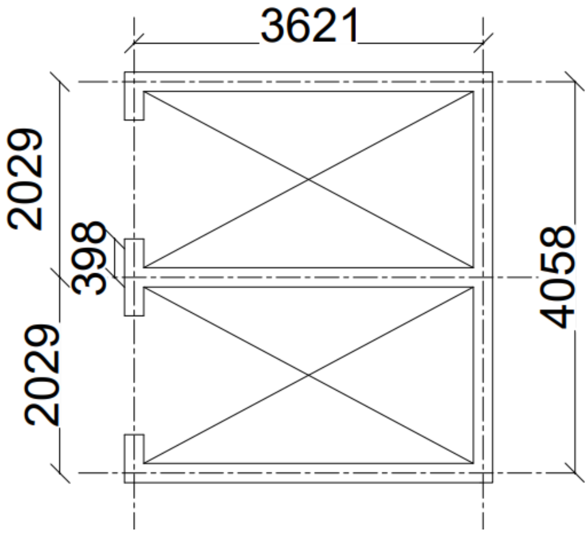


Figure 8-1: Dimension of Lift

Table 8-12: Design of Shear Wall

SN	Calculation	Reference
1.	**Material Properties** Grade of Concrete (f_{ck}) = 30 N/mm^2 Grade of Steel (f_y)= 500 N/mm^2 Unit Weight of Concrete (γ) = 25 kN/m^3	
2.	**Dimension of Lift.** Preliminary Thickness of Wall = 200 mm Length (Exterior edges) = 3621+200=3821 mm Breadth (Exterior edges) = 2029+200/2=2129 mm Smaller Breadth (Exterior edges) = 398+200/2=498 mm No. of Lift Cores = 2 nos Minimum Preliminary Thickness of Wall = 150 mm 200 mm>150 mm, Ok Floor Height (H) = 3658 mm No. of Storey = 3 Total length of wall=3821×3+2129×2+498×4=17713mm Live Load for Machine (LL) = 15 kN/m^2 Width of shear wall in x-direction = 3621+200 = 3821 mm	IS 13920:2016 Cl 10.1.2

SN	Calculation	Reference
	y-direction = 2029×2+200 = 4258 mm	
3.	**Check for Slenderness ratio** Effective height of wall h_e (0.75H)=0.75×3658 =2743.5 mm Slenderness ratio (h_e/t) = 2743.5/200= 13.718 < 30, OK Minimum eccentricity (e)= 0.05t = 10 mm Additional Eccentricity (e_a)= h_e/ (2500t) = 0.0055 mm	 IS 456:2000 Cl 32.2.2 IS 456:2000 Cl 32.2.5
4.	**Calculation of Load** 1. Base floor Length (Total Length) = 17.713 m Height(H_1) =H/2 = 3.658/2= 1.829 m Load(γ×TL×H_1×t)=25×17.713×1.829×0.2=161.985 kN 2. Typical Floor Length (Total Length) = 17.713 m Height (H_2)= H/2+H/2 = 3.658 m Load(γ×TL×H_2×t)=25×17.713×3.658×0.2=323.971 kN 3. Top Floor Length (Total Length) = 17.713 m Height(H_1) =H/2 = 3.658/2= 1.829 m Load(γ×TL×H_1×t)=25×17.713×1.829×0.2=161.985 kN Total Load = 161.985 +323.971+161.985 = 647.941 kN Total live load(LL×area)=15×3.621×2×2.029=220.41 kN	
5.	**Calculation of Base Shear** Total seismic weight(W)=647.941+0.3×220.41 =714.064 kN Total Height (H) = 3×3658/1000 = 10.974 m Seismic Zone Factor (Z) = 0.3 For Pokhara Importance Factor (I) = 1.5 For Moment Resisting Concrete Frames, K_t = 0.075	 NBC 105:2020 Cl 5.2 NBC 105:2020 Cl 4.1.5

SN	Calculation	Reference
	Time Period (T) = 1.25 $k_t H^{3/4}$= 0.565 sec	NBC 105:2020 Cl 5.1.2
	Building Height Exponent (k) = 1.033	NBC 105:2020 Cl 6.3
	Soil Type = C	
	Spectral Shape Factor (Ch (T)) = 2.5	Clause 4.1.2 (Case 2)
	Elastic Site Spectra ($C(T_1)$) = 1.125	Clause 4.1.1
	Ductility Factor (R_u) = 3	NBC 105:2020 Cl 5.3
	Over Strength Factor (ULS) Ω_u = 1.3	NBC 105:2020 Cl 5.4
	Over Strength Factor (SLS) = 1.15	NBC 105:2020 Cl 5.4
	Base shear Coefficient$C_d(T_1) = \frac{C(T_1)}{R_u \cdot \Omega_u}$ = 0.288 (For ULS)	
	Base Shear(V) = C_d (T_1)×W=0.288×714.064= 205.65 kN	
	Vertical Distribution of Forces (Fi)	NBC 105:2020 Cl 6.3
	$F_i = \frac{W_1 h_i^k}{\sum_{i=1}^{n} W_i h_i^k} V$	

Story	W_i	H_i	$W_i H_i^K$	Fx=Fy=F
Story 3	161.985	3.658	632.294	51.412
Story2	323.971	3.658	1264.592	102.825
Story 1	161.985	3.658	632.294	51.412
		Total	2529.18	205.65

SN	Calculation	Reference
	Moment at base (M_{base})	
	=51.412×3.658+102.825×3.658×2+51.412×3.658×3	
	= 1504.528 kN-m	
	The design axial strength of wall	IS 456:2000 Cl 32.2.5
	P_{uw} = 0.3(t-1.2e-2e_a)f_{ck} = 0.3×(200-1.2×10-2×0.0054)×30	
	= 1691.901 N/mm	
6	**Calculation of main vertical reinforcement**	
	Let, Clear cover = 20 mm and Diameter of bars = 12 mm	
	Effective cover (d') =20+12/2 = 26	
	a) When lateral load is acting along X direction	
	Width of shear wall in x-direction (D)= 3821 mm	
	Factored M_{ux} =1.5×M_{base}=1.5×1504.528= 2256.792 kNm	

SN	Calculation	Reference
	Factored V_{ux} = 1.5×V =1.5×205.65 = 308.475 kN Factored P_{ux} = 1.5×W = 1.5×714.064 = 1071.096 kN d'/D = 26/3812 = 0.007 $M_u/(f_{ck}\times b\times D^2) = 2256.792\times10^6/(30\times200\times3821^2)=0.026$ $P_u/(f_{ck}\times b\times D) = 1071.096\times10^3/(30\times200\times3821)=0.047$ From SP 16 chart-35, for above values, p/f_{ck} = 0.01 p = 0.01×25=0.25% A_{st} = 0.25%×3821×200=1910.5 mm^2`	
	Minimum reinforcement A_{st} (min) = 0.0025 of gross area = 0.0025×200×3821 = 1910.5 mm^2 Since A_{st}<$A_{st\ (min)}$, $A_{st,provided}$ = 1910.5 mm^2	NBC 105:2020 Cl 5.1(4)
	The largest diameter of longitudinal steel bars used in any part of a wall shall not exceed 1/10th of the thickness of that part. i.e maximum permissible diameter = 200/10 = 20 mm Providing bar of diameter 16 mm Area of a 16 mm dia. bar = $\pi\times16^2/4$ = 201.056 mm^2 No. of bars = 1910.5/201.056 = 9.502 ≈ 10 Spacing, S_v = 3821×201.056/1910.5 = 402.112 mm But, according to NBC 105:2020, Cl 5.1 (9) Spacing should be less than following a. 1/5th of the horizontal length l_w of wall = 1/5×3821 = 764.2 mm b. 3 times the thickness t_w of the web of wall = 3×200 = 600 mm c. 450 mm Hence, adopt 16 mm diameter bars @300 mm c/c on both faces of wall. **b) When lateral load is acting along Y direction** Width of shear wall in y-direction (D) = 4258 mm	NBC 105:2020 Cl 5.1(8)

SN	Calculation	Reference
	Factored M_{ux} = 1.5×1504.528 /3 = 752.264 kN-m Factored V_{ux} = 1.5×205.65/3 = 102.825 kN Factored P_{ux} = 1.5×714.064 /3 = 357.032 kN d'/D = 26/4258 = 0.006 $M_u/(f_{ck}\times b\times D^2) = 752.264\times10^6/(30\times200\times4258^2) = 0.007$ $P_u/(f_{ck}\times b\times D) = 357.032\times10^3/(30\times200\times4258) = 0.014$ From SP 16 chart-35, for above values, p/f_{ck} = 0.01 p = 0.01×25=0.25% A_{st} =0.25%×4258×200= 2129 mm^2` Minimum reinforcement A_{st} (min) = 0.0025 of gross area = 0.0025×200×4258 = 2129 mm^2 Since $A_{st}<A_{st\ (min)}$ $A_{st\ provided}$ = 2129 mm^2 The largest diameter of longitudinal steel bars used in any part of a wall shall not exceed 1/10th of the thickness of that part. i.e maximum permissible diameter = 200/10 = 20 Providing bar of diameter 16 mm Area of a 16 mm dia. bar = $\pi\times16^2/4$ = 201.056 mm^2 No. of bars = 2129/201.056 = 10.589≈11 Spacing, S_v = 4258×201.056/2129 = 402.112 mm But, according to NBC 105:2020, Cl 5.1, (9) Spacing should be less than following a. 1/5th of the horizontal length l_w of wall = 1/5×4258 = 851.6 mm b. 3 times the thickness t_w of the web of wall = 3×200 = 600 mm c. 450 mm Hence, adopt 16 mm diameter bars @300 mm c/c on both faces of wall.	NBC 105:2020 Cl 5.1(4) NBC 105:2020 Cl 5.1(8)

SN	Calculation	Reference
7	**Calculation of horizontal reinforcement:** Minimum reinforcement $A_{st\ (min)}$ = 0.0025 of gross area = 0.0025×200×10974 = 5487 mm^2 The largest diameter of longitudinal steel bars used in any part of a wall shall not exceed 1/10th of the thickness of that part. i.e maximum permissible diameter = 200/10 = 20 Providing bar of diameter 16 mm Area of a 16 mm dia. bar = $\pi \times 16^2/4$ = 201.056 mm^2 Spacing, S_v = 3821×201.056/5487 = 140.01 mm According to NBC 105:2020, Cl 5.1 (9) Spacing should be less than following a. 1/5th of the horizontal length l_w of wall = 1/5×3821 = 764.2 mm b. 3 times the thickness t_w of the web of wall = 3×200 = 600 mm c. 450 m Hence, adopt 16 mm diameter bars @120 mm c/c on both faces of wall.	NBC 105:2020 Cl 5.1(4) NBC 105:2020 Cl 5.1(8)
8	**Design from shear force criteria** According to NBC 105:2020, Cl 5.2, (1) Nominal shear stress $(\tau_v) = \frac{V_u}{t_\omega d_\omega}$ where, Thickness of the web (t_w) = 200 mm Factored shear force (V_u) = 1.5×205.65 = 308.475 kN Effective depth of wall section (along the length of the wall) (d_w) = 3821 mm τ_v = 308.475×1000/ (200×3821) = 0.404 N/mm^2 For design shear strength of concrete (τ_c) P_t = 100×1910.5/ (200×3821) = 0.25 $\tau_c = \frac{0.85\sqrt{(0.8f_{ck})}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$=0.37 N/mm^2	NBC 105:2020 Cl 5.2(2)

SN	Calculation	Reference
	$\beta = \frac{0.8f_{ck}}{6.89p_t} = \frac{0.8\times30}{6.89\times0.25} = 13.93 \text{ or } 1, \text{ greater value}$ And, Maximum design shear strength($\tau_{c,max}$)= $0.62\sqrt{f_{ck}} = 0.62\sqrt{30} = 3.396$ N/mm^2 Since, $\tau_v<\tau_{cmax}$ Design is safe in shear. Since, $\tau_v>\tau_c$ provide shear reinforcement. Providing 2 legged 16 mm dia bar. Area of a 16 mm dia. bar $A_{sv}=2\times\pi\times16^2/4$ = 402.123 mm^2 $S_{v,req}$=0.87×30×402.123×3821/{(0.404-0.37) ×200×3821}= 1543.4 mm But, according to NBC 105:2020, Cl 5.1 (9) Spacing should be less than following a. 1/5th of the horizontal length l_w of wall = 1/5×3821 = 764.2 mm b. 3 times the thickness t_w of the web of wall = 3×200 = 600 mm c. 450 mm Provide 300 mm spacing. $P_{t,provided}$=100×3821×402.123/(300×200×3821) =0.67% >0.25% Hence, adopt 2 legged 16 mm dia bars @300 mm c/c in horizontal direction	NBC 105:2020 Cl 5.2(3) IS 456:2000 Cl 40.4 NBC 105:2020 Cl 5.1(4)
9	**Boundary Element** L_w = 3821 mm and t_w = 200 mm A_g = 3821×200 = 764200 mm^2 $I = t_wL_w^3/12 = 200\times3821^3/12 = 92.978\times10^{10}$ mm^4 $f_c = P_u/A_g + M_u(L_w/2)/I_y$ = (1071.096×10^3/764200)+ {2256.792×10^6×(3821/2)/(92.987×10^{10})}=6.039 ≈6 MPa $0.2f_{ck}$ = 0.2×30 = 6 MPa Since, $f_c < 0.2f_{ck}$ Design for Boundary Element is not required.	IS 13920:2016 CL 10.4.1

8.7 Design of Footing

Foundation are structural elements that transfer load from the building or individual column to the earth below. If these loads are to be transmitted properly, foundations should be designed to prevent excessive settlement and rotation, to minimize differential settlement and to provide adequate safety against sliding and overturning. There are many types of foundation: Isolated footing, Strip foundation, Mat foundation, Pile foundation and Well foundation.

Selection of type of foundation depends upon the factors like type of building, soil strata, ground water table, economy, type of load and permissible differential settlement.

The footings of wall or column that individually transfers the load from superstructure to the soil is known as isolated footing. An isolated footing may be of masonry, plain concrete or RCC but here we design for RCC isolated footing.

Design step:

i. Calculate the area of footing.

ii. Area of footing = Total load / Allowable Bearing Pressure of soil

   Where, Total load = Service Load + Self weight of footing

iii. Provide size of footing to satisfy required area.

iv. Calculate Net upward pressure as:

   Net upward pressure = Factored load / Area of footing provided

v. Maximum occurring bending moment in the footing is calculated according to the clause 34.2.3.1 and 34.2.3.2 of IS code 456:2000.

vi. Depth of footing is calculated as per IS code 456:2000 ANNEX G-1.1 (c):

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

vii. The calculated d is increased 1.5 to 2 times for shear consideration.

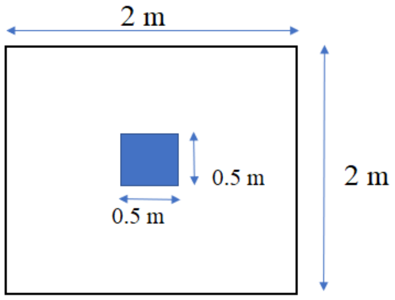
viii. Calculation of reinforcement is done as per IS code 456:2000 ANNEX G-1.1 (b):

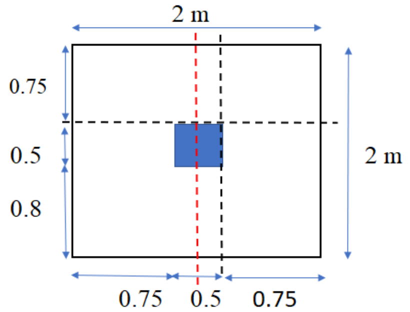
$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot d \cdot \left(\frac{1 - Ast \cdot fy}{b \cdot df_{ck}}\right)$$

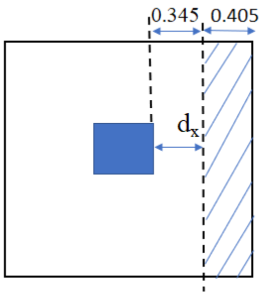
ix. Necessary checks are applied to check for one way shear and two-way shear.

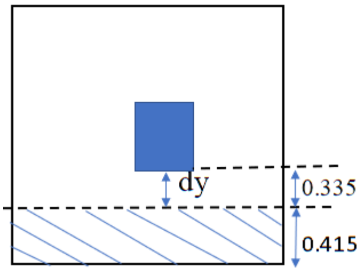
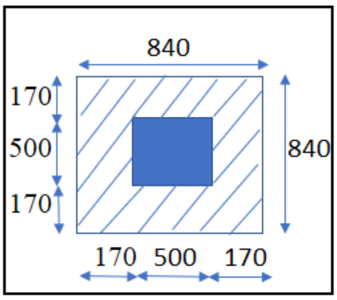
Sample design calculation for footing for column at grid C-4 is as below:

Table 8-13: Design of Isolated Footing

S.N	Description	Remarks
1.	**Basic data**  Size of column = a_1 = a_2 = 500 mm = 0.5 m Characteristic Load (P) = 439.5 kN Allowable Bearing Pressure of soil (ABP) = 150 kN/m^2 Grade of concrete (f_{ck}) = 25 N/mm^2 Grade of steel (f_y) = 500 N/mm^2 Clear cover (c) = 50 mm Self-weight of footing = 0.1×P = 0.1×439.5= 43.95 kN	
2.	**Size of footing** Total load (including self-wt.) = 439.5+43.95 = 483.45 kN Area of footing required(A)=Total load/ABP =483.45/150= 3.224 m^2 Dimensions of foundation required $\sqrt{A}$ = $\sqrt{3.224}$ = 1.795 m Dimensions of foundation provided (Adopting L/B = 1) X = 2 m Y = 2 m Area of footing provided = 2×2 = 4 m^2	
3.	**Net upward pressure** BCS = 1.5×439.5/2^2 = 164.813 kN/m^2	

S.N	Description	Remarks
4.	**Moment steel**	
		
	Cantilever length || to X(L_X) = X/2-a_1/2 = (2-0.5)/2= 0.75 m	
	Cantilever length || to Y(L_Y) = Y/2-a_2/2 = (2-0.5)/2= 0.75 m	
	UDL on the cantilever length L_1(w_X) = BCS×X	
	=164.813×2=329.626 kN/m	
	UDL on the cantilever length L_2(w_Y) = BCS×Y	
	=164.813×2=329.626 kN/m	
	Design BM along L_1(M_{ux}) = w_X×L_X^2/2 = 92.707 kNm	
	Design BM along L_2(M_{uy}) = w_Y×L_Y^2/2 = 92.707 kNm	
	Design BM(M_u) = 92.707 kNm	
	$M_{u,lim} = \frac{0.36x_{u,max}}{d}\left(1-0.42\frac{x_{u,max}}{d}\right) b\, d^2 f_{ck}$	IS 456:2000
	Required depth of footing (d_{req}) = [92.707×10^6/ {0.36×0.46(1-0.42×0.46)×2×1000×25}]$^{1/2}$ = 117.803 mm	ANNEX G-1.1 c
	Provided overall depth of footing (D) = 400 mm	
	Assume Diameter of bar (φ) = 10 mm	
	Effective depth provided	
	d_x = D-c-φ/2 = 400-50-10/2 = 345 mm	
	d_y = d_x-φ = 345-10 = 335 mm	
i	**R/F along X**	
	Bending Moment acting (M_{ux}) = 92.707 kNm	
	We know, $M_{ux} = 0.87 \cdot f_y \cdot A_{st} \cdot d_x \cdot \left(1-\frac{f_y \cdot A_{st}}{X \cdot d_x f_{ck}}\right)$	IS 456:2000 ANNEX G-1.1b
	$92.707\times10^6 = 0.87\times500\times A_{st}\times345\times\left(1-\frac{500\times A_{st}}{2\times1000\times345\times25}\right)$	
	On solving, Area of steel required ($A_{st,req}$) = 629.213 mm^2	

S.N	Description	Remarks
	Let's provide diameter of bar (φ) = 10 mm Area of 10 mm bar = $\pi \times 10^2/4$ = 78.540 mm^2 Spacing required (S_{req}) = 78.540×2000/629.213 = 249.645 mm Let's provide spacing of 160 mm Area of steel provided ($A_{st,prov}$) = 78.540×2000/160= 981.75 mm^2 $P_{t,prov.}$= 100×981.75/(345×2000) = 0.142 >0.12% Provide 10 mm @ 160 mm c/c	 Cl 34.5.1
ii	**R/F along Y** Bending Moment acting (M_{uy}) = 92.707 kNm We know, $M_{uy} = 0.87 \cdot f_y \cdot A_{st} \cdot d_y \cdot \left(1 - \frac{f_y \cdot A_{st}}{Y \cdot d_y f_{ck}}\right)$ $92.707\times10^6 = 0.87\times500\times A_{st} \times 335\times\left(1-\frac{500\times A_{st}}{2\times1000\times335\times25}\right)$ On solving, area of steel required ($A_{st,req}$) = 648.741 mm^2 Let's provide diameter of bar (φ) = 10 mm Area of 10 mm bar = $\pi \times 10^2/4$ = 78.540 mm^2 Spacing required (S_{req}) = 78.540×2000/648.741 = 242.131 mm Let's provide spacing of 160 mm Area of steel provided ($A_{st,prov}$) = 78.540×2000/160= 981.75 mm^2 $P_{t,prov.}$= 100×981.75/(335×2000) = 0.147 >0.12% Provide 10 mm @ 160 mm c/c	 IS 456:2000 ANNEX G-1.1 b
5. i	**Check for one-way shear** **Along X**  $V_u = BCS \cdot X \cdot \left(\frac{Y-a_1}{2} - d_x\right)$ = 164.813×2000× [{(2000-500)/2}-345]=133.499 kN	

S.N	Description	Remarks
	Nominal Shear Stress (τ_v) = Vu/ ($X \times d_x$) = 0.193 N/mm^2	CL 40.1
	For $100A_{st}/bd$ = 0.142% and M25 grade	Table 19
	Design shear strength of concrete (τ_c) = 0.29 N/mm^2	
	k= 1 for d $\geq$ 300 mm	Cl 40.2.1.1
	$\tau_c' = k \times \tau_c = 1 \times 0.29 = 0.29$ N/mm^2	
	$\tau_v < \tau_c'$, hence safe.	
ii	**Along Y**	
		
	$V_u = BCS \cdot Y \cdot \left(\frac{X-a_2}{2} - d_y\right)$	
	= 164.813×2000× [{(2000-500)/2}-335]=136.795 kN	
	Nominal Shear Stress (τ_v) = Vu/ ($Y \times d_y$) = 0.204 N/mm^2	
	For $100A_{st}/bd$ = 0.142% and M25 grade	Table 19
	Design shear strength of concrete (τ_c) = 0.29 N/mm^2	
	k= 1 for d $\geq$ 300 mm	Cl 40.2.1.1
	$\tau_c' = k \times \tau_c = 1 \times 0.29 = 0.29$ N/mm^2	
	$\tau_v < \tau_c'$, hence safe.	
6.	**Check for two-way shear**	
		
	Average depth(d) = $(d_x+d_y)/2$ = (345+335)/2= 340 mm	
	Width at distance d/2 along X(b''_X) = a_1+d = 500+340 = 840 mm	
	Width at distance d/2 along Y(b''_Y) = a_2+d = 500+340 = 840 mm	

S.N	Description	Remarks
	Critical perimeter (b_o) = 2×(b''_X+b''_Y) = 2×(840+840)=3360 mm Upward Pressure (w) = 164.813 kN/m^2 Total Area of foundation (A) = 4 m^2 Critical area for shear (a) = 840×840/10^6 = 0.7056 m^2 Factored shear force (V''_u) = w×(A-a) = 164.813× (4-0.7056) = 542.960 kN Shear Stress (τ_v) = 542.960×1000/ (3360×345) = 0.468 N/mm^2 Permissible Shear Stress (τ'_c) = K_s×τ_c = 1×1.25 N/mm^2 where, $k_s=(0.5+\beta_c)\leq 1$ = (0.5+500/500) = 1.5 ≤1 β_C is the ratio of short side to long side of column. τ_c= $0.25\sqrt{f_{ck}}$ = $0.25\times\sqrt{25}$ =1.25 N/mm^2 Since τ_v < τ_c', footing is safe in two-way shear.	• CL 31.6.3.1
7.	**Check for development length** Design bond stress(τ_{bd}) = 1.4 $L_{d,req} = \frac{\Phi*\sigma s}{4*\tau bd}$ =10×0.87×500/(4×1.6×1.4) = 485.491 mm $L_{d(available)}$ = {(Y-d_y)/2}-Cover = {(2000-335)/2}-50 = 782.5 mm > $L_{d,req}$ footing has sufficient development length	CL 26.2.1 CL 26.2.1.1
8.	**Design Summary** Dimension of footing = 2000 mm × 2000 mm Depth of footing= 400 mm Reinforcement in either direction= 10 mm @ 160 mm c/c bar.	

8.7.1 Justification For Selection of Isolated Footing

As shown in sample calculation above, footings for columns corresponding different load were designed. The columns are assigned to different types of footings as shown in Foundation Layout and Trench Plan (drawing sheet A111). Detail dimension and reinforcement of each footing are shown in drawing sheet A111 of ANNEX A.

Table 9-14 shows calculation of total footing area. The total footing area is 758.327 m^2 which is less than (50% of total plinth area of 2815.616 m^2), this fact further justifies the selection of isolated footing for foundation.

Table 8-14: Calculation of Total Footing Area

S.N	Type	Nos	L_x(m)	L_y(m)	Area(m^2)
1	F1	15	1.500	1.500	33.750
2	F2	4	1.750	1.750	12.250
3	F3	6	2.250	2.250	30.375
4	F4	1	3.000	3.000	9.000
5	F5	13	2.500	2.500	81.250
6	F6	7	2.000	2.000	28.000
7	F7	3	2.750	2.750	22.688
8	F8	1	4.250	4.250	18.063
9	F9	23	3.000	3.000	207.000
10	F10	11	3.250	3.250	116.188
11	F11	2	3.250	3.250	21.125
12	F12	2	3.750	3.750	28.125
13	F13	1	3.360	2.750	9.240
14	F14	1	3.858	3.840	14.815
15	F15	1	3.708	3.100	11.495
16	F16	1	3.610	3.000	10.830
17	F17	1	3.860	3.250	12.545
18	F18	1	3.000	3.590	10.770
19	F19	1	4.110	3.500	14.385
20	F20	1	4.860	4.250	20.655
21	F21	1	4.108	3.500	14.378
22	F22	1	3.610	3.000	10.830
23	F23	1	4.250	4.840	20.570
TOTAL FOOTING AREA					758.327 m^2
					<50% of plinth area, OK

9. ESTIMATION

Estimation is process of calculating the quantities and cost of various items of works of the construction project. Total estimated cost as well as total quantities of different items of works to complete the projects must be known before inviting tender. Without tender contractor cannot be selected and without hiring the contractor construction is not possible.

One of the important data required for estimating is rates. Per unit item rate is worked out through analysis of rate by considering cost of materials, labors, tools, plants and equipment, as well as contractors profits and overheads including taxes. Rate for each item was taken from Kaski District Rate 2079/80.

9.1 Rate Analysis

9.1.1 Site Clearance per 100 sq.m

Description of Item: Site Clearance including removal of top soils, removal of roots and other surface obstacles

Table 9-1: Rate Analysis for Site Clearance per 100 sq.m

S.N	Items	Unit	Qty	Wastage %	Total Qty	Total Rate	Amt	Rmk
1.	**Labor**							
	Foreman	hr	1	5	1.05	1000	1050	
	Helper	hr	1	5	1.05	700	735	
	Operator	hr	1	5	1.05	780	819	
	Hd Driver	hr	1	5	1.05	700	735	
					Sub-totals		**Rs 3339**	
2.	**Plant and Equipment @ 3% of labor cost**						**Rs 100.17**	
					Sub-totals		**Rs 3439.17**	
3.	**Miscellaneous**							
	Fuel for Equipment	lit/hr	50		50	80	4000	

S.N	Items	Unit	Qty	Wastage %	Total Qty	Total Rate	Amt	Rmk
	Fuel for Dumper	lit/hr	12.5		12.5	80	1000	
				Sub-total miscellaneous			**Rs 5000**	
						Grand Total	**Rs 11778.17**	

9.1.2 Earthwork in Excavation per 1 cu.m

Table 9-2: Rate Analysis Earthwork in Excavation per 1 cu.m

S.N	Description	Qty	Rate	Amt (Rs)	Remarks
1.	**Labors**				For additional depth 1.5 to 3 m, add 0.50 numbers of unskilled labor
	Unskilled	0.7 nos	790	553	
2.	**Equipment & Materials 3% of labor cost**			**Rs 16.6**	
	Sub-total			**Rs 570**	
3.	**Contactor's profit@ 15% of Sub-totals**			**Rs 93.7**	
	Totals			**Rs 663**	
4.	**Tax/Vat @13% of totals**			**Rs 93.4**	
	Grand totals			**Rs 663**	

9.1.3 Stone soiling per 1 cu.m

Table 9-3: Rate Analysis for Stone Soiling per 1 cu.m

S.N	Description	Qty	Rate	Amt (Rs)	Remarks
1.	**Materials**				
	Blockstone	1 m^3	2200	2200	
	Bond stone	0.2 m^3	1983	396.6	
	Stone dust	0.2 m^3	2310	462	
2.	**Labors**				
	Unskilled	1.5 nos	790	1185	
			Sub-total	**Rs 4243.6**	

S.N	Description	Qty	Rate	Amt (Rs)	Remarks
	Contractor's profit and overhead @ 15% sub total			**Rs 636.54**	
			Total	**Rs 4880.14**	
	Tax / Vat @ 13% of total			**Rs 634.418**	
			Grand Total	**Rs 5514.558**	

9.1.4 Brick Soiling per 1 cu.m

Table 9-4: Rate Analysis Brick Soiling per 1 cu.m

S.N	Description	Qty	Rate	Amt (Rs)	Remark
1.	**Materials**				
	Brick	1 m^3	11250	11250	
	Stone dust	0.2 m^3	2310	462	
2.	**Labors**				
	Unskilled	1.5 nos	790	1185	
			Sub-total	**Rs 12897**	
	Contractor's profit and overhead @ 15% sub-total			**Rs 1934.55**	
			Total	**Rs 14831.55**	
	Tax / Vat @ 13% of total			**Rs 1928.102**	
			Grand Total	**Rs 16759.652**	

9.1.5 1st Class B/W with 1:3 Cement Sand Mortar per 1 cu.m

Table 9-5: Rate Analysis of 1st class B/W with 1:3 Cement Sand Mortar per 1 cu.m

S.N	Description	Qty	Rate	Amt (Rs)	Remarks
1.	**Materials**				Nos. of labors same for other proportions of
	Bricks	560 nos.	17	9520	
	Cement	2.6 bags.	607.5	1579.5	
	Sand	0.27 m^3	3300	891	
	Water	104 liters	0.25	26	

S.N	Description	Qty	Rate	Amt (Rs)	Remarks
			Sub-total 1	**Rs 12016.5**	mortar. Add 0.70 no. of unskilled labors for additional story. Additional 0.20 no for ground floor.
2	**Labors**				
	Skilled	1.5 nos	1125	1687.5	
	Unskilled	2.2 nos	790	1738	
			Sub-total 2	**Rs 3425.5**	
3	**Scaffolding @ 3% of labor cost**			Rs 102.765	
			Sub total	**Rs 15544.765**	
4	**Contactor's Profit @15% of total**			Rs 2331.715	
			Totals	**Rs 17876.48**	
5	**Tax @13% of total**			**Rs 2323.942**	
			Grand Totals	**Rs 20200.422**	

9.1.6 P.C.C M10 (1:3:6) Work per 1 cu.m

Table 9-6: Rate Analysis of P.C.C M10 (1:3:6) Work per 1 cu.m

S.N.	Description	Qty	Rate	Amt (Rs)	Remarks
1.	**Materials**				
	Coarse Aggregates	0.924 m^3	3500	3234	
	Cement	4.435 bags	697.5	3093.413	
	Sand	0.462 m^3	3300	1524.6	
	Water	110.875 liters	0.25	27.719	
			Sub-total 1	**Rs 7879.732**	
2.	**Labor**				
	Skilled	1 no.	1125	1125	
	Unskilled	4 nos	790	3160	
			Sub-total 2	**Rs 4285**	
			Sub-totals	**Rs 12164.732**	
3.	**Contactor's Profit @15% of total**			**Rs 1824.71**	
			Totals	**Rs 13989.442**	
4.	**Tax @13% of total**			**Rs 1818.627**	
			Grand Total	**Rs 15808.069**	

9.1.7 P.C.C M15 (1:2:4) Work in Foundation per 1 cu.m

Table 9-7: Rate Analysis for P.C.C M15 (1:2:4) Work in Foundation per 1 cu.m

S.N.	Description	Qty	Rate	Amt (Rs)	Remarks
1.	**Materials**				
	Coarse Aggregates	0.88 m^3	3500	3080	
	Cement	6.34 bags	697.5	4422.15	
	Sand	0.444 m^3	3300	1465.2	
	Water	159 liters	0.25	39.75	
			Sub-total 1	**Rs 9007.1**	
2.	**Labors**				
	Skilled	1 no.	1125	1125	
	Unskilled	4 nos	790	3160	
			Sub-total 2	**Rs 4285**	
			Sub-totals	**Rs 13292.1**	
3.	**Contactor's Profit @15% of total**			**Rs 1993.815**	
			Totals	**Rs 15285.915**	
4.	**Tax @13%of total**			**Rs 1987.169**	
			Grand Total	**Rs 17273.084**	

9.1.8 P.C.C M25 (1:1:2) for R.C.C Work

Table 9-8: Rate Analysis for P.C.C M25 (1:1:2) for R.C.C Work

S.N.	Description	Qty	Rate	Amt (Rs)	Remarks
1.	**Materials**				
	Coarse Aggregates	0.77 m^3	3500	2695	
	Cement	11.088 bags	697.5	7733.88	
	Sand	0.385 m^3	3300	1270.5	
	Water	277.2 liters	0.25	69.3	
			Sub-total 1	**Rs 11768.68**	
2.	**Labor**				
	Skilled	0.8 nos	1125	900	

S.N.	Description	Qty	Rate	Amt (Rs)	Remarks
	Unskilled	7 nos	790	5530	
			Sub-total 2	**Rs 6430**	
			Sub-totals	**Rs 18198.68**	
3.	**Contactor's Profit @15% of total**			**Rs 2729.802**	
			Totals	**Rs 20928.482**	
4.	**Tax @13% of total**			**Rs 2720.703**	
			Grand Total	**Rs 23649.185**	

9.1.9 P.C.C M30 (1:0.75:1.5) for Shear Wall

Table 9-9: Rate Analysis for P.C.C M30 (1:0.75:1.5) for Shear Wall

S.N.	Description	Qty	Rate	Amt (Rs)	Remarks
1.	**Materials**				
	Coarse Aggregates	0.705 m^3	3500	2467.5	
	Cement	13.536 bags	697.5	9441.36	
	Sand	0.353 m^3	3300	1163.25	
	Water	338.4 liters	0.25	84.6	
			Sub-total 1	**Rs 13156.71**	
2.	**Labor**				
	Skilled	0.8 nos	1125	900	
	Unskilled	7 nos	790	5530	
			Sub-total 2	**Rs 6430**	
			Sub-totals	**Rs 19586.71**	
3.	**Contactor's Profit @15% of total**			**Rs 2938.007**	
			Totals	**Rs 22524.717**	
4.	**Tax @13% of total**			**Rs 2928.213**	
			Grand Total	**Rs 25452.93**	

9.1.10 Plastering and Painting per 100 sq.m

Table 9-10: Rate Analysis for 12.5 mm Thick 1:4 Plastering per 100 sq.m

S.N.	Description	Qty	Rates	Amt (Rs)	Remarks
1.	**Materials**				
	a) cement	10.8 bags	607.5	6561.00	
	b) sand	1.5 m^3	2475	3712.5	
	c) water	432 liters	0.25	108	
			Sub-totals 1	**Rs 10381.5**	
2.	**Labors**				.
	Skilled	12 nos	1125	13500	increased 25% unskilled labor for ceiling
	Unskilled	16 nos	790	12640	
			Sub-totals 2	**Rs 26140**	
			Sub-totals	**Rs 36521.5**	
3.	**Contractor's profit and overheads @ 15% of subtotal**			**Rs 5478.225**	
			Totals	**Rs 41999.725**	
4.	**Tax/Vat @13% of total**			**Rs 5459.964**	
			Grand Totals	**Rs 47459.689**	

Table 9-11: Rate Analysis for 20 mm Thick 1:4 Plastering per 100 sq.m

S.N	Description	Qty	Rates	Amt (Rs)	Remarks
1.	**Materials**				
	a) cement	17.28 bags	607.5	10497.6	
	b) sand	2.4 m^3	2475	5940	
	c) water	691.2 liters	0.25	172.8	
			Sub-totals 1	**Rs 16610.4**	

S.N	Description	Qty	Rates	Amt (Rs)	Remarks
2.	**Labours**				increased 25% unskilled labour for ceiling.
	Skilled	14 nos	1125	15750	
	Unskilled	19 nos	790	15010	
			Sub-totals 2	**Rs 30760**	
			Sub-totals	**Rs 47370.4**	
3.	**Contractor's profit and overheads @ 15% of subtotal**			**Rs 7105.56**	
			Totals	**Rs 54475.96**	
4.	**Tax/Vat @ 13% of total**			**Rs 7081.875**	
			Grand Totals	**Rs 61557.835**	

Table 9-12: Rate Analysis for Painting per 100 sq.m Area

S.N	Description	Qty	Rate	Amt (Rs)	Remarks
1.	**First Coat**				
i.	**Materials**				
	a) white wash/ primer	12 liters	330	3960	
	b) Gum	0.48 liters	210	100.8	
			Sub-total	**Rs 4060.8**	
ii.	**Labor**				
	a) Skilled	0.8 nos	1125	900	increased 25% unskilled labor for ceiling.
	b) Unskilled	0.7 nos	790	553	
			Sub-total	**Rs 1453**	
2.	**Second Coat**				
i.	**Materials**				

S.N	Description	Qty	Rate	Amt (Rs)	Remarks
	a) Gum	0.88 liters	210	184.8	
	b) Enamel	22 liters	470	10340	
			Sub-total	**Rs 10524.8**	
ii.	**Labor**				
	a) Skilled	1.5 nos.	1125	1687.5	
	b) Unskilled	1 no.	790	790	
			Sub-total	**Rs 2477.5**	
3.	**Distemper**	8 kg	273	2184	
	Labor				
	a) Skilled	2 nos	1125	2250	
	b) Unskilled	2 nos	790	1580	
			Sub-total	**Rs 3830**	
4.	**Waterproof paint**				
i.	First coat	30 liters	637	19110	
	a) Skilled labor	1.7 nos	1125	1912.5	
	b) Unskilled labor	1.7 nos	790	1343	
			Sub-total	**Rs 3255.5**	
ii.	Second Coat	48.5 liters	637	30894.5	
	a) Skilled labor	5 nos	1125	5625	
	b) Unskilled labor	5 nos	790	3950	
			Sub-total	**Rs 9575**	
			Total	**Rs 87365.1**	

9.1.11 Steel Reinforcement per 1 MT

Table 9-13: Rate Analysis of Steel Reinforcement for 1 MT

S.N.	Description	Qty	Rates	Amt (Rs)	Remarks
1.	**Materials (adding 5% wastages)**				
	TMT bars 10 mm	1050 kg	92	96600	
	TMT bars 12 mm	1050 kg	92	96600	
	TMT bars 16 mm	1050 kg	92	96600	

	TMT bars 20 mm	1050 kg	92	96600	
	TMT bars 25 mm	1050 kg	92	96600	
	TMT bars 28 mm	1050 kg	94	98700	
	TMT bars 32 mm	1050 kg	94	98700	
	Binding wires	13 kg	111.75	1452.75	
			Sub-totals 1	**Rs 98652.75**	Taking average of TMT bars
2.	**Labors**				
	Skilled	12 nos	1125	13500	
	Unskilled	12 nos	790	9480	
			Sub-totals 2	**Rs 22980**	
			Sub-totals	**Rs 121632.75**	
3.	**Contractor's profit and overheads @ 15% of subtotal**			**Rs 18244.913**	
			Totals	**Rs 139877.663**	
4.	**Tax/Vat @13% of total**			**Rs 18184.096**	
			Grand Totals	**Rs 158061.76**	

9.2 Quantity Estimation of Civil Works

Project: ABC 100 BED HOSPITAL

Site: Pokhara-32, Kaski

Table 9-14: Quantity Estimation of Civil Works

SN	Description	No	L	B	H	Qty	Unit	Remark
A.	**Site clearance and Earthwork**							
	I. Site clearance work							
1.	**Site clearance work cutting & uprooting herbs & shrubs, topsoil cutting, removal waste all complete**	1	85.000	82.000		6970.000	m^2	
	II. Earth work							
2.	**Earth work in excavation in foundation in ordinary soil lead up to 30 m lift up to 1.50 m steaking the soil. 10 m away from foundation trenches as per specification & instruction of site engineer**							
	Ground Floor							
	Column Foundation F1	15	1.515	1.515	1.500	51.643	m^3	
	Column Foundation F2	4	1.765	1.765	1.500	18.691	m^3	
	Column Foundation F3	6	2.250	2.250	1.500	45.563	m^3	
	Column Foundation F4	1	3.015	3.015	1.500	13.635	m^3	
	Column Foundation F5	13	2.515	2.515	1.500	123.342	m^3	
	Column Foundation F6	7	2.015	2.015	1.500	42.632	m^3	
	Column Foundation F7	3	2.765	2.765	1.500	34.404	m^3	
	Column Foundation F8	1	4.265	4.265	2.500	45.476	m^3	
	Column Foundation F9	23	3.015	3.015	1.500	313.613	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
	Column Foundation F10	11	3.265	3.265	1.500	175.894	m^3	
	Column Foundation F11	2	3.265	3.265	2.500	53.301	m^3	
	Column Foundation F12	2	3.765	3.765	2.500	70.876	m^3	
	Column Foundation F13	1	3.375	2.765	1.500	13.998	m^3	
	Column Foundation F14	1	3.875	3.855	1.500	22.407	m^3	
	Column Foundation F15	1	3.725	3.115	1.500	17.405	m^3	
	Column Foundation F16	1	3.625	3.015	2.500	27.323	m^3	
	Column Foundation F17	1	18.860	3.265	2.500	153.945	m^3	
	Column Foundation F18	1	3.015	3.615	2.500	27.248	m^3	
	Column Foundation F19	1	4.125	3.515	2.500	36.248	m^3	
	Column Foundation F20	1	4.875	4.265	2.500	51.980	m^3	
	Column Foundation F21	1	4.115	3.515	2.500	36.161	m^3	
	Column Foundation F22	1	3.625	3.015	2.500	27.323	m^3	
	Column Foundation F23	1	4.265	4.855	2.500	51.766	m^3	
	Foundation wall From Foundation Beam to Ground Level							
	Along Grid 1-1,2-2,13-13,14-14	8	4.500	0.380	1.200	16.416	m^3	
	Along Grid 3-3	1	19.000	0.380	1.200	8.664	m^3	
	Along Grid 4-4	1	28.820	0.380	1.200	13.142	m^3	
	Along Grid 5-5	1	28.400	0.380	1.200	12.950	m^3	
	Along Grid 6-6	1	20.800	0.380	1.200	9.485	m^3	
	Along Grid 7-7	1	33.870	0.380	1.200	15.445	m^3	
	Along Grid 8-8	1	26.640	0.380	1.200	12.148	m^3	
	Along Grid 9-9	1	40.800	0.380	1.200	18.605	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
	Along Grid 10-10	1	22.350	0.380	1.200	10.192	m^3	
	Along Grid 11-11	1	20.400	0.380	1.200	9.302	m^3	
	Along Grid 12-12	1	21.850	0.380	1.200	9.964	m^3	
	Along Grid A-A, B-B, M-M, N-N	1	24.700	0.380	1.200	11.263	m^3	
	Along Grid C-C	1	21.410	0.380	1.200	9.763	m^3	
	Along Grid D-D	1	19.780	0.380	1.200	9.020	m^3	
	Along Grid E-E	1	15.090	0.380	1.200	6.881	m^3	
	Along Grid F-F	1	38.520	0.380	1.200	17.565	m^3	
	Along Grid G-G	1	34.410	0.380	1.200	15.691	m^3	
	Along Grid H-H	1	40.800	0.380	1.200	18.605	m^3	
	Along Grid I-I	1	26.580	0.380	1.200	12.120	m^3	
	Along Grid J-J	1	25.740	0.380	1.200	11.737	m^3	
	Along Grid K-K	1	4.520	0.380	1.200	2.061	m^3	
	Along Grid L-L	1	16.380	0.380	1.200	7.469	m^3	
				Total earth work in excavation		**1713.362**	m^3	
3	**Earth back filling in foundation trenches with compaction**							
	Back filling Quantity (50% of excavated)		1713.362×50%			856.681	m^3	
	Floor filling area	74	6.016	6.016	0.610	1633.718	m^3	
	Deduct P.C.C	74	6.016	6.016	0.075	200.867	m^3	
	Net earth back filling					**2289.532**	m^3	
	III. Concreting work							
4.	**Plain cement concrete M10 (1:3:6) with cement, sand & 10 to 38 mm stone ballast Proper mixing, laying &compaction, curing the Job all complete**							
	Column foundation							
	Column Foundation F1	15	1.515	1.515	0.075	2.582	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
	Column Foundation F2	4	1.765	1.765	0.075	0.935	m^3	
	Column Foundation F3	6	2.250	2.250	0.075	2.278	m^3	
	Column Foundation F4	1	3.015	3.015	0.075	0.682	m^3	
	Column Foundation F5	13	2.515	2.515	0.075	6.167	m^3	
	Column Foundation F6	7	2.015	2.015	0.075	2.132	m^3	
	Column Foundation F7	3	2.765	2.765	0.075	1.720	m^3	
	Column Foundation F8	1	4.265	4.265	0.075	1.364	m^3	
	Column Foundation F9	23	3.015	3.015	0.075	15.681	m^3	
	Column Foundation F10	11	3.265	3.265	0.075	8.795	m^3	
	Column Foundation F11	2	3.265	3.265	0.075	1.599	m^3	
	Column Foundation F12	2	3.765	3.765	0.075	2.126	m^3	
	Column Foundation F13	1	3.375	2.765	0.075	0.700	m^3	
	Column Foundation F14	1	3.875	3.855	0.075	1.120	m^3	
	Column Foundation F15	1	3.725	3.115	0.075	0.870	m^3	
	Column Foundation F16	1	3.625	3.015	0.075	0.820	m^3	
	Column Foundation F17	1	18.860	3.265	0.075	4.618	m^3	
	Column Foundation F18	1	3.015	3.615	0.075	0.817	m^3	
	Column Foundation F19	1	4.125	3.515	0.075	1.087	m^3	
	Column Foundation F20	1	4.875	4.265	0.075	1.559	m^3	
	Column Foundation F21	1	4.115	3.515	0.075	1.085	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
	Column Foundation F22	1	3.625	3.015	0.075	0.820	m^3	
	Column Foundation F23	1	4.265	4.855	0.075	1.553	m^3	
	Foundation Wall					0.000		
	Along Grid 1-1,2-2,13-13,14-14	8	4.500	0.500	0.075	1.350	m^3	
	Along Grid 3-3	1	19.000	0.500	0.075	0.713	m^3	
	Along Grid 4-4	1	28.820	0.500	0.075	1.081	m^3	
	Along Grid 5-5	1	28.400	0.500	0.075	1.065	m^3	
	Along Grid 6-6	1	20.800	0.500	0.075	0.780	m^3	
	Along Grid 7-7	1	33.870	0.500	0.075	1.270	m^3	
	Along Grid 8-8	1	26.640	0.500	0.075	0.999	m^3	
	Along Grid 9-9	1	40.800	0.500	0.075	1.530	m^3	
	Along Grid 10-10	1	22.350	0.500	0.075	0.838	m^3	
	Along Grid 11-11	1	20.400	0.500	0.075	0.765	m^3	
	Along Grid 12-12	1	21.850	0.500	0.075	0.819	m^3	
	Along Grid A-A, B-B, M-M, N-N	1	24.700	0.500	0.075	0.926	m^3	
	Along Grid C-C	1	21.410	0.500	0.075	0.803	m^3	
	Along Grid D-D	1	19.780	0.500	0.075	0.742	m^3	
	Along Grid E-E	1	15.090	0.500	0.075	0.566	m^3	
	Along Grid F-F	1	38.520	0.500	0.075	1.445	m^3	
	Along Grid G-G	1	34.410	0.500	0.075	1.290	m^3	
	Along Grid H-H	1	40.800	0.500	0.075	1.530	m^3	
	Along Grid I-I	1	26.580	0.500	0.075	0.997	m^3	
	Along Grid J-J	1	25.740	0.500	0.075	0.965	m^3	
	Along Grid K-K	1	4.520	0.500	0.075	0.170	m^3	
	Along Grid L-L	1	16.380	0.500	0.075	0.614	m^3	
	Flooring at Plinth Level							
	Area of Grids	74	6.016	6.016	0.075	200.867	m^3	
					Total	**283.235**	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
5	**Brick soling in foundation & level with packing sand in the gap as per specification & instruction of site engineer**							
	Column foundation							
	Column Foundation F1	15	1.515	1.515	0.075	2.582	m^3	
	Column Foundation F2	4	1.765	1.765	0.075	0.935	m^3	
	Column Foundation F3	6	2.250	2.250	0.075	2.278	m^3	
	Column Foundation F4	1	3.015	3.015	0.075	0.682	m^3	
	Column Foundation F5	13	2.515	2.515	0.075	6.167	m^3	
	Column Foundation F6	7	2.015	2.015	0.075	2.132	m^3	
	Column Foundation F7	3	2.765	2.765	0.075	1.720	m^3	
	Column Foundation F8	1	4.265	4.265	0.075	1.364	m^3	
	Column Foundation F9	23	3.015	3.015	0.075	15.681	m^3	
	Column Foundation F10	11	3.265	3.265	0.075	8.795	m^3	
	Column Foundation F11	2	3.265	3.265	0.075	1.599	m^3	
	Column Foundation F12	2	3.765	3.765	0.075	2.126	m^3	
	Column Foundation F13	1	3.375	2.765	0.075	0.700	m^3	
	Column Foundation F14	1	3.875	3.855	0.075	1.120	m^3	
	Column Foundation F15	1	3.725	3.115	0.075	0.870	m^3	
	Column Foundation F16	1	3.625	3.015	0.075	0.820	m^3	
	Column Foundation F17	1	18.860	3.265	0.075	4.618	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
	Column Foundation F18	1	3.015	3.615	0.075	0.817	m^3	
	Column Foundation F19	1	4.125	3.515	0.075	1.087	m^3	
	Column Foundation F20	1	4.875	4.265	0.075	1.559	m^3	
	Column Foundation F21	1	4.115	3.515	0.075	1.085	m^3	
	Column Foundation F22	1	3.625	3.015	0.075	0.820	m^3	
	Column Foundation F23	1	4.265	4.855	0.075	1.553	m^3	
					Total	**61.110**	m^3	
6	**Stone soling & Level including filling joints with Sand as per Specification & Instruction of Site engineer**							
	Flooring at Plinth Level							
	Area of Grids	74	6.016	6.016	0.075	200.867	m^3	
	Foundation Wall							
	Along Grid 1-1,2-2,13-13,14-14	8	4.500	0.500	0.075	1.350	m^3	
	Along Grid 3-3	1	19.000	0.500	0.075	0.713	m^3	
	Along Grid 4-4	1	28.820	0.500	0.075	1.081	m^3	
	Along Grid 5-5	1	28.400	0.500	0.075	1.065	m^3	
	Along Grid 6-6	1	20.800	0.500	0.075	0.780	m^3	
	Along Grid 7-7	1	33.870	0.500	0.075	1.270	m^3	
	Along Grid 8-8	1	26.640	0.500	0.075	0.999	m^3	
	Along Grid 9-9	1	40.800	0.500	0.075	1.530	m^3	
	Along Grid 10-10	1	22.350	0.500	0.075	0.838	m^3	
	Along Grid 11-11	1	20.400	0.500	0.075	0.765	m^3	
	Along Grid 12-12	1	21.850	0.500	0.075	0.819	m^3	
	Along Grid A-A, B-B, M-M, N-N	1	24.700	0.500	0.075	0.926	m^3	
	Along Grid C-C	1	21.410	0.500	0.075	0.803	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
	Along Grid D-D	1	19.780	0.500	0.075	0.742	m^3	
	Along Grid E-E	1	15.090	0.500	0.075	0.566	m^3	
	Along Grid F-F	1	38.520	0.500	0.075	1.445	m^3	
	Along Grid G-G	1	34.410	0.500	0.075	1.290	m^3	
	Along Grid H-H	1	40.800	0.500	0.075	1.530	m^3	
	Along Grid I-I	1	26.580	0.500	0.075	0.997	m^3	
	Along Grid J-J	1	25.740	0.500	0.075	0.965	m^3	
	Along Grid K-K	1	4.520	0.500	0.075	0.170	m^3	
	Along Grid L-L	1	16.380	0.500	0.075	0.614	m^3	
					Total	**222.125**	m^3	
B.	**Brick Work First class brickwork masonry work in cement sand mortar 1:6 in foundation plinth and superstructure**							
	Ground Floor							
	In Y-Direction							
i.	**Main wall**							
	Along Grid 1-1	2	5.596	0.230	3.158	8.129	m^3	6.096-.5/2-.5/2 =5.596
	Along Grid 3-3	2	5.596	0.230	3.158	8.129	m^3	
		1	5.646	0.230	3.158	4.101	m^3	6.096-.5/2-.4/2 =5.646
	Along Grid 4-4	3	5.596	0.230	3.158	12.194	m^3	
	Along Grid 6-6	6	5.596	0.230	3.158	24.388	m^3	
	Along Grid 7-7	9	5.596	0.230	3.158	36.581	m^3	
	Along Grid 9-9	7	5.596	0.230	3.158	28.452	m^3	
	Along Grid 12-12	5	5.596	0.230	3.158	20.323	m^3	
	Along Grid 14-14	2	5.596	0.230	3.158	8.129	m^3	
ii.	**Partition Wall**							
	Along Grid 2-2	1	5.596	0.115	3.533	2.274	m^3	
	Along Grid 3-3	1	5.596	0.115	3.533	2.274	m^3	
	Along Grid 4-4	1	5.696	0.115	3.533	2.314	m^3	
	Along Grid 5-5	5	5.596	0.115	3.533	11.368	m^3	
		1	8.826	0.115	3.533	3.586	m^3	3.13 +5.696 =8.826

SN	Description	No	L	B	H	Qty	Unit	Remark
	Along Grid 8-8	8	5.596	0.115	3.533	18.189	m^3	
	Along Grid 9-9	1	1.961	0.115	3.158	0.712	m^3	
	Along Grid 10-10	3	5.596	0.115	3.533	6.821	m^3	
		1	3.657	0.115	3.533	1.486	m^3	
	Along Grid 11-11	4	5.596	0.115	3.533	9.095	m^3	
		1	2.106	0.115	3.533	0.856	m^3	
	Along Grid 12-12	1	3.780	0.115	3.533	1.536	m^3	
	Along Grid 13-13	2	5.596	0.115	3.533	4.547	m^3	
	Refraction	1	5.912	0.115	3.533	2.402	m^3	
	Dermatology	2	5.912	0.115	3.533	4.804	m^3	
	Dental	1	5.981	0.115	3.533	2.430	m^3	
	Public Toilet	8	1.700	0.115	3.533	5.526	m^3	
	Emergency	1	18.325	0.115	3.533	7.445	m^3	
	Drug Dispensing	1	5.912	0.115	3.533	2.402	m^3	
	Emergency Labs	1	0.500	0.115	3.533	0.203	m^3	
	Radiology	1	18.110	0.115	3.533	7.358	m^3	
	Dark room	1	5.918	0.115	3.533	2.404	m^3	
		1	5.924	0.115	3.533	2.407	m^3	6.096-.23/2-.115/2 =5.924
	Changing room	1	1.600	0.115	3.533	0.650	m^3	
	Control room	1	3.918	0.115	3.533	1.592	m^3	
		1	3.048	0.115	3.533	1.238	m^3	
	Pathology Lab	1	5.918	0.115	3.533	2.404	m^3	
	In X-Direction							
iii.	**Main Wall**							
	Along Grid A-A	2	5.596	0.230	3.533	9.095	m^3	
	Along Grid C-C	5	5.596	0.230	3.158	20.323	m^3	
	Along Grid F-F	5	5.596	0.230	3.158	20.323	m^3	
	Along Grid H-H	8	5.596	0.230	3.158	32.517	m^3	
	Along Grid J-J	3	5.596	0.230	3.158	12.194	m^3	
	Along Grid L-L	3	5.696	0.230	3.158	12.412	m^3	5.696= 6.096-0.4

SN	Description	No	L	B	H	Qty	Unit	Remark
	Along Grid N-N	2	5.596	0.230	3.158	8.129	m^3	
iv.	**Partition Wall**							
	Along Grid B-B	1	5.596	0.115	3.533	2.274	m^3	
	Along Grid C-C	1	5.596	0.115	3.533	2.274	m^3	
	Along Grid D-D	4	5.596	0.115	3.533	9.095	m^3	
	Along Grid E-E	3	5.596	0.115	3.533	6.821	m^3	
		1	3.658	0.115	3.533	1.486	m^3	
	Along Grid F-F	5	5.596	0.115	3.533	11.368	m^3	
		1	3.471	0.115	3.533	1.410	m^3	
	Along Grid G-G	6	5.596	0.115	3.533	13.642	m^3	
		1	3.630	0.115	3.533	1.475	m^3	
	Along Grid H-H	2	5.596	0.115	3.533	4.547	m^3	
		1	3.344	0.115	3.533	1.359	m^3	
	Along Grid I-I	4	5.596	0.115	3.533	9.095	m^3	
		1	2.256	0.115	3.533	0.917	m^3	
	Along Grid J-J	1	5.696	0.115	3.533	2.314	m^3	
	Along Grid K-K	2	3.344	0.115	3.533	2.717	m^3	
	Along Grid L-L	1	5.596	0.115	3.533	2.274	m^3	
		3	3.471	0.115	3.533	4.231	m^3	
	Along Grid M-M	1	5.596	0.115	3.533	2.274	m^3	
	Kitchen & Dining	1	5.851	0.115	3.533	2.377	m^3	
	Dental	1	2.300	0.115	3.533	0.934	m^3	
	Report & Record Room	1	5.969	0.115	3.533	2.425	m^3	
	Public Toilet	3	5.307	0.115	3.533	6.469	m^3	
	Radiology	1	3.092	0.115	3.533	1.256	m^3	
		1	1.397	0.115	3.533	0.568	m^3	
		1	12.141	0.115	3.533	4.933	m^3	
	Changing room	1	1.800	0.115	3.533	0.731	m^3	
	Report & Record Room	1	6.060	0.115	3.533	2.462	m^3	
	Emergency	1	1.090	0.115	3.533	0.443	m^3	
	Staff Accommodation	1	3.048	0.115	3.533	1.238	m^3	
	instrument Sterilization	1	6.096	0.115	3.533	2.477	m^3	
	ECG Room	1	3.480	0.115	3.533	1.414	m^3	
	Emergency beds	1	1.210	0.115	3.533	0.492	m^3	
					Total	**481.534**	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
	Deductions							
	Window W1	36	2.150	0.230	1.500	26.703	m^3	
	Window W2	4	1.800	0.230	0.800	1.325	m^3	
	Ventilation V	4	0.410	0.230	0.610	0.230	m^3	
	Metal Door D1	40	1.320	0.115	2.200	13.358	m^3	
	Double Winged Door	8	1.900	0.115	2.200	3.846	m^3	
	Bathroom Door	11	0.762	0.115	2.200	2.121	m^3	
	Shear wall	20	3.183	0.230	3.158	46.239	m^3	
			Total Deduction			**93.822**	m^3	
		Total brickwork in Ground Floor				**387.712**	m^3	
	First floor							
	In Y-Direction							
i.	**Main wall**							
	Along Grid 3-3	2	5.596	0.230	3.158	8.129	m^3	6.096-.5/2-.5/2 =5.596
	Along Grid 4-4	3	5.596	0.230	3.158	12.194	m^3	
	Along Grid 6-6	4	5.596	0.230	3.158	16.258	m^3	
	Along Grid 7-7	8	5.596	0.230	3.158	32.517	m^3	
	Along Grid 9-9	4	5.596	0.230	3.158	16.258	m^3	
	Along Grid 12-12	5	5.596	0.230	3.158	20.323	m^3	
	Along Grid 14-14	2	5.596	0.230	3.158	8.129	m^3	
ii.	**Partition Wall**							
	Along Grid 5-5	2	5.596	0.115	3.533	4.547	m^3	
	Along Grid 8-8	8	5.596	0.115	3.533	18.189	m^3	
	Along Grid 10-10	4	5.596	0.115	3.533	9.095	m^3	
	Along Grid 13-13	2	5.596	0.115	3.533	4.547	m^3	
	Rest room	3	12.145	0.115	3.533	14.803	m^3	
		4	1.830	0.115	3.533	2.974	m^3	
		1	2.150	0.115	3.533	0.874	m^3	
	Intensive care unit	1	18.240	0.115	3.533	7.411	m^3	
		4	5.918	0.115	3.533	9.618	m^3	
	Ventilation	3	3.530	0.115	3.533	4.303	m^3	
		1	16.000	0.115	3.533	6.501	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
	Minor theatre	1	5.970	0.115	3.533	2.426	m^3	
	In X-Direction							
iii.	**Main Wall**							
	Along Grid A-A	2	5.596	0.230	3.158	8.129	m^3	
	Along Grid C-C	5	5.596	0.230	3.158	20.323	m^3	
	Along Grid F-F	2	5.596	0.230	3.158	8.129	m^3	
	Along Grid H-H	6	5.596	0.230	3.158	24.388	m^3	
	Along Grid J-J	3	5.596	0.230	3.158	12.194	m^3	
	Along Grid M-M	2	5.596	0.230	3.158	8.129	m^3	
iv.	**Partition Wall**							
	Along Grid B-B	1	5.596	0.115	3.533	2.274	m^3	
	Along Grid D-D	1	5.596	0.115	3.533	2.274	m^3	
	Along Grid E-E	4	5.596	0.115	3.533	9.095	m^3	
	Along Grid F-F	2	5.596	0.115	3.533	4.547	m^3	
		1	3.471	0.115	3.533	1.410	m^3	
	Along Grid G-G	3	5.596	0.115	3.533	6.821	m^3	
		1	3.344	0.115	3.533	1.359	m^3	
	Along Grid H-H	2	5.596	0.115	3.533	4.547	m^3	
		1	3.344	0.115	3.533	1.359	m^3	
	Along Grid I-I	4	5.596	0.115	3.533	9.095	m^3	
		1	3.471	0.115	3.533	1.410	m^3	
	Along Grid K-K	1	6.194	0.115	3.533	2.517	m^3	
	Along Grid L-L	2	5.596	0.115	3.533	4.547	m^3	
	Intensive care unit	5	5.918	0.115	3.533	12.022	m^3	
		1	19.379	0.115	3.533	7.874	m^3	
	GW restroom	18	1.549	0.115	3.533	11.328	m^3	
		1	5.923	0.115	3.533	2.406	m^3	
		1	1.033	0.115	3.533	0.420	m^3	
	drug store	2	4.087	0.115	3.533	3.321	m^3	
		1	3.507	0.115	3.533	1.425	m^3	
	Changing room	1	5.967	0.115	3.533	2.424	m^3	
					Total	**372.863**	m^3	
	Deductions							
	Window W1	27	2.150	0.230	1.500	20.027	m^3	
	Window W2	4	1.800	0.230	0.800	1.325	m^3	
	Ventilation V	39	0.410	0.230	0.610	2.243	m^3	
	Metal Door D1	24	1.320	0.115	2.200	8.015		

SN	Description	No	L	B	H	Qty	Unit	Remark
	Double Winged Door	9	1.900	0.115	2.200	4.326	m^3	
	Bathroom Door	38	0.762	0.115	2.200	7.326	m^3	
	shear wall	20	3.183	0.230	3.158	46.239	m^3	
			Total Deductions			**89.501**	m^3	
		Total brickwork in First Floor				**283.362**	m^3	
	Second Floor							
	In Y-Direction							
i.	**Main wall**							
	Along Grid 3-3	2	5.596	0.230	3.158	8.129	m^3	
	Along Grid 4-4	3	5.596	0.230	3.158	12.194	m^3	
	Along Grid 6-6	3	5.596	0.230	3.158	12.194	m^3	
		1	3.284	0.230	3.158	2.385	m^3	
	Along Grid 7-7	6	5.596	0.230	3.158	24.388	m^3	
		1	3.284	0.230	3.158	2.385	m^3	
	Along Grid 9-9	3	5.596	0.230	3.158	12.194	m^3	
	Along Grid 12-12	7	5.596	0.230	3.158	28.452	m^3	
ii.	**Partition Wall**							
	Along Grid 4-4	1	5.596	0.115	3.533	2.274	m^3	
	Along Grid 5-5	3	5.596	0.115	3.533	6.821	m^3	
		1	3.313	0.115	3.533	1.346	m^3	
	Along Grid 8-8	4	5.596	0.115	3.533	9.095	m^3	
		1	3.380	0.115	3.533	1.373	m^3	
	Along Grid 10-10	3	5.596	0.115	3.533	6.821	m^3	
	Along Grid 11-11	3	5.596	0.115	3.533	6.821	m^3	
	Public Toilet	3	12.018	0.115	3.533	14.649	m^3	
	Toilet	1	2.200	0.115	3.533	0.894	m^3	
	NICU	1	12.141	0.115	3.533	4.933	m^3	
	Pediatric ward	5	5.918	0.115	3.533	12.022	m^3	
		3	3.460	0.115	3.533	4.217	m^3	
	Reception	1	8.168	0.115	3.533	3.319	m^3	
	Staff Restroom	1	5.845	0.115	3.533	2.375	m^3	
		1	4.932	0.115	3.533	2.004	m^3	
	In X-Direction							
iii.	**Main Wall**							
	Along Grid A-A	2	5.596	0.230	3.158	8.129	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
	Along Grid C-C	5	5.596	0.230	3.158	20.323	m^3	
	Along Grid F-F	1	5.596	0.230	3.158	4.065	m^3	
	Along Grid H-H	2	5.596	0.230	3.158	8.129	m^3	
	Along Grid J-J	3	5.596	0.230	3.158	12.194	m^3	
	Along Grid L-L	2	5.596	0.230	3.158	8.129	m^3	
iv.	**Partition Wall**							
	Along Grid C-C	1	5.596	0.115	3.533	2.274	m^3	
	Along Grid D-D	3	5.596	0.115	3.533	6.821	m^3	
	Along Grid E-E	3	5.596	0.115	3.533	6.821	m^3	
	Along Grid F-F	2	5.596	0.115	3.533	4.547	m^3	
		1	3.471	0.115	3.533	1.410	m^3	
	Along Grid G-G	2	5.596	0.115	3.533	4.547	m^3	
	Along Grid H-H	3	5.596	0.115	3.533	6.821	m^3	
	Along Grid I-I	3	5.596	0.115	3.533	6.821	m^3	
		1	3.342	0.115	3.533	1.358	m^3	
	Along Grid K-K	2	3.048	0.115	3.533	2.477	m^3	
	Administration	1	12.192	0.115	3.533	4.954	m^3	
	Staff Restroom	7	1.870	0.115	3.533	5.318	m^3	
	Public Toilet	18	1.549	0.115	3.533	11.328	m^3	
	NICU Bathroom	1	6.000	0.115	3.533	2.438	m^3	
	Pediatric ward	1	10.846	0.115	3.533	4.407	m^3	
					Total	**314.596**	m^3	
	Deductions							
	Window W1	21	2.150	0.230	1.500	15.577	m^3	
	Window W2	4	1.800	0.230	0.800	1.325	m^3	
	Ventilation V	43	0.410	0.230	0.610	2.473	m^3	
	Metal Door D1	12	1.320	0.115	2.200	4.008	m^3	
	Double Winged Door	5	1.900	0.115	2.200	2.404	m^3	
	Bathroom Door	38	0.762	0.115	2.200	7.326	m^3	
	Shear wall	20	3.183	0.230	3.158	46.239	m^3	
			Total Deductions			**79.352**	m^3	
		Total brickwork in Second Floor				**235.244**	m^3	
	Third Floor							
	In Y-Direction							
i.	**Main wall**							
	Along Grid 4-4	1	5.596	0.230	3.158	4.065	m^3	
	Along Grid 5-5	1	5.596	0.230	3.158	4.065	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
	Along Grid 8-8	6	5.596	0.230	3.158	24.388	m^3	
	Along Grid 9-9	6	5.596	0.230	3.158	24.388	m^3	
ii.	**Parapet Wall**							
	Along Grid 3-3	1	12.700	0.115	1.000	1.461	m^3	
	Along Grid 4-4	1	12.156	0.115	1.000	1.398	m^3	
	Along Grid 7-7	1	12.192	0.115	1.000	1.402	m^3	
		1	18.288	0.115	1.000	2.103	m^3	
	Along Grid 9-9	1	6.096	0.115	1.000	0.701	m^3	
	Along Grid 12-12	1	43.180	0.115	1.000	4.966	m^3	
	In X-Direction							
iii.	**Main Wall**							
	Along Grid A-A	1	5.596	0.230	3.158	4.065	m^3	
	Along Grid F-F	2	5.596	0.230	3.158	8.129	m^3	
	Along Grid H-H	1	5.596	0.230	3.158	4.065	m^3	
	Along Grid I-I	1	5.596	0.230	3.158	4.065	m^3	
iv.	**Parapet Wall**							
	Along Grid C-C	1	12.332	0.115	1.000	1.418	m^3	
		1	18.144	0.115	1.000	2.087	m^3	
	Along Grid F-F	1	6.096	0.115	1.000	0.701	m^3	
	Along Grid H-H	1	18.000	0.115	1.000	2.070	m^3	
	Along Grid J-J	1	18.288	0.115	1.000	2.103	m^3	
	Along Grid L-L	1	12.420	0.115	1.000	1.428	m^3	
					Total	**99.068**	m^3	
	Deductions							
	Window W2	18	1.800	0.230	0.800	5.962	m^3	
	Metal Door D1	4	1.320	0.115	2.200	1.336	m^3	
			Total Deductions			**7.298**	m^3	
		Total brickwork in Third Floor				**91.770**	m^3	
C.	**Concrete work for Foundation**							
	Footing F1	15	1.500	1.500	0.400	13.500	m^3	
	Footing F2	4	1.750	1.750	0.400	4.900	m^3	
	Footing F3	6	2.250	2.250	0.400	12.150	m^3	
	Footing F4	1	3.000	3.000	0.400	3.600	m^3	
	Footing F5	13	2.500	2.500	0.400	32.500	m^3	
	Footing F6	7	2.000	2.000	0.400	11.200	m^3	
	Footing F7	3	2.750	2.750	0.400	9.075	m^3	
	Footing F8	1	4.250	4.250	1.350	24.384	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
	Footing F9	23	3.000	3.000	0.700	144.900	m^3	
	Footing F10	11	3.250	3.250	0.700	81.331	m^3	
	Footing F11	2	3.250	3.250	0.900	19.013	m^3	
	Footing F12	2	3.750	3.750	0.900	25.313	m^3	
	Footing F13	1	3.360	2.750	0.700	6.468	m^3	
	Footing F14	1	3.858	3.840	0.700	10.370	m^3	
	Footing F15	1	3.708	3.100	0.700	8.046	m^3	
	Footing F16	1	3.610	3.000	0.900	9.747	m^3	
	Footing F17	1	3.860	3.250	0.900	11.291	m^3	
	Footing F18	1	3.000	3.590	0.900	9.693	m^3	
	Footing F19	1	4.110	3.500	0.900	12.947	m^3	
	Footing F20	1	4.860	4.250	1.200	24.786	m^3	
	Footing F21	1	4.108	3.500	1.200	17.254	m^3	
	Footing F22	1	3.610	3.000	1.000	10.830	m^3	
	Footing F23	1	4.250	4.840	1.000	20.570	m^3	
					Total	**523.868**	m^3	
D.	**Plain Cement Concrete M25(1:1:2) work for RCC Columns upto plinth Ground Level**							
	Footing F1							
	C1	1	0.500	0.500	1.560	0.390	m^3	
	C8	3	0.300	0.300	1.560	0.421	m^3	
	C6	11	0.400	0.400	1.560	2.746	m^3	
	Footing F2							
	C1	3	0.500	0.500	1.560	1.170	m^3	
	C6	1	0.400	0.400	1.560	0.250	m^3	
	Footing F3							
	C1	3	0.500	0.500	1.560	1.170	m^3	
	C2	3	0.500	0.500	1.560	1.170	m^3	
	Footing F4							
	C1	1	0.500	0.500	1.560	0.390	m^3	
	Footing F5							
	C1	12	0.500	0.500	1.560	4.680	m^3	
	C4	1	0.500	0.500	1.560	0.390	m^3	
	Footing F6							
	C1	4	0.500	0.500	1.560	1.560	m^3	
	C2	1	0.500	0.500	1.560	0.390	m^3	
	C4	2	0.500	0.500	1.560	0.780	m^3	
	Footing F7							

SN	Description	No	L	B	H	Qty	Unit	Remark
	C1	2	0.500	0.500	1.560	0.780	m^3	
	C3	1	0.500	0.500	1.560	0.390	m^3	
	Footing F8							
	C1	1	0.500	0.500	1.610	0.403	m^3	
	Footing F9							
	C1	21	0.500	0.500	1.260	6.615	m^3	
	C2	1	0.500	0.500	1.260	0.315	m^3	
	C4	1	0.500	0.500	1.260	0.315	m^3	
	Footing F10							
	C1	10	0.500	0.500	1.260	3.150	m^3	
	C2	1	0.500	0.500	1.260	0.315	m^3	
	Footing F11							
	C1	2	0.500	0.500	2.060	1.030	m^3	
	Footing F12							
	C1	2	0.500	0.500	2.060	1.030	m^3	
	Footing F13							
	C1	1	0.500	0.500	1.260	0.315	m^3	
	C6	1	0.400	0.400	1.260	0.202	m^3	
	Footing F14							
	C1	2	0.500	0.500	1.260	0.630	m^3	
	C6	1	0.400	0.400	1.260	0.202	m^3	
	Footing F15							
	C1	1	0.500	0.500	1.260	0.315	m^3	
	C2	1	0.500	0.500	1.260	0.315	m^3	
	Footing F16							
	C1	2	0.500	0.500	2.770	1.385	m^3	
	Footing F17							
	C1	2	0.500	0.500	2.770	1.385	m^3	
	Footing F18							
	C1	2	0.500	0.500	2.770	1.385	m^3	
	Footing F19							
	C1	2	0.500	0.500	2.770	1.385	m^3	
	Footing F20							
	C1	2	0.500	0.500	1.760	0.880	m^3	
	Footing F21							
	C1	2	0.500	0.500	1.760	0.880	m^3	
	C6	1	0.400	0.400	1.760	0.282	m^3	
	Footing F22							
	C1	2	0.500	0.500	1.960	0.980	m^3	
	Footing F23							

SN	Description	No	L	B	H	Qty	Unit	Remark
	C1	2	0.500	0.500	1.960	0.980	m^3	
					Total	**41.371**	m^3	
E.	**Plain Cement Concrete M25(1:1:2) work for RCC Column**							
i.	**Ground Floor**							
	Column C1	82	0.500	0.500	3.650	74.825	m^3	
	Column C2	7	0.500	0.500	3.650	6.388	m^3	
	Column C3	1	0.500	0.500	3.650	0.913	m^3	
	Column C4	3	0.500	0.500	3.650	2.738	m^3	
	Column C6	15	0.400	0.400	3.650	8.760	m^3	
	Column C8	3	0.300	0.300	3.650	0.986	m^3	
ii.	**First Floor**							
	Column C1	77	0.500	0.500	3.650	70.263	m^3	
	Column C2	8	0.500	0.500	3.650	7.300	m^3	
	Column C3	1	0.500	0.500	3.650	0.913	m^3	
	Column C4	3	0.500	0.500	3.650	2.738	m^3	
iii.	**Second Floor**							
	Column 1	69	0.500	0.500	3.650	62.963	m^3	
	Column 2	7	0.500	0.500	3.650	6.388	m^3	
	Column 3	1	0.500	0.500	3.650	0.913	m^3	
	Column 4	3	0.500	0.500	3.650	2.738	m^3	
iv.	**Third Floor**							
	Column 1	22	0.500	0.500	3.650	20.075	m^3	
	Column 2	2	0.500	0.500	3.650	1.825	m^3	
					Total	**270.726**	m^3	
F.	**Plain Cement Concrete M25(1:1:2) work for RCC beam**							
i.	**First Floor**							
	Primary Beam	180	6.096	0.350	0.500	192.024	m^3	
	Secondary Beam	70	6.096	0.250	0.300	32.004	m^3	
ii.	**Second Floor**							
	Primary Beam	144	6.096	0.350	0.500	153.619	m^3	
	Secondary Beam	53	6.096	0.250	0.300	24.232	m^3	
iv.	**Third Floor**							
	Primary Beam	129	6.096	0.350	0.500	137.617	m^3	

SN	Description	No	L	B	H	Qty	Unit	Remark
	Secondary Beam	45	6.096	0.250	0.300	20.574	m^3	
					Total	**560.070**	m^3	
G.	**Plain Cement Concrete M25(1:1:2) work for RCC Slab**							
	Slab Ground Floor	62	6.096	6.096	0.120	276.479	m^3	
	Slab First Floor	47	6.096	6.096	0.120	209.589	m^3	
	Slab Second Floor	41	6.096	6.096	0.120	182.833	m^3	
					Total	**668.901**	m^3	
H.	**Plain Cement Concrete M15 (1:2:4) work for Ground Floor**							
	PCC For Flooring at Ground Floor	74	6.096	6.096	0.075	206.245	m^3	
	Entry Porch	1	22.246	3.693	0.075	6.162	m^3	
	Entry Porch	1	13.874	5.482	0.075	5.704	m^3	
	Entry Porch	1	15.576	3.808	0.075	4.449	m^3	
	Entry Porch	1	12.802	4.338	0.075	4.165	m^3	
					Total	**226.725**	m^3	
I.	**Plain Cement Concrete M30 (1:0.75:1.5) work for Shear Wall**							
	Block C	8×3	3.183	0.200	3.158	48.249	m^3	
	Block A	4×3	3.183	0.200	3.158	24.125	m^3	
	Block B	4×3	6.096	0.200	3.158	46.203	m^3	
	Near Lift	1×3	17.493	0.200	3.658	38.394	m^3	
					Total	**156.971**	m^3	

S.N	Description	No	Length	Wt. per meter	Qty	Unit	Remarks
J.	**Calculation of Steel Reinforcement for footing**						

S.N	Description	No	Length	Wt. per meter	Qty	Unit	Remarks
1.	10Φ 160mm c-c	10	1.700	0.620	21.080	kg	In both direction
2.	10Φ 150mm c-c	12	1.950	0.620	29.016	kg	In both direction
3.	10Φ 130mm c-c	18	2.450	0.620	54.684	kg	In both direction
4.	10Φ 130mm c-c	24	3.200	0.620	95.232	kg	In both direction
5.	10Φ 100mm c-c	25	2.700	0.620	83.700	kg	In both direction
6.	10Φ 160mm c-c	13	2.200	0.620	35.464	kg	In both direction
7.	10Φ 90mm c-c	30	2.950	0.620	109.740	kg	In both direction
8.	10Φ 125mm c-c	35	4.450	0.620	193.130	kg	In both direction
9.	10Φ 90mm c-c	34	3.200	0.620	134.912	kg	In both direction
10.	10Φ 90mm c-c	36	3.450	0.620	154.008	kg	In both direction
11.	16Φ 190mm c-c	18	3.450	1.580	196.236	kg	In both direction
12.	16Φ 180mm c-c	22	3.950	1.580	274.604	kg	In both direction
13.	12Φ 110mm c-c	31	3.560	0.890	98.220	kg	Along X-direction
	12Φ 100mm c-c	28	2.950	0.890	73.514	kg	Along Y-direction
14.	10Φ 90mm c-c	43	4.058	0.620	108.186	kg	Along X-direction
	10Φ 90mm c-c	43	4.040	0.620	107.706	kg	Along Y-direction
15.	12Φ 120mm c-c	31	3.908	0.890	107.822	kg	Along X-direction
	10Φ 100mm c-c	31	3.300	0.620	63.426	kg	Along Y-direction
16.	16Φ 160mm c-c	23	3.810	1.580	138.455	kg	Along X-direction
	16Φ 200mm c-c	16	3.200	1.580	80.896	kg	Along Y-direction
17.	16Φ 170mm c-c	24	4.060	1.580	153.955	kg	Along X-direction
	16Φ 200mm c-c	17	3.450	1.580	92.667	kg	Along Y-direction

S.N	Description	No	Length	Wt. per meter	Qty	Unit	Remarks
18.	16Φ 200mm c-c	16	3.200	1.580	80.896	kg	Along X-direction
	16Φ 160mm c-c	23	3.790	1.580	137.729	kg	Along Y-direction
19.	16Φ 175mm c-c	24	4.310	1.580	163.435	kg	Along X-direction
	16Φ 200mm c-c	18	3.700	1.580	105.228	kg	Along Y-direction
20.	16Φ 120mm c-c	41	5.060	1.580	327.787	kg	Along X-direction
	16Φ 140mm c-c	31	4.450	1.580	217.961	kg	Along Y-direction
21.	16Φ 130mm c-c	32	4.308	1.580	217.812	kg	Along X-direction
	16Φ 150mm c-c	24	3.700	1.580	140.304	kg	Along Y-direction
22.	16Φ 150mm c-c	25	3.810	1.580	150.495	kg	Along X-direction
	16Φ 180mm c-c	18	3.200	1.580	91.008	kg	Along Y-direction
23.	16Φ 180mm c-c	24	4.450	1.580	168.744	kg	Along X-direction
	16Φ 150mm c-c	33	5.040	1.580	262.786	kg	Along Y-direction
				Total	**4470.838**	kg	
K.	**Reinforcement works including cutting, bending, placing and binding**						
a.	Slab	0.01×7850×668.902			52508.807	kg	Density of steel=7850kg/m^3
b.	Beams	0.015×7850×560.07			65948.243	kg	
c.	Columns	0.03×7850×312.0897			73497.124	kg	
d.	Shear wall						
	16Φ 120mm c-c in X-direction	8	12.64 kg of 16mm rebar/m^2				
	16Φ 300mm c-c in Y-direction	18	28.44 kg of 16mm rebar/m^2				
	For 1m^3 of wall				205.400	kg	41.08/1×1×0.2=205.4
	Total reinforcement				32241.638	kg	For 156.97 cu.m of

S.N	Description	No	Length	Wt. per meter	Qty	Unit	Remarks
	required for shear wall						concrete work
	Total reinforcement works for slab, beam, columns and shear wall			**224195.812**		kg	

S.N	Description	No	L	B	Height	Qty	Unit	Remarks
L.	**Ramp Works**							
	Plain Cement Concrete M25(1:1:2) Work on Ramp							
	i. Flight	6	25.360	2.400	0.150	54.778	m^3	
	ii. Landing	6	5.900	2.400	0.150	12.744	m^3	
					Total	**67.522**	m^3	
	Plastering work on Ramp							
	i. Flight	6	25.360	2.400		365.184	m^2	
	sides of flight	6	25.360	0.170		25.867	m^2	
	ii. Landing	6	5.900	2.400		84.960	m^2	
	sides of landing	6	0.991	0.170		1.010	m^2	
					Total	**477.021**	m^2	
	Reinforcement on Ramp							
	16mmØ rebar @140mmC/C	11.28kg of 16mm rebar per meter square of ramp						
	8mmØDistribut-ion rebar @250mmC/C	1.58 kg of 8mm rebar per meter square of ramp						
	i.Flight							
	16mmØ rebar @140mmC/C	6	25.360	2.400	11.280	4119.276	kg	
	8mmØ Distribution rebar @250mmC/C	6	25.360	2.400	1.580	576.991	kg	
	ii. Landing						kg	
	16mmØ rebar @140mmC/C	6	5.900	2.400	11.280	958.349	kg	
	8mmØ Distribution rebar @250mmC/C	6	5.900	2.400	1.580	134.237	kg	

S.N	Description	No	L	B	Height	Qty	Unit	Remarks
					Total	**5788.853**	kg	
M.	**Works on staircase**							
	RCC Work (1:1:2) Including Centering and Shuttering but excluding Reinforcement							
	i) Base on Toe Wall: H = 0.40	1	1.524	0.400	0.400	0.244	m^3	
	ii) Waist Slab of Flight: L = $\sqrt{3.353^2+1.676^2}$ = 3.749m	2	3.749	1.524	0.160	1.828	m^3	
	iii) Landing(Middle & First Floor) B = 1.524 + 0.15(Bearing) = 1.674m	2	3.758	1.674	0.160	2.013	m^3	
	iv) Steps without nosing	2×11	1.524	0.305	0.5×0.1524	0.779	m^3	
					Total	**4.864**	m^3	
	Multiplying Total by 6 for whole building					**29.184**	m^3	
	20 mm Cement Plaster (1:6) in Steps:							
	Tread	2×11	1.524	0.305		10.226	m^2	
	Riser	2×12	1.524	0.152		5.560	m^2	
	Ends of Steps	2×11	0.305	0.5×0.152		0.510	m^2	
	Inner edge of waist slab	2	3.749	0.152		1.140	m^2	
					Total	**17.436**	m^2	
	Multiplying Total by 6 for whole building					**104.616**	m^2	
	2.5 cm nosing in steps in 1:3 Cement Mortar	2×12	1.524			**36.576**	m	
	Multiplying Total by 6 for whole building					**219.456**	m	
	2.5 cm C. C. Floor Finishing for							

S.N	Description	No	L	B	Height	Qty	Unit	Remarks
	middle and first floor landing							
	i) Front Landing	1	3.758	1.12		4.209	m^2	
	ii) Middle Landing	1	3.758	1.524		5.727	m^2	
					Total	**9.936**	m^2	
	Multiplying Total by 6 for whole building					**34.362**	m^2	
	M.S ornamental grill for railing	2	3.749	1.524		**11.427**	m^2	
	Multiplying Total by 6 for whole building					**68.562**	m^2	
	Reinforcement: **Main Steel 20 mm dia. @ 130 mm c/c**							
	i) Lower flight and landing:	12	5.893	@	2.470	174.669	kg	
	L = 3.7485 + 0.3(at toe) +1.524 +2x9x0.02(hook) -2x0.02(cover)=5.893 m							
	No. of Bars = (1.524/0.14) + 1= 11.88 =12							
	ii) For upper flight, middle and first floor landing	12	7.013	@	2.470	207.865	kg	
	L =0.15+1.524+3.7485+1.120+0.15+ 2x9x0.02-2x0.02= 7.013							
	iii) Top bars (20 mm dia.) in middle and First floor landing.							
	L = 0.15 +1.524 +0.97(Ld)+2x9x 0.02- 0.02 = 2.984m	2×12	2.984	@	2.470	176.892	kg	
					Total	**559.426**	kg	
	Multiplying Total by 6 for whole building					**3356.556**	kg	
	Distribution Steel:							
	8 mm dia. - @ 250 mm c/c For Lower flight and Upper Flight							
	L = 1.524+ 2x9x0.008- 2x0.02 = 1.628m							

S.N	Description	No	L	B	Height	Qty	Unit	Remarks
	Nos =(3.7485/0.25) + 1= 16nos, 2x16 = 32	32	1.628	@	0.390	20.317	kg	
	For Landing:							
	Top landing							
	L = 1.524+ 2x9x0.008 - 2x0.02= 1.628m							
	nos. = (1.524+0.15)/ 0.25+1= 7.696= 8nos.	8	1.628	@	0.390	5.079	kg	
	Middle Landing:							
	L = 3.353+ 2x9x0.008- 2x0.02= 3.457 m							
	Nos. = (1.524+0.15)/ 0.25+1= 7.696 = 8 nos.	8	3.457		0.390	10.786	kg	
					Total	**36.182**	kg	
	Multiplying Total by 6 for whole building					**217.092**	kg	

9.3 Abstract of Cost

Table 9-15: Abstract of Cost

S.N	Description	Quantity	Units	Rate per unit		Amount	Remarks
				(In figure)	(In Words)		
	A. CIVIL WORKS						
	I. Dismantle and Site Preparation Work						
1.1	Site clearance work cutting & uprooting herbs & shrubs, topsoil cutting, removal waste all complete	6970	sq.m	117.782		820938.449	
	II. Earth Work						
2.1	Earth work in excavation in foundation trenches in hard soil including lead upto 30m & lift 1.5 m staking the excavated earth as per specification & instruction of site engineer	1713.362	cu.m	663.324		1136513.279	
2.2	Earth back filling in foundation trenches & floor in 15 cm layer with sprinkling water as per specification & instruction of site engineer	2289.532	cu.m	663.324		1518700.380	

S.N	Description	Quantity	Units	Rate per unit		Amount	Remarks
				(In figure)	(In Words)		
	III. Stone Work						
3.1	Stone soling on floor & foundation in line & level including filling the joint with sand as per specification & instruction of site engineer	222.125	cu.m	5514.558		1224921.196	
	IV. Brick Work						
4.1	Brick soling in foundation & floor in line & level with packing sand in the gap as per specification & instruction of site engineer	61.11	sq.m	16759.652		1024182.334	
4.2	First Class Chimney Bhatta Brick masonry work in cement sand mortar 1:3 in foundation plinth & Super Structure in line & level with proper mixing the mortar, laying & packing the joint and curing the job all complete as per specification & instruction	998.088	cu.m	20200.422		20161798.793	
	V. PCC Work						
5	Plain cement concrete work in foundation & floor with cement, fine sand & crushed stone ballast of 10-38 mm size aggregate, proper mixing with mixture machine, laying & placing in position compacting with						

S.N	Description	Quantity	Units	Rate per unit		Amount	Remarks
				(In figure)	(In Words)		
	vibrator, including curing the job as per specification & instruction of site engineer						
5.1	P.C.C. M10 (1:3:6) work	283.235	cu.m	15808.069		4477398.423	
5.2	P.C.C. M15 (1:2:4) work on floor & other places	226.725	cu.m	17273.084		3916239.970	
	VI. RCC Work						
6	Plain cement concrete work in column, beam & slab with cement, fine sand & crushed stone ballast of 10-38 mm size, proper mixing with mixture machine, laying & placing in position compacting with vibrator, including curing the job as per specification & instruction of site engineer						
6.1	P.C.C. M25 (1:1:2) for R.C.C work	2161.642	cu.m	23649.185		51121071.562	
6.2	P.C.C. M30 (1:0.75:1.5) on shear wall	156.971	cu.m	25452.930		3995371.875	
7	TMT/ Tor Steel Reinforcement work for R.C.C. including supply, straightening, cutting, laying, bending & binding with G.I. wire as per design, drawing & specification & instruction of site engineer	238029.151	kg	158.062		37623306.538	

S.N	Description	Quantity	Units	Rate per unit		Amount	Remarks
				(In figure)	(In Words)		
	VII. Plastering & finishing work						
8	12.5 mm thick cement sand plaster 1:4 in wall inside & outside of the building with cement sand proper mixing wetting the surface & curing the job all complete as per specification & instruction of site engineer	21352.6248	sq.m	474.597		10133889.323	
9	20 mm plaster 1:4 in wall, cement & sand with 3 mm neat cement punning work proper mixing, wetting & racking the surface with better finishing in line & level including curing the job as per specification & instruction of site engineer	494.47	sq.m	615.578		304385.027	
10	12.5 mm thick cement sand plaster 1:3 in Ceiling with cement sand proper mixing wetting the surface & curing the job all complete as per specification & instruction of site engineer	5342.7146	sq.m	474.597		2535635.733	
	VIII. Painting and coloring work	21352.625	sq.m	873.651		18654742.009	
					Total	**Rs 15,86,49,094.891**	

10. LIMITATIONS

Although the project work is an academic one, every effort has been made for the project work as practicable as possible. However, as with any project, there will be some limitations which needs to be acknowledged and taken into account while interpreting the results. The limitations in our project work are as follows:

i. Geological investigations are not carried out.
ii. Environmental impact assessment of the project is not performed.
iii. Design, layout and estimation of the building services like water supply pipelines, electrical fittings, sanitary and sewage are not covered.
iv. Detail design of truss and connection joints is not done.
v. Analysis for shear wall reinforcement is not carried out.
vi. Only estimation of major civil works is carried out, complete detailed estimation is not done.

11. CONCLUSION

The fact that Nepal lies in a seismic zone V is a big factor to be accounted for in the design of any structure that aims to be safe, durable and serviceable especially for important buildings like hospitals. And also acknowledging the fact that there is dire need of health service infrastructures this project titled "**SEISMIC ANALYSIS AND DESIGN OF HOSPITAL BUILDING**" was carried out with the best of our team's knowledge incorporating necessary design procedures from established design standards. Hence, we as the students of Civil Engineering hope that this project meets the expectations of our respected supervisor and the rest of our teachers to whom we owe the sum total of our knowledge in this subject.

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ANNEX A

Drawings

List of Drawings