TRIBHUWAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
DEPARTMENT OF CIVIL ENGINEERING

FINAL YEAR PROJECT REPORT on EARTHQUAKE RESISTANT ANALYSIS AND DESIGN OF MULTISTORIED HOSPITAL BUILDING

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Supervisor:
Asst. Prof. Arun Paudel

April 2023

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EARTHQUAKE RESISTANT ANALYSIS AND DESIGN OF MULTISTORIED HOSPITAL BUILDING

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

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CERTIFICATE

This is to certify that this project work entitled "**SEISMIC ANALYSIS AND DESIGN OF MULTISTORIED HOSPITAL BUILDING**" has been examined and declared successful for the fulfilment of academic requirement towards the completion of Bachelor Degree in Civil Engineering.

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ABSTRACT

The objective of this project is not solely restricted to the B.E. Civil final semester curriculum, as it also offers the student community a comprehensive report on **"Seismic Analysis and Design of Multistoried Hospital Building,"** covering a range of topics.

A building must satisfactorily fulfill numerous functions, both structural and aesthetic. These functions include the building's usability for its intended purpose and occupancy, its structural safety, its ability to withstand fires, and its adherence to hygienic sanitation, ventilation, and daylight standards. Additionally, the building must be stable and structurally sound, with its design dependent on the minimum requirements for each of these functions.

Following the Gorkha Earthquake that occurred on April 25th, 2015, the construction of multistoried buildings has been under close scrutiny and supervision by structural engineers. Before commencing construction, it is vital to verify the design quality of various structural elements of a building and maintain quality control during the construction process. Additionally, it is essential to consider the different types of loads that the structure will encounter during its service life. This report outlines the considerations, procedures, and results of the structural design of a hospital building to be constructed in Pokhara.

In this report, special care has been taken to the analysis of vertical and lateral forces and detailing of structural elements and is conformed to respective codes in every way possible. Efforts been made to ensure that this report is free of errors, but mistakes may still occur. Constructive criticism is warmly welcomed, and we would be obliged if any errors are brought to our attention.

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ABBREVIATIONS

X_u	Actual Depth of Neutral Axis
A_{sc}	Area of Steel in Compression
A_{st}	Area of Steel in Tension
A_{sv}	Area of Stirrups
f_{ck}	Characteristic Compressive Strength of Concrete
f_y	Characteristic Strength of Steel
L_d	Development Length
d’	Effective Cover to Reinforcement
D	Effective Depth of Member
l_{eff}	Effective Length of Member
A_g	Gross Area of Concrete
H	Height of Building
A_h	Horizontal Seismic Coefficient
$X_{u,max}$	Limiting Depth of Neutral Axis
E_s	Modulus of Elasticity of Steel
D	Overall Depth of Member
γ_m	Partial Safety Factor for Material
p_e	Percentage of Compression Steel
p_t	Percentage of Tension steel
τ_c	Shear Stress
f_{sc}	Stress in Steel in Compression
f_s	Stress in Steel in Tension
e	Structural Eccentricity
E	Young’s Modulus of Elasticity
CM	Centre of Mass
DL	Dead Load
EQ	Earthquake Load
IS	Indian Standards
IOE	Institute of Engineering

LL	Live Load
RC	Reinforced Concrete
SP	Special Publication
R/F	Reinforcement
BM	Bending Moment
L	Live load
X	Spacing of stirrups
P	Percentage of steel in the section
P_u	Factored design axial load
P_{uz}	Capacity of the cross section under pure axial load
M_u	Factored design moment
M_{lim}	Limiting factored moment of resistance
M_{uy}	Factored designed moment about Y-axis
M_{uyl}	Maximum moment capacity for bending along Y-axis
M_{ux}	Factored design moment along X-axis
EQ_X	Earthquake load along x-direction
EQ_Y	Earthquake load along Y-direction
RLL	Reduced live load
V_B	Base shear
V_u	Factored shear force
M.O.I	Moment Of Inertia

1 INTRODUCTION

1.1 Background

The proposed project is **"Seismic Analysis and Design of Multistoried Institutional Building"** to be built in Pokhara. The sky rocketing population and haphazard land use has decreased the land availability for construction of any structure requiring large plinth area. With due consideration to this fact, a high-rise building seemed to be one of the best options. Taking into account this fact, we have come up with a project work on "Computer Aided Structural Analysis and Design of Multi Story Building".

The bearing capacity of foundation soil at site condition is taken as 140 KN/m^2 and foundation was designed accordingly. The structural analysis was done with help of computer software ETABS and design was done on spreadsheet applications like MS-EXCEL, MS-WORD.

Designers face the challenge of dealing with a wide range of structures, including simple and complex ones like multistoried frame buildings. These structures are subject to various loads, such as concentrated loads, uniformly distributed loads, live loads, earthquake loads, wind loads, and more. As the structure transfers the loads to the supports and eventually to the ground, all members of the structure are subjected to internal forces such as axial forces, shear forces, bending moments, and torsion moments.

The field of Structural Analysis involves the examination of internal forces within structural members that arise due to a variety of loading conditions or combinations.

Structural design involves determining the appropriate size of different members within a structure in order to withstand the internal forces to which they are exposed throughout their life cycle. To ensure that the structural design is effective, it is essential to adopt the appropriate structural detailing method. Following a Standard Code of Practice (such as the Indian Standard code) is critical for ensuring proper analysis, design, and detailing that prioritize safety, economy, stability, and strength.

Due to the prevalence of earthquakes in areas such as Pokhara, seismic activity is a major factor to consider when designing multistory frame buildings. In accordance with IS1893:2016, Pokhara is classified as being in the V zone, which is the most severe seismic zone, meaning that the impact of earthquakes is more significant than that of wind loads. Therefore, the building is analyzed using earthquake as the lateral load. The seismic coefficient design method specified in IS 1893:2016 is used to analyze the building for earthquake resistance. The main structural system of the building is a three-dimensional moment resistance frame with shear walls.

Our project does not take wind load into account except for the truss portion, based on our location's geography. We assume that wind and seismic loads do not occur simultaneously. The building is designed to withstand whichever load is the most severe. It is possible to estimate live and dead loads with a reasonable degree of accuracy, but earthquake loads are difficult to predict accurately. Therefore, statistical and probabilistic methods are used, taking into account factors such as economy.

This project work has been undertaken for the partial fulfillment of requirements for the Bachelor’s Degree in Civil Engineering. This project work contains structural analysis, design and detailing of a multi-story building. All the theoretical knowledge on analysis and design acquired during the course works are utilized with practical application. The main objective of the project work is to acquaint us in the practical aspects of civil engineering.

1.2 Title and Theme of Project work

The group working on this project has focused on Computer Aided Structural Analysis and Design of a Multi-Story Building. The main objective of this project is to gain knowledge and skills with an emphasis on practical application. In addition to utilizing analytical methods and design approaches, another objective is to gain exposure and experience in the application of various available codes of practice.

1.3 Objectives

The goal of reinforced concrete design is to ensure that there is an acceptable probability that structures being designed will perform satisfactorily throughout their intended life with an appropriate degree of safety. They should be able to withstand the entire load and deformation expected during normal construction and use, while also providing adequate durability and resistance to the effects of misuse and fire.

The specific objectives of the project work are:

a. Preparation of the plan of the building to meet the requirements for its intended use.

b. Identification of the structural arrangement of the plan.
c. Building modeling for structural analysis
d. Analyzing the structure using structural analysis program.
e. Sectional design of the structural members.
f. Preparation of detail structural drawing of the design.

1.4 Building Description

a. Building Type: Multi Story RCC Framed Hospital Building
b. Structural System: RCC Space Frame
c. Plinth area covered: 4122.6506 m^2 (44375.84 sq.ft)
d. Type of Foundation: Mat Foundation
e. No. of Story Floor: 8
f. Floor Height: 3.9 m
g. Type of Sub-Soil: Medium
h. Soil Seismic zone: V

As per Clause 27 of IS 456-2000, structures that undergo sudden changes in plan dimensions must be equipped with expansion joints at the section where such changes occur. It is important to ensure that the reinforcement does not extend across the expansion joint, and that the break between the sections is complete. Typically, structures that exceed 45 meters in length are designed with one or more expansion joints. The purpose of the expansion joint is to allow for the expansion and contraction of the structure due to temperature changes or other factors, thereby preventing cracking or other forms of damage.

1.5 Identification of Load

a. Dead loads are calculated according to **IS: 875 (Part I) – 1987**
b. Seismic load is calculated according to **IS: 1893 (Part I) -2016** assuming the location falling in **Zone V (Pokhara)**
c. Imposed loads as per **IS: 875 (Part II) – 1987**

1.6 Code of Practices

The analysis and design of the building were carried out in accordance with the codes of practice established by the Bureau of Indian Standards as following:

a. **IS 456:2000** (Code for practice for plain and reinforced concrete)
b. **IS 1893 (part 1):2016** (Criteria for earthquake resistant design of structures)
c. **IS 13920: 2016** (Code of practice for ductile detailing of reinforced concrete structures subjected to seismic forces)
d. **IS 875 (part 1):1987** (assessment of dead loads)
e. **IS 875 (part 2):1987** (assessment of live loads)
f. **SP 16 and SP 34** (design aid and hand-book)

1.7 Method of Analysis

The space frame of the building was created using **ETABS 19.0.2**, which is a Finite Element Method-based program used for analysis. The stresses, displacements, and fundamental time periods of the building were obtained from **ETABS 19.0.2**, considering the possible actions in the building. These results were used for the design of the members.

Mat foundation, steel structures (if any), staircase, and slabs have been analyzed separately.

1.8 Design

The following materials have been adopted for the design of the elements:

- Concrete Grade:
  i. M30 for the beams, columns, slabs and foundation
- Reinforcement Steel:
  i. Fe 500 for staircase and other structural members

The design of RC elements is carried out using Limit State Method. The design is based on Indian Standards Code of Practice for Plain and Reinforced Concrete **IS 456-2000**, Design Aids for Reinforced Concrete to **IS 456:1987(SP-16),** Criteria for

Earthquake Resistant Design Structures **IS 1893-2016**, Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces **IS 13920:2016**.

The worst possible combination of design moments, shear forces, axial forces, and torsions are obtained through the computer software program **"ETABS 19.0.2,"** and a series of manual calculations are performed to ensure the accuracy and dependability of the design results suggested by the software.

1.9 Detailing

The space frame is considered as a Special Moment Resisting Frame (SMRF) with a special detailing to provide ductile behavior and comply with the requirements given in **IS 13920:2016** aided by **SP 34**.

1.10 Scope

The scope of the project work is restricted to the analysis and design aspects of the structure only, and does not extend to construction, project management or other related aspects. The design and detailing of the structural elements that have been performed are:

1. Slab
2. Beam
3. Column
4. Staircase
5. Mat Foundation

- The design and layout of building services such as pipelines, electrical appliances, sanitary and sewage systems are not included in the project.
- The design of the underground basement for parking facilities is not covered in the project.
- The project does not consider the existing soil condition of the locality.
- The bearing capacity of the soil is assumed in the project.
- The project does not take into account the environmental, social and economic conditions of the locality.

- The project work is solely related to the practical application of the studied courses in the field.

1.11 Salient features of the project

Project name "**Seismic Analysis and Design of Multistoried Hospital Building.**"

A. Physical properties:

- Location: Pokhara
- Total number story: 7+1 (Basement)
- Total height: 24.2 m
- Floor to floor height: 3.9 m
- Overall length: 66.056 m
- Overall breadth: 62.411m
- Plinth area: 4122.6506 m^2
- Earthquake zone: V

B. Structural properties:

- Total number of Column: 113
- Section of column: 750 mm x 750 mm
- Section of beam: 400 mm x 600 mm, 550 mm x 700 mm
- Depth of slab: 130 mm
- Foundation depth: 1000 mm

C. Some values adopted:

- Bearing Capacity of soil: 140 KN/m^2
- Unit weight of concrete: 25 KN/m^3
- Unit weight of masonry: 20 KN/m^3 (Including plaster)
- Live load on staircase: 5 KN/m^2
- Live load on floor: 4 KN/m^2
- Live load on roof: 1.5 KN/m^2
- Seismic Zone Factor, Z: 0.36 (Zone V)
- Response Reduction Factor, R: 5 (SMRF)
- Importance Factor, I:Bas 1.5

2 Review of literature

2.1 Background

The focus of our work is on seismic analysis and structural design of RCC framed concrete structures. Specifically, we aim to use the limit state method to obtain design output based on structural design that incorporates seismic considerations. Earthquakes are a natural phenomenon caused by the release of seismic waves (p-waves and s-waves) from the earth's surface, ranging from faint tremors to wild motions due to the sudden release of energy stored in the rocks beneath the earth's surface. Although earthquakes have been occurring since the beginning of the earth's history, our knowledge and interpretations of their behavior and ways to minimize damage are recent. While most earthquakes are minor and go unnoticed, major earthquakes can cause huge loss of life and property.

The theoretical development of earthquake forces in structures reveals that the maximum elastic response acceleration during an earthquake, for which the structure is designed, would be several times larger than the design acceleration, i.e., the seismic coefficient specified in most codes. This situation is different from the approach taken in codes for loads such as design loads, which are usually higher than the actual ones. It is based on the probability of the infrequent occurrence of large earthquakes and the energy absorption capacity of the structure.

It is assumed that the structure will respond in a nonlinear manner in severe earthquakes, thereby dissipating the energy of motion using material and structural ductility. To achieve ductile behaviors, brittle modes of failure due to shear, anchorage, and bond should be avoided. This concept is based on the philosophy that damage to the building is permissible as long as the structure does not collapse catastrophically during a severe earthquake. Therefore, the vertical load-bearing members providing the basic support of the structure should be strong, and this can be achieved by applying the strong column-weak beam concept.

2.2 Design Philosophy

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

a. Limit State Method

b. Working Stress Method
c. Ultimate Load Method

Among above, **Limit State Method** has been adopted for the design of the structural elements.

Limit State Method

Limit state design, which has its roots in ultimate or plastic design, aims to ensure an acceptable probability that a structure will not become unserviceable during its intended lifetime, that is, it will not reach a limit state. A structure must be able to withstand all loads that may act on it safely and meet the serviceability requirements, with an appropriate level of reliability. To ensure adequate safety and serviceability, all relevant limit states must be considered in the design process.

Assumptions for the limit state of collapse in flexure

a. The plane section normal to the axis of member remains plane after bending.

b. The maximum strain in concrete at the outermost compression fiber is 0.0035.

c. The relationship between the compressive stress distribution in concrete and the strain in concrete may be assumed to be rectangle, trapezoid, parabola or any other shape. For design purpose, the compressive strength of concrete in the structure shall be assumed to be 0.67 times the characteristic strength. The partial safety factor y_m =1.5 shall be applied.

d. The tensile strength of concrete is ignored.

e. The stresses in the reinforcement are derived from the representative stress-strain curve for the type of steel used. For design purpose the partial safety factor y_m =1.15 shall be applied.

f. The maximum strain in the tension reinforcement in the section at failure shall not be less than:

$$\frac{f_y}{1.15E_s} + 0.002$$

Where, f_y = characteristic strength of steel.

E_s = modulus of elasticity of steel.

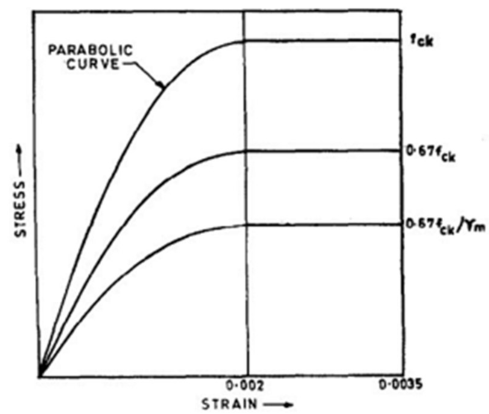


Figure 2.1 Stress-Strain curve for concrete

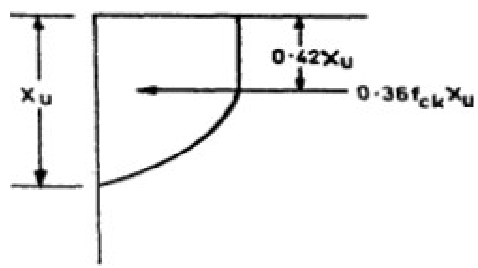


Figure 2.2 Stress block parameters

In addition to the assumptions for limit state of collapse in flexure from 1 to 5, the following shall be assumed:

a. The maximum compressive strain in concrete in axial compression is taken as 0.002.
b. The maximum compressive strain at the highly compressed extreme fiber in concrete subjected to axial compression and bending and when there is no tension on the section shall be 0.0035 minus 0.75 times the strain at least compressed extreme fiber.

The most important of these limit states which must be examined in design are as follows:

Limit state of collapse

This limit state corresponds to the maximum load carrying capacity of a structure.

Exceeding the collapse limit state means that a clearly defined limit state of structural usefulness has been exceeded, but it does not necessarily imply a complete collapse. This limit state may correspond to:

A. Flexure
B. Compression
C. Shear and
D. Torsion.

Limit state of serviceability

This limit state corresponds to excessive deformation and is used to check members where the magnitude of deformation may restrict the use of the structure or its components. This limit state may correspond to:

Control of Deflection

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

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

Control of Cracking

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

- Expression for crack width and spacing, and (Annex F of IS: 456-2000).
- Allowable crack widths under different service conditions with due considerations to corrosion and durability of concrete (clause no 35.3.2 of IS: 456-2000).
- Unless the calculation of crack widths shows that a greater spacing is

acceptable, for the flexural members in normal internal or external conditions of exposure, the maximum distance between bars in tension shall not exceed the value as given in IS:456-2000, clause no 26.3.3.

Cracks due to bending in compression member subjected to design **axial load >0.2fck*Ac**, need not be checked. For flexural members (A member which is subjected to design load <0.2fck*Ac) if greater spacing of reinforcements as given in clause 26.3.2, IS456-2000 is required, the expected crack width should be checked by formula given in Annex F of IS 456-2000.

Control of Vibration

In the design of reinforced concrete structures, the limit state concept considers the probabilistic and structural variations in material properties, loads, and safety factors. Dynamic loads, which vary in magnitude, direction, or position over time, can affect almost any RCC structural system during its lifetime. The resulting stresses or deflections in the structure, known as structural response, are also time-varying or dynamic and are expressed in terms of displacements.

2.3 Loads

Basic objective of constructing building or any structure is to support loads. There are different types of loads, which come across and have to be dealt during analysis and design of any structure.

2.3.1 Design loads

The buildings and structures are subjected to number of loads, forces and effects during their service life such as those listed in **IS: 456-17** and **IS: 875-8.1**. The following loads usually determine the size of structural element:

a) Dead load (DL)
b) Imposed load (IL)
c) Wind load (WL)
d) Earthquake load (EL)

The following are the cause which generally causes internally-equilibrated stresses forming cracks in structure, but not collapse.

a) Foundation movement,
b) Axial elastic shortening,
c) Shrinkage,
d) Temperature changes, etc.

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

a) Fatigue
b) Construction loads
c) Accidental loads
d) Impact and collision
e) Explosions
f) Fire, etc.

2.3.2 Load assessment

The proposed building is a RCC framed structure, located in Kathmandu. Thus wind loads, snow loads, and other special types of loads described by **IS: 875 (part 5):1987** can be taken as negligible as compared to the dead, live and seismic loads.

Dead Loads

According to the **IS 875:1987(Part I)**, the dead load in a building shall comprise the weights of all walls, partitions, beam, column, floors and roofs and shall include the weights of all other permanent features in the building.

Live Loads

It means the load assumed or known resulting from the occupancy or use of a building and includes the load on balustrades and loads from movable goods, machinery and plant that are not an integral part of the building. These are to be chosen from codes as **IS:875: 1987(Part 2)** for various occupancies where required. These codes permit certain modifications in the load intensities where large contributory areas are involved, or when the building consists of many stories.

Eccentricity of vertical loads

When transferring the loads from parapets, partition walls, cladding walls and façade walls etc. to the supporting beams or columns, the eccentricity with these loads should be properly considered in the case of rigid frames of reinforce concrete. Such eccentricities will produce externally-applied joint moments similar to these arising from projecting cantilevers and these should be included in frame design.

Seismic Loads

These are the load resulting from the vibration of the ground underneath the super-structure during an earthquake. The earthquake is an unpredictable natural phenomenon. Nobody knows the exact timing and magnitude of such loads. Seismic loads are to be determined essentially to produce an earthquake resistant design.

Since the probable maximum earthquake occurrence is not frequent, designing building for such earthquake isn't practical as well as economically prudent. Instead, reliance is placed on kinetic dissipation in the structure through plastic deformation of elements and joints and the design forces are reduced accordingly. Thus, the philosophy of seismic design is to obtain a no-collapse structure rather than no-damage structure.

SEISMIC DESIGN CRITERIA

Earthquake	Desired Behavior	Controlling Parameter
Minor	No damage to non-structural components.	Controlling deflection by providing stiffness.
Moderate	No significant structural damage, minor cracks in beams and column should be pre-dominantly elastic.	Avoid yielding of members or permanent damage by providing strength.
Severe Catastrophic	No collapse of system which could cause loss of life.	Allow structure to enter into inelastic range and absorb energy by providing ductility.

Table 2.1 Seismic design criteria

An earthquake-resistant building has four virtues in it, namely:

I. Good Structural Configuration:

Its size, shape and structural system carrying loads are such that they ensure a direct and smooth flow of inertia forces to the ground.

II. Lateral Strength:

The maximum lateral (horizontal) force that it can resist is such that the damage induced in it does not result in collapse.

III. Adequate Stiffness:

Its lateral load resisting system is such that the earthquake-induced deformations in it do not damage its contents under low-to-moderate shaking.

IV. Good Ductility:

Its capacity to undergo large deformations under severe earthquake shaking even after yielding, is improved by favorable design and detailing strategies. Seismic codes cover all these aspects.

2.3.3 Estimation of loads

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

I. Dead loads

1. Calculate the weight of those elements of building whose dimensions are fixed already from functional considerations and can be worked out carefully. These are generally non-structural elements and of parapets, rooftop, railings etc.
2. From pre-design, calculate weight of structural elements such as beam, column, slab etc.
3. Put all loads systematically on sketches, say plan wise, showing their gravity lines with reference to column center-lines.

II. Live loads

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

III. Seismic or earthquake loads

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

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

a. **Seismic coefficient method**
b. **Response spectrum method**

The seismic coefficient method or static method is generally applicable to building up to 40m in height and those are more are less symmetrical in plan and elevation.

IV. Wind Load

Wind load calculation

Design wind speed (V,) - The basic wind speed (V,) for any site shall be obtained from Table and shall be modified to include the following effects to get design wind velocity at any height (V,) for the chosen structure:

a) Risk level;
b) Terrain roughness, height and size of structure; and
c) Local topography.

It can be mathematically expressed as follows:

$$V_z = V_b k_1 k_2 k_3$$

where V_z = design wind speed at any height z in m/s;

k_1 = probability factor (risk coefficient) *(IS 875 part III see 5.3.1);*

k_2 = terrain, height and structure size factor (*IS 875 part III* see 5.3.2); and

k_3 = topography factor *(IS 875 part III see 5.3.3).*

Design wind speed up to 10 m height from mean ground level shall be considered constant.

For Pokhara, basic wind speed V_b = 47m/s

For 50 years design life of structures k_1 = 1

For k2, Terrain with numerous closely spaced obstructions having the size of building-structures up to 10 m in height with or without a few isolated tall structures, so it falls in Category 3. For 20m to 50m vertical dimension, it lies in Class B. k_2 = 1.04

For low inclination of area k_3 = 1

Design mean speed $\left(V_z\right) = V_b k_1 k_2 k_3$ = 47 * 1* 1.04* 1 = 48.88 m/s

The reaction of truss obtained at columns at ETABS, has been manually added back as point loads from hinge joint at top of columns.

2.4 TERMINOLOGY

Response spectra:

Response spectrum is a representation of the maximum response of an idealized single degree of freedom system subjected to a given earthquake, with a certain period of vibration and damping. It shows the maximum response, such as the maximum absolute acceleration, maximum relative velocity or maximum relative displacement, of the system plotted against the damped natural period for various damping values. Seismic analysis can be carried out using the design spectrum provided in the figure below, which is based on strong motion records of eight earthquakes in India.

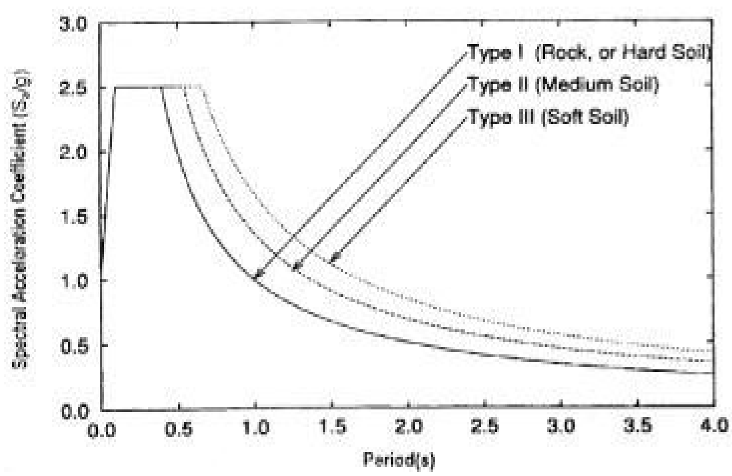


Figure 2.3 Response spectra for rock and soils for 5% damping

An elastic response spectrum has been proposed for Maximum Considered Earthquake (MCE), which is divided by factor 2 to get Design Basis Earthquake (DBE) and again by factor R to get inelastic response spectra. MCE is a very rare event which has a 10% probability of being exceeded in 100 years. DBE is that earthquake which has reasonably been expected to occur at least once during design life of structure and has a 10% probability of being exceeded in 50 years. The intention is to let the designer know about the whole scenario of elastic and inelastic response spectra and also to know the need of providing ductility in structure.

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

Base shear $(\boldsymbol{Vb}) = \boldsymbol{Ah} * \boldsymbol{W}$

where,

A_h=Design horizontal acceleration spectrum.

W= Seismic weight of building

$$\boldsymbol{A_h = \frac{ZISa}{2Rg}}$$

where,

Z = Zone factor, From Table clause 6.4.2

I = Importance factor, Table 6 s1 No. 1(i), clause 6.4.2

R = Response reduction factor

$(\frac{Sa}{g})$ = Structural response factor

The fundamental time period of the vibration,

- **Clause 7.6.1 IS 1893:2016**

  $T_a = 0.075 * h^{0.75}$ (Assuming no brick infill faces)

- **Clause 7.6.2 IS 1893:2016**

  $T_a = \frac{0.09*h}{\sqrt{d}}$

where,

T_a = Fundamental natural time

h=Height of building in meters.

d = Base dimensions of the building at the plinth level in meter along considered direction of the lateral force

Response reduction factor:

The response reduction factor is determined by considering the energy absorption capacity of a structural system through damping and inelastic action due to load reversals. The assigned factor is based on design and construction experience, as well as the performance of the structure during earthquakes. The value of R reflects the degree of continuity and ductility provided in the structural system. A value of 1.0 indicates little or no ductility, while a value greater than 1.0 implies the ability to undergo inelastic cyclic deformation. For RCC structures, damping is usually taken as 5% of critical damping.

The value of R is taken as 5 for RCC moment resisting frame specially designed to provide ductile behavior and comply with requirements given in IS:13920-2016.

Number of modes:

The number of modes to be considered in the response spectrum analysis should be such that at least 90% of the seismic mass of the structure gets excited in each of the principal horizontal directions.

Closely spaced modes:

Closely spaced modes of structure are those of its natural modes of vibration whose natural frequencies differ from each other by 10 percent or less of the lower frequency

i.e.

$$\frac{\omega j - \omega i}{\omega i} \leq 0.1$$

where,

ω_j= any frequency of modes

ω_i= lower frequency of the mode.

Modes failing to fulfill above criteria are **widely spaced modes**.

Story drift:

The relative inter-story horizontal displacement is referred to as story drift. A limitation on story drift is necessary to avoid discomfort to occupants of the building and to save nonstructural elements from damage. A drift limitation of **0.004 times or (0. 4%) the story height** in the elastic range is imposed by IS: 1893:2016

Regularity:

Regular structures have a uniform configuration and lateral force resisting system without any significant physical discontinuities in plan or vertical configuration. On the other hand, irregular structures have significant physical discontinuities in their configuration or lateral force resisting system, which may include plan irregularity, vertical irregularity, or mass irregularity. Past earthquakes have demonstrated that irregular structures are more susceptible to damage compared to regular structures.

2.5 Load Combinations

The analysis was performed for various 26 combinations and time history separately. Following are those 26 combinations as suggested by IS: 1893-2016, Clause 6.3.1.2

i.1.5 (DL + IL)

ii.1.5 DL

iii.1.2 (DL + IL ± ELx)

iv. 1.2 (DL + IL ± ELy)

v.1.2 (DL + IL ± RSx)

vi.1.2 (DL + IL ±RSy)

vii.1.5 (DL ±ELx)

viii.1.5 (DL ± ELy)

ix.1.5 (DL ± RSx)

x.1.5 (DL ± RSy)

xi.0.9DL ± 1.5 ELx

xii.0.9DL ± 1.5 Ely

xiii.0.9DL ± 1.5 RSy

xiv.0.9DL ± 1.5 RSx

To consider eccentricity in building, additional combinations are defined by replacing EL with EL±e and RS with RS±e.

3 Methodology and Preliminary design

3.1 Structural System

A structure consists of both load-carrying elements, such as beams and columns, and non-structural elements like partitions, false ceilings, and doors. These load-carrying elements are combined to form the structural system, which is designed to withstand the effects of gravitational and environmental loads and transmit resulting forces to the ground while maintaining the structure's geometry, integrity, and serviceability.

3.2 Structural Arrangement Plan

The planning of the building has been done by the group. The final plan of the building was a result of review of various codes of practice and some other reference books suggested by our supervisor. The positioning of the columns and staircases was carefully considered to ensure an aesthetically pleasing, functional, and cost-effective design, with beam arrangements carried out accordingly.

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

The methodology adopted in the seismic analysis and structural analysis/design of 8-storied RCC framed apartment is described below.

3.3 Data Collection

All the data required for this project work have taken from different literatures. Design parameters have been taken from **Indian Standard code of practices**.

3.4 Load Calculation

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

3.4.1 Vertical load

a. Loads on slab are:
   a) Dead load
   b) Live load
b. Load on Beam are:
   a) Self-weight of beam
   b) Load transferred from slab

The **finite element method** can be used to analyze slabs of any shape, boundary condition and subject to any loading. This method can also account for stiffness of the supporting beams. This method is extremely useful for slabs with opening and those subjected to concentrated loads.

Two-way slabs can also be analyzed using the ultimate load theory, **Yield line theory** is the most popular. In this theory, the strength of slab is assumed to be governed by

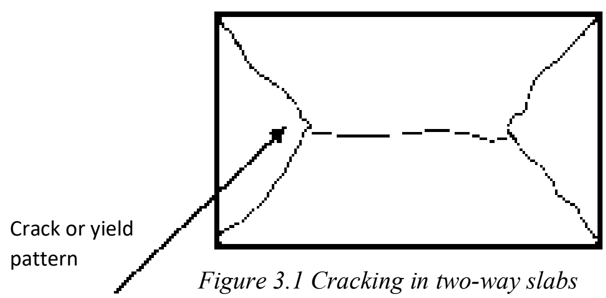


Figure 3.1 Cracking in two-way slabs

flexure alone. The effects of shear and deflection are to be considered separately. It is assumed that a mechanism is formed in the slab at failure. The reinforcing steel is assumed to have fully yielded along the yield lines or cracks at failure.

3.4.2 Lateral load

Here, considered lateral load is from earthquake only. Seismic load of building or structure depends upon its geographic location, lateral stiffness, soil upon which it is erected and it is reversible. Thus, this effect has been considered along both axes of the building taken one at the time.

3.5 Preliminary Design

The following remarks will be helpful in choosing the sections;

1. Too many variations in the sizes of beam and columns, width and depth are not desirable from both aesthetic and economical point of view. Minimum dimensions of 200mm for small spans and 230mm~ 300mm for large spans may be set for structural members.
2. Richer concrete mixes can be used in lower story elements to avoid frequent change in sections. Some size variation can also be avoided by reducing column steel upwards in building.
3. Frequently column steel may be at odds with the longitudinal steel of beams crossing it from one or more directions. Also cover required differs. It may be useful to keep column wider than the beam and the number of bars be kept even in column and odd in beam or vice-versa so that bars pass uninterruptedly.
4. Narrow-deep beams may show shrinkage, temperature cracking in web and also lateral buckling if laterally unsupported. This should be considered in surface reinforcement detailing and ensuring lateral support on the compression face at less than 25*b, b being beam breadth, where the effective depth of beam exceeds 3 times of b.

At the preliminary design stage, calculation of reinforcement may be excessive, but it will be good to know the maximum steel required to check that it lies within a reasonable percentage of the concrete section and can be located in it without congestion.

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

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

3.5.1 Preliminary Design of RCC Slab Element

The preliminary design of RCC slab for the floor and roof of the proposed building is based on fulfillment of deflection control criteria of IS 456:2000 and behavior of floor slab as a rigid diaphragm in earthquake resistant design based on IS 1893:2016.

The project building has largest span of 6750 mm X 7250 mm.

l_x = 6750 mm (i.e., the smallest of the two dimensions of the slab)

l_y = 7250 mm (i.e., the largest of the two dimensions of the slab)

$$\frac{ly}{lx} = \frac{6750}{7250} = 1.074 < 2$$

Hence the slab is two-way continuous slab.

From IS 456:2000 Clause 23.2

Control of Deflection, we can draw out the following methodical steps to get the thickness of slab. Here d represents the thickness of the slab

$$\frac{span(l)}{effective\ dept\ \ (d)} \leq \alpha\,ß\,\gamma\,\delta\,\lambda$$

Here, l= lx= 6750 mm

α= 26 **(Ref. IS 456: 2000 Cl. 24.1 Note2)**

(For continuous slab)

ß = span factor =1 for largest span less than 10m **(Ref. IS 456: 2000 Cl. 23.2.1 b.)**

Assuming 0.2% reinforcement and fs = 290 N/mm^2

From **Fig. 4, (Page 38) IS 456: 2000**

γ = 1.38

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

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

$d_{eff} = \frac{6750}{1*1*1*1.347*26}$ = 192.85 mm > 150mm

(Slab depth > 150mm is considered uneconomical for large area commercial buildings.)

Add secondary beam dividing slab to two equal halves.

After:

l_x = 6750 mm (i.e., the smallest of the two dimensions of the slab)

l_y = 3625 mm (i.e., the largest of the two dimensions of the slab)

$$\frac{ly}{lx} = \frac{6553.2}{4521.2} = 1.862 < 2$$

Hence the slab is two-way continuous slab.

From IS 456:2000 Clause 23.2

Here, l= lx= 3625 mm

α = 26 **(Ref. IS 456: 2000 Cl. 24.1 Note2)**

(For continuous slab)

ß = Span factor =1 for largest span less than 10m **(Ref. IS 456: 2000 Cl. 23.2.1 b.)**

Assuming 0.2% reinforcement and fs = 290 N/mm^2

From **Fig. 4, (Page 38) IS 456: 2000**

γ = 1.346

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

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

$d_{eff} = \frac{3625}{1*1*1*1.346*26}$ = 103.5 mm ~ 110 mm

D = 110+ 0.5*10+ 15 = 130 mm (too close to upper maximum, so we prefer using secondary beam across as well)

Secondary Beam will be required for all slab with span in shorter direction greater than = 26* 1.346* 110 mm = 3850 mm

3.5.2 Preliminary Design of RCC Beam Element

CENTRE MAXIMUM SPAN:

A) GROUND FLOOR

From the architectural plan of the proposed building, the maximum span length of the beam is **7250 mm** c/c at III-D to III-C.

Maximum span 7250 mm c/c at III-D to III-C is taken. Preliminary Design can be done either by

1) Deflection Control Criteria
2) By Moment Criteria

1. Design by deflection control criteria

The methodical steps in designing the beam element preliminarily with respect to IS 456:2000, IS 1893, IS13920, IS 4326 are same as the slab element.

From **Clause 23.2IS 456:2000** we can draw out the following methodical steps to get minimum depth of beam;

$$\frac{span(l)}{effectivedep \quad (d)} \leq \alpha\ ß\ \gamma\ \delta\ \lambda$$

where,

Span (l)= 7250 mm (i.e., the largest available span length from the floor plan.)

$d_{eff} = \frac{7250}{15} = 483.33$ mm ~ 490 mm

A minimum clear cover of 40 mm is taken for main bar of diameter 20 mm, then we get the overall depth of beam as:

Overall Depth of Beam (D) = Effective Depth + 0.5x Dia. of main bar + Clear Cover

= 490 + 0.5 X 20 + 40

= 540 mm

Therefore, take D = 540 mm

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

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

Therefore b = $\frac{540}{1.5}$ = 360 mm

D = 540 mm

b = 360 mm

2. Determination of depth of beam by moment criteria

Area of Influence = 26.156 m^2

Load Calculations:

Dead Load of Slab = 25*0.125*26.156 = 85.008 KN

Dead Load of Beam = 26.75KN

Live Load (LL) = 4 KN/m^2

Floor Finish (FP) = 1 KN/m^2

LL + FF on influence area= (4+1) *26.156= 130.78KN

Total Load = Sum of above loads = 242.538 kN

Factored Load= 1.5* Total Load=363.807 KN

Factored UDL=50.18 KN/m

Secondary Beam depth = 75% of original beam = 400mm

Load of secondary beam = 0.27*(0.4-0.13)*6.75*25= 12.30 kN

Factored dead load of secondary beam = 18.45 KN

Max Moment (Mu) = $\frac{wl*l}{10} + \frac{w\prime l}{4} =$ = 297.199 KN-m

Moment of Resistance (Mu) =0.133 f_{ck}*b*d^2

Mu = 0.133* fck* b* d^2

Take b = d/1.5

F_{ck} = 30Mpa

Thus d_{eff} = 511.8184mm

Overall Depth of Slab (D) = Effective Depth + 0.5x Dia. of main bar + Clear Cover

$= d_{eff} + \frac{\varnothing}{2}$ + clear cover

= 544 mm

D = 560 mm

Size of Beam = (560*375) mm^2

Size of secondary Beam = (420*285) mm^2

But adopting for safety,

Size of Beam = (600*400) mm^2

Size of secondary Beam = (450*300) mm^2

3.5.3 Preliminary Design of RCC column element

From our plan, we chose column B3 which has maximum influence area of 45.56 m^2 excluding the assumed area of column (0.75m x 0.75m). It is the intermediate column and carries maximum axial load. The column of basement is designed, and it has the maximum size among all. It contains maximum percentage of steel reinforcement. We have considered maximum 3% of steel reinforcement. We consider M30 grade mix and Fe500 bars.

Load Calculations:

Dead Load (DL)

<u>Dead load of slab</u>

Formula: γ_c*D*area (γ_c=25KN/m^3, IS 875 part I – 1987 Table 1,22)

D=0.13m

Floors	No. of floors	Area (m^2)
Basement, Ground, First, Second, Third, Fourth, Top	7	45.56
	Total	318.92

Total load = 25*0.13*318.92 = 1036.49 KN

Dead load of beam

Formula: γ_c*B*D*length*no. of stories

Total load = [25*0.375*(0.56-0.13) *6.75*7] *2 = 380.953KN

Dead load of Secondary beam

Formula: γ_c*B*D*length

Total load = [25*0.285*(0.42-0.13) *6750*7] *2 = 195.260625KN

Dead load of column

Formula: γc*area*height

Total load = 329.062 KN

Floor finish

Formula: 1 KN/m^2 *area

Total load = 318.92 KN

Masonry wall load

Formula: g_m*area *ht (g_m=19KN/m^3)

Wall load = 19.6*0.7*0.23*3.9*3.9

=479.967 KN

Live Load

Floor	Area (m^2)	Factor	Description
Basement	45.56	5	Parking
Ground	45.56	4	Lobby
First	45.56	2	Wards
Second	45.56	2	Wards
Third	45.56	2	Wards
Fourth	45.56	3	Library
Top	45.56	4	Balcony

Total Live Load = 956.76 KN

Total load on column = Total Dead Load+ Total Live Load = 3697.4126 KN

Factored Load (Pu) = 1.5*Total load on column

=1.5*3697.4126

=5546.1189 KN

For axially loaded short column (Ref. IS 456: 2000 Cl. 39.3)

Pu= 0.4fck*Ac+ 0.67*f_y*A_{st}

5546.1189 = 0.4*30*0.97*Ag+0.67*500*0.02*Ag

(% steel = 2%, f_{ck} = 30MPa, f_y=500 KN/m^2)

Solving for Ag, Ag= 336127.455 mm^2

D = $(Ag)^{0.5}$= 579.765 mm

Hence, for safety adopting **750*750** column size.

3.6 Base Shear

3.6.1 BLOCK-I

SEISMIC ANALYSIS

UNIT WEIGHTS FOR DEAD LOAD **<u>(Ref. IS: 875 – Part I, Table 1)</u>**
Unit weight of brick masonry (Table 1, 36) =19 KN/m^3
Unit weight of RCC (Table 1, 22) =25 KN/m^3

UNIT WEIGHT FOR LIVE LOAD **<u>(Ref. IS: 875 – Part II, Table 1, 1)</u>**
Unit weight for live load = 3 KN/ m^2 (General Floor)
Unit weight for live load = 4 KN/ m^2 (Staircase)
Unit weight for live load = 0.75 KN/ m^2 (Terrace)

<u>Calculation/ Analysis</u>

A. <u>SLAB AREA CALCULATION</u>

First Floor Area= 569.18 m^2
Second Floor Area= 569.518 m^2
Third Floor Area= 569.518 m^2
Fourth Floor Area= 569.518 m^2
Top Floor Area= 569.518 m^2
Roof Floor Area= 212.614 m^2

B. **<u>SLAB LOAD CALCULATION</u>**

$Y_c = 25\ KN/m^3$ *(***<u>IS 875 Part I – 1987 Table 1</u>***)*

Slab load = Y_c*slab area (A)*slab thickness (D)
First Floor Load= 1850.934 KN
Second Floor Load= 1850.934 KN
Third Floor Load= 1850.934 KN
Fourth Floor Load= 1850.934 KN
Top Floor Load= 1850.934 KN
Roof Floor Load= 690.9939 KN

C. <u>COLUMN LOAD CALCULATION</u>

$Y_c = 25\ kN/m^3$ **<u>(IS 875 Part I – 1987 Table 1)</u>**

Column load for each floor = N * Y_C * h * A_{column}
where, N = No. of column in each Floor = 21 nos.
No. of column in top floor=10 nos.
h = column height of 1^{st} ,2^{nd} ,3^{rd} floor= 3.9 m
column height of 4^{th} floor= 4 m
column height of top floor= 3.6 m
A = Area of the column = 600mm * 600mm

First Floor = 1151.71875 KN
Second Floor = 1151.71875 KN
Third Floor = 1151.71875 KN
Fourth Floor = 1166.48438 KN

Top Floor = 843.75 KN
Roof Floor = 253.125 KN

D. **PRIMARY BEAM LENGTH CALCULATION**

First Floor = 209.246 m
Second Floor = 209.46 m
Third Floor = 209.46 m
Fourth Floor = 209.46 m
Top Floor = 209.46 m
Roof Floor = 87.746 m

E. **PRIMARY BEAM LOAD CALCULATION**

$Y_c = 25\ kN/m^3$ **(IS 875 Part I – 1987 Table I)**
Primary Beam load for each floor = Yc * Area of beam * Length of beam
Where, Area = B *D = 375mm * 560mm

First Floor = 1098.5415 KN
Second Floor = 1098.5415 KN
Third Floor = 1098.5415 KN
Fourth Floor = 1098.5415 KN
Top Floor = 1098.5415 KN
Roof Floor = 460.6665 KN

F. **SECONDARY BEAM LENGTH CALCULATION**

First Floor = 134.997 m
Second Floor = 134.997 m
Third Floor = 134.997 m
Fourth Floor = 134.997 m
Top Floor = 134.997 m
Roof Floor = 53.998 m

G. **SECONDARY BEAM LOAD CALCULATION**

$Y_c = 25\ kN/m^3$ **(IS 875 Part I – 1987 Table I)**

Secondary Beam load (each floor) = Yc *Area of beam *Length of beam
where, Area = B *D = 285mm * 420mm

First Floor = 403.9785 KN
Second Floor = 403.9785 KN
Third Floor = 403.9785 KN
Fourth Floor = 403.9785 KN
Top Floor = 403.9785 KN
Roof Floor = 161.589 KN

H. **MASONRY WALL LENGTH CALCULATION**

Exterior Masonry Wall Length:
First Floor = 242.6 m
Second Floor = 242.6 m
Third Floor = 242.6 m
Fourth Floor = 242.6 m

Top Floor = 242.6 m
Roof Floor = 46.5 m

Parapet Wall Length:
Top floor = 126 m
Roof = 42 m

I. MASONRY WALL LOAD CALCULATION

$\Upsilon_{masonry} = 19\ KN/m^3$ **(IS 875 Part I – 1987 Table I)**

Exterior wall thickness = 0.23 m
Wall load = $\Upsilon_{masonry}$* length* thickness* height* (100 – 35) %
Where, Height of wall = (3.9-0.13-0.56)
= 3.21 m
(35 % area is neglected taking into the consideration of void)

Exterior Masonry Wall Load:
First Floor = 2212.03 KN
Second Floor = 2212.03 KN
Third Floor = 2212.03 KN
Fourth Floor = 2212.03 KN
Top Floor = 1318.01 KN
Roof Floor= 211.994 KN

Parapet Wall Load:
Top Floor = 495.338 KN
Roof Floor = 165.113 KN

J. FLOOR FINISH LOAD CALCULATION

Thickness of FF = 0.04 m
Unit weight of Floor Finish = 25 KN/m^3
Now,
W_{ff} = Unit weight of Floor Finish* slab area* Thickness of FF

First Floor = 569.51825 KN
Second Floor = 569.51825 KN
Third Floor = 569.51825 KN
Fourth Floor = 569.51825 KN
Top Floor = 569.51825 KN
Roof Floor = 212.6135 KN

K. LIVE LOAD CALCULATION

(IS 1893 Part I: 2016 Cl. 7.3.1 Table 8)

Live load = Live load intensity* Slab area* Live load Reduction Factor
LL Reduction Factor = 50 % (**if >3 KN/m^3**)
= 25% (**if <= 3 KN/m^3**

First Floor = 577.1096 KN
Second Floor = 577.1096 KN
Third Floor = 554.3284 KN
Fourth Floor = 599.8957 KN
Top Floor = 435.2019 KN
Roof Floor = 39.8650 KN

L. SHEAR WALL LOAD CALCULATION:

First Floor = 1346.42 KN
Second Floor= 690.384 KN
Third Floor = 690.384 KN
Fourth Floor=690.384 KN
Top Floor = 701.498 KN
Roof Floor = 491.991 KN

M. GLASS WALL LOAD CALCULATION:

First Floor = 61.9796 KN
Second Floor= 61.9796 KN
Third Floor = 61.9796 KN
Fourth Floor=68.0716 KN
Top Floor = 37.0818KN
Roof Floor = 0 KN

L. LUMPED LOAD CALCULATION

Lumped load = $W_{\text{Slab}} + W_{\text{Column}} + W_{\text{Primary beam}} + W_{\text{Secondary beam}} + W_{\text{Masonry}} + W_{\text{FF}} + W_{\text{Live Load}} + W_{\text{shear wall}} + W_{\text{staircase}}$

Now,

First Floor Load
=1850.934+1151.71875+1098.5415+403.9785+2212.03+569.51825+577.1096+1346.42+61.9796
=9272.2302 KN

Second Floor Load
=1850.934+1151.71875+1098.5415+403.9785+2212.03+569.51825+577.1096+1346.42+61.9796
= 9272.2302 KN

Third Floor Load
=1850.934+1151.71875+1098.5415+403.9785+2212.03+569.51825+554.3284+1346.42+61.9796
= 9249.449 KN

Fourth Floor Load
=1850.934+1166.48438+1098.5415+403.9785+2212.03+569.51825+599.8957+690.384+68.0716
= 8659.83793 KN

Top Floor Load
=1850.934+843.75+1098.5415+403.9785+1318.01+495.338+569.51825+435.2019+701.498+37.0818

= 7753.85195 KN

Roof Floor Load
=690.9939+253.125+460.6665+161.589+211.994+165.113+212.6135+39.865 0+491.991+0
=2687.9509 KN

M. TOTAL LUMP LOAD CALCULATION

Total lump load (W) = $W_1 + W_2 + W_3 + W_4 + W_5 + W_6 + W_7$
=9272.2302+9272.2302+9249.449+8659.83793+7753.85195+2687.9509
= 46895.55KN

Base Shear Calculation

According to **IS 1893 (Part I): 2016 Cl. No. 6.4.2** the design horizontal seismic coefficient A_h for a structure shall be determined by the following expression:

$$A_h = \frac{Z\,I\,S_a}{2\,R\,g}$$

Where,

Z = Zone factor **[IS 1893 (Part I): 2016 Table 2]**

For Zone V, Z = 0.36

I = Importance Factor, depending upon the functional use of the structure. [**IS 1893 (Part I): 2016 Table 6]**

= 1.5

R = Response reduction factor [**IS 1893 (Part I): 2016 Table 7**]

= 5.0 (SMRF)

S_a/g = Average response acceleration coefficient which depends on Fundamental natural period of vibration (T_a).

According to **IS 1893 (Part I): 2016 Cl. No. 7.6**

$T_a = (0.075h^{0.75}$)/((Aw)^0.5)sec

where, h = height of building in m, h = 23.2 m

Therefore, Aw=0.9765

T_a= 0.8023 sec

For T_a = 0.8023 sec and medium soil type Sa/g = 1.36/0.8023= 1.695

Now,

Ah=0.09153

According to **<u>IS 1893 (Part I): 2016 Cl. No. 7.5.3</u>** the total design lateral force or design seismic base shear (V_B) along any principle direction is given by

$V_B = A_h$ x Wdd

Where, W = Seismic weight of the building

= 46895.55 KN

Hence, $\mathbf{V_B}$ = 0.09153*46895.55

= 4292.3497 KN

According to **<u>IS 1893 (Part I): 2016 Cl. No. 7.7.1</u>** the design base shear (V_B) computed above shall be distributed along the height of the building as per the following expression:

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$ (For relatively flexible structure)

Where,

V_B= Base Shear = 4292.3497 KN

Q_i = Design lateral force at floor i

W_i = Seismic weight of floor i

h_i = Height of floor I measured from base

n = No. of story in the building

Lateral Force Distribution					
Story	**W_i (kN)**	**H_i (m)**	$\boldsymbol{W_i h_i^2}$	$\boldsymbol{\frac{W_i h_i^2}{\sum W_i h_i^2}}$	**Q_i (kN)**
Ground	9272.2302	3.9	141030.6213	0.016583	71.18
First	9272.2302	7.8	564122.4854	0.066334	284.729
Second	9249.449	11.7	1266157.074	0.148888	639.079
Third	8659.8379	15.6	2107458.151	0.247812	1063.696
Fourth	7753.85195	19.6	2978719.765	0.350262	1503.447
Roof	2687.9509	23.2	1446762.692	0.170122	730.223
		Sum	**8504250.789**		**4292.354**

Table 3.1 Table for lateral load calculation of Block I

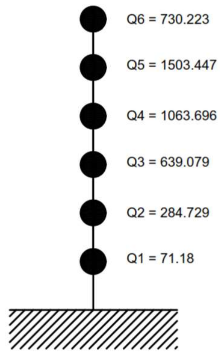


Figure 3.2 Visualization of Qi for Block I

3.6.2 BLOCK -II

SEISMIC ANALYSIS

UNIT WEIGHTS FOR DEAD LOAD **(Ref. IS: 875 – Part I, Table 1)**
Unit weight of brick masonry (Table 1, 36) =19 KN/m^3
Unit weight of RCC (Table 1, 22) =25 KN/m^3

UNIT WEIGHT FOR LIVE LOAD **(Ref. IS: 875 – Part II, Table 1, 1)**
Unit weight for live load is taken according to the occupancy classification
For Staircase, Unit weight for live load = 4 KN/m^2
For Terrace, Unit weight for live load = 0.75 KN/m^2

Calculation/ Analysis

A. SLAB AREA CALCULATION

Ground Floor Area = 1115.639 m^2
First Floor Area = 1115.639 m^2
Second Floor Area = 1056.155 m^2
Third Floor Area = 1056.155 m^2
Fourth Floor Area = 1056.155 m^2
Top Floor Area = 1010.593 m^2
Roof Floor Area = 227.813 m^2

B. SLAB LOAD CALCULATION

Y_c = 25 KN/ m^3 (**IS 875 Part I – 1987 Table 1**)

Slab load = Y_c*slab area (A)*slab thickness (D)
Ground Floor Load = 3625.828 kN
First Floor Load = 3625.828 kN
Second Floor Load = 3432.504 kN
Third Floor Load = 3432.504 kN

Fourth Floor Load = 3432.504 kN
Top Floor Load = 2505.307 kN
Roof Floor Load = 740.391 kN

C. COLUMN LOAD CALCULATION

$Y_c = 25\ kN/\mathrm{m}^3$ **(IS 875 Part I – 1987 Table 1)**

Column load for each floor = N * Y_c * h * A_{column}
where, N = No. of column in each Floor
h = column height = 3.9 m
A = Area of the column = 750mm * 750mm

Ground Floor = 1069.453 KN
First Floor = 2084.063 KN
Second Floor = 2029.219 KN
Third Floor = 2029.219 KN
Fourth Floor = 2084.063 KN
Top Floor = 1316.250 KN
Roof Floor = 301.641 KN (Half Column)

D. PRIMARY BEAM LOAD CALCULATION

$Y_c = 25\ kN/\mathrm{m}^3$ **(IS 875 Part I – 1987 Table I)**

Primary Beam load for each floor = Yc * Area of beam * Length of beam
Where, Area = B *D = 400mm * 550mm

Ground Floor = 2161.688 KN
First Floor = 2161.688 KN
Second Floor = 2055.375 KN
Third Floor = 2055.375 KN
Fourth Floor = 2055.375 KN
Top Floor = 1701.000 KN
Roof Floor = 531.563 KN

E. SECONDARY BEAM LOAD CALCULATION

$Y_c = 25\ KN/\mathrm{m}^3$ **(IS 875 Part I – 1987 Table I)**

Secondary Beam load (each floor) = Yc *Area of beam *Length of beam
Where, Area = B *D = 250mm * 400mm

Ground Floor = 955.356 KN
First Floor = 955.356 KN
Second Floor = 914.957 KN
Third Floor = 914.957 KN
Fourth Floor = 914.957 KN
Top Floor = 632.166 KN
Roof Floor = 201.994 KN

F. MASONRY WALL LOAD CALCULATION

$Y_{masonry} = 19\ KN/\mathrm{m}^3$ **(IS 875 Part I – 1987 Table I)**

Wall thickness = 0.23 m
Wall load = $Y_{masonry}$* length* thickness* height* (100 – 35) %
(35 % area is neglected taking into the consideration of void)

Ground Floor = 3069.650 KN
First Floor = 6139.300 KN
Second Floor = 6139.300 KN
Third Floor = 6139.300 KN
Fourth Floor = 4039.661 KN
Top Floor = 1269.484 KN
Roof Floor = 552.608 KN

G. FLOOR FINISH LOAD CALCULATION

Thickness of FF = 0.04 m
Unit weight of Floor Finish = 25 KN/ m^3
Now,
W_{ff} = Unit weight of Floor Finish* slab area* Thickness of FF

Ground Floor Area = 1115.639 kN
First Floor Area = 1115.639 kN
Second Floor Area = 1056.155 kN
Third Floor Area = 1056.155 kN
Fourth Floor Area = 1056.155 kN
Top Floor Area = 1010.593 kN
Roof Floor Area = 227.813 kN

H. LIVE LOAD CALCULATION

(IS 1893 Part I: 2016 Cl. 7.3.1 Table 8)

Live load = Live load intensity* Slab area* Live load Reduction Factor
Where, Live load intensity = 3 kN/m^2 (General Floor)
=0.75 kN/m^2 (terrace Floor without access)
= 1.5 kN/m^2 (Terrace Floor with access)
LL Reduction Factor = 50 % (**if >3 KN/ m^3**)
= 25% (**if <= 3 KN/ m^3**)

Ground Floor = 1143.369 KN
First Floor = 1143.369 KN
Second Floor = 1110.146 KN
Third Floor = 1110.146 KN
Fourth Floor = 1095.908 KN
Top Floor = 188.1045 KN
Roof Floor = 51.258 KN

I. SHEAR WALL LOAD CALCULATION

$Y_{shear\ wall} = 25\ KN/\ m^3$ **(IS 875 Part I – 1987 Table I)**

Shear wall load = $Y_{shear\ wall}$* Area* height
where, area is taken from ETABS
Height = 1.95m for ground & roof floor
= 3.9m for others

Ground Floor = 391.463 KN
First Floor = 782.925 KN
Second Floor = 782.925 KN
Third Floor = 782.925 KN
Fourth Floor = 782.925 KN
Top Floor = 782.925 KN
Roof Floor = 391.463 KN

J. STAIRCASE LOAD

$Y_c = 25\ kN/\ m^3$ (**IS 875 Part I – 1987 Table I)**

Live Load Intensity = 4 KN/m^2 (**IS 1893 Part I: 2016 Cl. 7.3.1 Table 8**)
Live Load Reduction Factor = 50% **(>3 KN/ m^3**)

Ground Floor = 126.824 KN
First Floor = 253.648 KN
Second Floor = 253.648 KN
Third Floor = 253.648 KN
Fourth Floor = 253.648 KN
Fifth Floor = 126.824 KN
Sixth Floor = 0

K. **LUMPED LOAD CALCULATION**

Lumped load = $W_{Slab} + W_{Column} + W_{Primary\ beam} + W_{Secondary\ beam} + W_{Masonry} + W_{FF} + W_{Live\ Load} + W_{shear\ wall} + W_{staircase}$

Now,
Ground Floor Load = 13659.269 kN
First Floor Load = 18261.815 kN
Second Load = 17774.229 kN
Third Floor Load = 17774.229 kN
Fourth Floor Load = 15715.196 kN
Top Floor Load = 9532.653 kN
Roof Floor Load = 2998.729 kN

L. TOTAL LUMP LOAD CALCULATION

Total lump load (W) = $W_1 + W_2 + W_3 + W_4 + W_5 + W_6 + W_7$
= 95716.121 kN

Base Shear Calculation

According to **IS 1893 (Part I): 2016 Cl. No. 6.4.2** the design horizontal seismic coefficient A_h for a structure shall be determined by the following expression:

$$A_h = \frac{Z\,I\,S_a}{2\,R\,g}$$

where,

Z = Zone factor **[IS 1893 (Part I): 2016 Table 2]**

For Zone V, Z = 0.36

I = Importance Factor, depending upon the functional use of the structure. [**IS 1893 (Part I): 2016 Table 6]**

= 1.5

R = Response reduction factor [**IS 1893 (Part I): 2016 Table 7**]

= 5.0 (SMRF)

S_a/g = Average response acceleration coefficient which depends on Fundamental natural period of vibration (T_a).

According to **IS 1893 (Part I): 2016 Cl. No. 7.6**

$T_a = 0.075h^{0.75}$ sec

where, h = height of building in m, h = 27.3 m

Therefore,

$T_a = 0.075*27.3^{0.75}$

= 0.896 sec

For T_a = 0.896 sec and medium soil type Sa/g = 1.36/0.896 = 1.518

Now,

$$A_h = \frac{0.36x1.518}{2x3.33} = 0.082$$

According to **IS 1893 (Part I): 2016 Cl. No. 7.5.3** the total design lateral force or design seismic base shear (V_B) along any principal direction is given by

$V_B = A_h$ x W_S

Where, W_S = Seismic weight of the building

= 95716.121 kN

Hence,

V_B = 0.082 * 95716.121

= 7853.896 KN

According to **IS 1893 (Part I): 2016 Cl. No. 7.7.1** the design base shear (V_B) computed above shall be distributed along the height of the building as per the following expression:

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

(For relatively flexible structure)

where,

V_B= Base Shear = 7853.896 KN

Q_i= Design lateral force at floor i

W_i = Seismic weight of floor i

h_i = Height of floor I measured from base

n = No. of story in the building

Lateral Force Distribution					
Story	**W_i (kN)**	**H_i (m)**	$\boldsymbol{W_i h_i^2}$	$\boldsymbol{\frac{W_i h_i^2}{\sum W_i h_i^2}}$	**Q_i (kN)**
Ground	2998.729	3.9	45610.668	0.00134	10.523
First	9532.653	7.8	579966.609	0.01704	133.811
Second	15715.196	11.7	2151253.180	0.06320	496.341
Third	17774.229	15.6	4325536.369	0.12707	997.995
Fourth	17774.229	19.5	6758650.577	0.19855	1559.367
Top	18261.815	23.4	9999439.421	0.29375	2307.087
Roof	13659.269	27.3	10180116.593	0.29906	2348.773
		Sum	**34040573.418**		**7853.896**

Table 3.2 Table for lateral load calculation of Block II

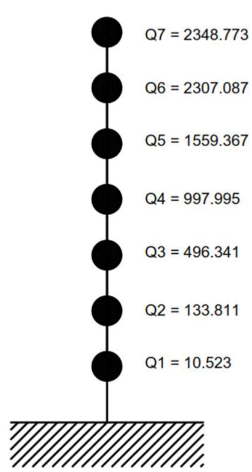


Figure 3.3 Visualization of Qi for Block II

3.6.3 BLOCK-III

SEISMIC ANALYSIS

UNIT WEIGHTS FOR DEAD LOAD **(Ref. IS: 875 – Part I, Table 1)**

Unit weight of brick masonry (Table 1, 36) =19 KN/m^3

Unit weight of RCC (Table 1, 22) =25 KN/m^3

UNIT WEIGHT FOR LIVE LOAD **(Ref. IS: 875 – Part II, Table 1, 1)**

Unit weight for live load is taken according to the occupancy classification

For Staircase, Unit weight for live load = 4 KN/m^2

For Terrace, Unit weight for live load = 0.75 KN/m^2

Calculation/ Analysis

A) SLAB AREA CALCULATION

First Floor Area = 825.125m^2

Second Floor Area = 825.315 m^2

Third Floor Area = 825.404 m^2

Fourth Floor Area = 825.038 m^2

Top Floor Area = 600.392 m^2

Roof Floor Area = 229.678 m^2

B) SLAB LOAD CALCULATION

$Y_C = 25\ KN/m^3$ (**IS 875 Part I – 1987 Table 1**)

Slab load = Y_C*slab area (A)*slab thickness (D)

First Floor Load = 2681.656 kN

Second Floor Load = 2682.274 kN

Third Floor Load = 2682.563 kN

Fourth Floor Load = 2681.374 kN

Top Floor Load = 1951.274 kN

Roof Floor Load = 728.540 kN

C) COLUMN LOAD CALCULATION

Y_C = 25 kN/m^3 **(IS 875 Part I – 1987 Table 1)**

Column load for each floor = N * Y_C * h * A_{column}

Where, N = No. of column in each Floor

h = column height = 3.9 m

A = Area of the column = 750mm *750mm

First Floor = 1809.844 KN

Second Floor = 1809.844 KN

Third Floor = 1809.844 KN

Fourth Floor = 1755 KN

Top Floor = 1179.141 KN

Roof Floor = 329.062 KN

D) PRIMARY BEAM LOAD CALCULATION

Y_C = 25 kN/m^3 **(IS 875 Part I – 1987 Table I)**

Primary Beam load for each floor = Yc * Area of beam * Length of beam

Where, Area = B *D = 400mm * 550mm

First Floor = 1874.25 KN

Second Floor = 1874.25 KN

Third Floor = 1874.25 KN

Fourth Floor = 1874.25 KN

Top Floor = 1626.188 KN

Roof Floor = 434.438 KN

E) SECONDARY BEAM LOAD CALCULATION

Y_C = 25 KN/m^3 **(IS 875 Part I – 1987 Table I)**

Secondary Beam load (each floor) = Yc *Area of beam *Length of beam

Where, Area = B *D = 250mm * 400mm

First Floor = 359.848 KN

Second Floor = 359.848 KN

Third Floor = 359.848 KN

Fourth Floor = 359.848 KN

Top Floor = 238.652 KN

Roof Floor = 68.828 KN

F) <u>MASONRY WALL AND SHEAR WALL LOAD CALCULATION</u>

$Y_{masonry}$ = 19 KN/m³ **<u>(IS 875 Part I – 1987 Table I)</u>**

$Y_{shearwall}$ = 25 KN/m³ **<u>(IS 875 Part I – 1987 Table I)</u>**

Shear wall load = $Y_{shearwall}$* Area* height, where area is taken from ETABS.

Wall thickness = 0.23 m

Wall load = $Y_{masonry}$* length* thickness* height* (100 – 30) %

(30 % area is neglected taking into the consideration of void)

First Floor = 7127.566 KN

Second Floor = 7726.787 KN

Third Floor = 8087.73 KN

Fourth Floor = 6595.964 KN

Top Floor = 3662.142 KN

Roof Floor = 1538.014 KN

G) <u>FLOOR FINISH LOAD CALCULATION</u>

Thickness of FF = 0.04 m

Unit weight of Floor Finish = 25 KN/m³

Now, W_{ff} = Unit weight of Floor Finish* slab area* Thickness of FF

First Floor Area= 825.125 kN

Second Floor Area= 825.315 kN

Third Floor Area= 825.404 kN

Fourth Floor Area= 825.038 kN

Top Floor Area= 600.392 kN

Roof Floor Area= 229.678 kN

H) <u>LIVE LOAD CALCULATION</u>

<u>(IS 1893 Part I: 2016 Cl. 7.3.1 Table 8)</u>

Live load = Live load intensity* Slab area* Live load Reduction Factor

where, Live load intensity = 3 kN/m² (General Floor)

= 0.75 kN/m² (terrace Floor without access)

= 1.5 kN/m² (Terrace Floor with access)

LL Reduction Factor = 50 % (**if >3 KN/m^3**)

= 25% (**if <= 3 KN/m^3**

First Floor = 719.220 KN

Second Floor = 456.507 KN

Third Floor = 411.224 KN

Fourth Floor = 951.288 KN

Top Floor = 903.890 KN

Roof Floor = 172.258 KN

I) STAIRCASE AND RAMP LOAD

$Y_C = 25\ kN/m^3$ (**IS 875 Part I – 1987 Table I)**

Live Load Intensity = 4 KN/m^2 (**IS 1893 Part I: 2016 Cl. 7.3.1 Table 8**)

Live Load Reduction Factor = 50% (**>3 KN/m^3**)

First Floor = 298.271 KN

Second Floor = 298.271 KN

Third Floor = 298.271 KN

Fourth Floor = 298.271 KN

Top Floor = 149.136 KN

Roof Floor = 0

J) LUMPED LOAD CALCULATION

Lumped load = $W_{Slab} + W_{Column} + W_{Primary\ beam} + W_{Secondary\ beam} + W_{Masonry} + W_{FF} + W_{Live\ Load} + W_{shear\ wall} + W_{staircase}$

Now,

1st Floor Load = 15695.780 kN

2nd Load = 16033.096 kN

3rd Floor Load = 16349.134 kN

4th Floor Load = 15341.033 kN

Top Floor Load = 10324.173 kN

Roof Floor Load = 3500.817 kN

K) <u>TOTAL LUMP LOAD CALCULATION</u>

Total lump load (W) = $W_1 + W_2 + W_3 + W_4 + W_5 + W_6$

= 77244.033 kN

Base Shear Calculation

According to **IS 1893 (Part I): 2016 Cl. No. 6.4.2** the design horizontal seismic coefficient A_h for a structure shall be determined by the following expression:

$$A_h = \frac{Z\,I\,S_a}{2\,R\,g}$$

Where,

Z = Zone factor **[IS 1893 (Part I): 2016 Table 2]**

For Zone V, Z = 0.36

I = Importance Factor, depending upon the functional use of the structure. [**IS 1893 (Part I): 2016 Table 6]**

= 1.5

R = Response reduction factor [**IS 1893 (Part I): 2016 Table 7**]

= 5.0 (SMRF)

S_a/g = Average response acceleration coefficient which depends on Fundamental natural period of vibration (T_a).

According to **IS 1893 (Part I): 2016 Cl. No. 7.6**

$T_a = 0.075h^{0.75}$ sec

where,

h = height of building in m, h = 23.4 m

Therefore,

$T_a = 0.075*23.4^{0.75}$

= 0.7979 sec

For T_a = 0.7979 sec and medium soil type Sa/g = 1.36/0.7979 = 1.705

Now, Ah= 0.092

According to **IS 1893 (Part I): 2016 Cl. No. 7.5.3** the total design lateral force or design seismic base shear (V_B) along any principle direction is given by

$V_B = A_h$ x Ws

Where, Ws = Seismic weight of the building

= 77244.033 kN

Hence, $\mathbf{V_B}$ = 7109.541 KN

According to **IS 1893 (Part I): 2016 Cl. No. 7.7.1** the design base shear (V_B) computed above shall be distributed along the height of the building as per the following expression:

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

(For relatively flexible structure)

where, V_B= Base Shear = 7109.541 KN

Q_i = Design lateral force at floor i

W_i = Seismic weight of floor i

h_i = Height of floor I measured from base

n = No. of story in the building

Lateral Force Distribution					
Story	**W_i (kN)**	**H_i (m)**	$\boldsymbol{W_i h_i^2}$	$\boldsymbol{\frac{W_i h_i^2}{\sum W_i h_i^2}}$	**Q_i (kN)**
First	15695.780	3.9	238732.811	0.01832418984	130.276
Second	16033.096	7.8	975453.544	0.07487197021	532.305
Third	16349.134	11.7	2238032.967	0.1717825916	1221.295
Fourth	15341.033	15.6	3733393.766	0.286560594	2037.314
Top	10324.173	19.5	3925766.811	0.3013263909	2142.292
Roof	3500.817	23.4	1916907.466	0.1471342635	1046.057
		Sum	**13028287.36**		**7109.540**

Table 3.3 Table for lateral load calculation of Block III

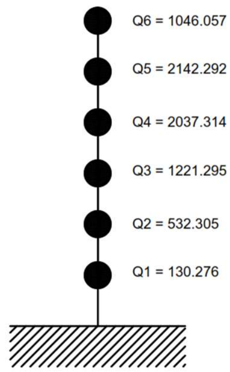


Figure 3.4 Visualization of Qi for Block III

3.6.4 BLOCK-IV

SEISMIC ANALYSIS

UNIT WEIGHTS FOR DEAD LOAD **(Ref. IS: 875 – Part I, Table 1)**

Unit weight of brick masonry (Table 1, 36) =19 KN/m^3

Unit weight of RCC (Table 1, 22) =25 KN/m^3

UNIT WEIGHT FOR LIVE LOAD **(Ref. IS: 875 – Part II, Table 1, 1)**

Unit weight for live load = 3 KN/m^2 (General Floor)

Unit weight for live load = 4 KN/m^2 (Staircase)

Unit weight for live load = 0.75 KN/m^2 (Terrace)

Calculation/ Analysis

A) **SLAB AREA CALCULATION**

First Floor Area=640.8813 m^2

Second Floor Area= 342.1938 m^2

Third Floor Area= 342.1938 m^2

Fourth Floor Area= 342.1938 m^2

Top Floor Area= 342.1938 m^2

B) SLAB LOAD CALCULATION

$Y_C = 25\ KN/m^3$ (**IS 875 Part I – 1987 Table 1**)

Slab load = Y_C*slab area (A)*slab thickness (D)

First Floor Load= 1706.625 KN

Second Floor Load= 1112.13 KN

Third Floor Load= 1112.13 KN

Fourth Floor Load= 1112.13 KN

Top Floor Load= 1112.13 KN

C) COLUMN LOAD CALCULATION

$Y_C = 25\ kN/m^3$ (**IS 875 Part I – 1987 Table 1)**

Column load for each floor = N * Y_C * h * A_{column}

Where, N = No. of column in each Floor

h = column height =3.9 m

A = Area of the column = 750mm * 750mm

First Floor = 1096.875 KN

Second Floor = 932.3438 KN

Third Floor = 932.3438 K

Fourth Floor = 932.3438 KN

Top Floor = 239.0625 KN (Half Column)

D) PRIMARY BEAM LOAD CALCULATION

$Y_C = 25\ kN/m^3$ (**IS 875 Part I – 1987 Table I)**

Primary Beam load for each floor = Yc * Area of beam * Length of beam

Where, Area = B *D = 400mm * 600mm

First Floor = 1163.1375 KN

Second Floor = 808.7625 KN

Third Floor = 808.7625 KN

Fourth Floor = 808.7625 KN

Top Floor = 808.7625 KN

E) SECONDARY BEAM LOAD CALCULATION

$Y_C = 25\ KN/m^3$ **(IS 875 Part I – 1987 Table I)**

Secondary Beam load (each floor) = Yc *Area of beam *Length of beam

where, Area = B *D = 285mm * 420mm

First Floor = 436.082 KN

Second Floor = 314.885 KN

Third Floor = 314.885 KN

Fourth Floor = 314.885 KN

Top Floor = 314.885 KN

F) MASONRY WALL LOAD CALCULATION

$Y_{masonry} = 19\ KN/m^3$ **(IS 875 Part I – 1987 Table I)**

Exterior wall thickness = 0.23 m

Wall load = $Y_{masonry}$* length* thickness* height* (100 – 35) %

Where, Height of wall =1.67

(35 % area is neglected taking into the consideration of void)

Exterior Masonry Wall Load:

First Floor = 1729.944KN

Second Floor = 1371.493 KN

Third Floor = 1910.808 KN

Fourth Floor = 720.8028 KN

Top Floor+Parapet = 690.692 KN

G) FLOOR FINISH LOAD CALCULATION

Thickness of FF = 0.04 m

Unit weight of Floor Finish = 25 KN/m^3

Now, W_{ff} = Unit weight of Floor Finish* slab area* Thickness of FF

First Floor = 524.444 KN

Second Floor = 342.194 KN

Third Floor = 342.194 KN

Fourth Floor = 342.194 KN

Top Floor = 342.194 KN

H) LIVE LOAD CALCULATION

(IS 1893 Part I: 2016 Cl. 7.3.1 Table 8)

Live load = Live load intensity* Slab area* Live load Reduction Factor

LL Reduction Factor = 50 % (**if >3 KN/m^3**)

= 25% (**if <= 3 KN/m^3**

First Floor = 745.602 KN

Second Floor = 318.2 KN

Third Floor = 238.128 KN

Fourth Floor = 684.38KN

Top Floor = 684.38KN

I) LUMPED LOAD CALCULATION

Lumped load = $W_{Slab} + W_{Column} + W_{Primary\ beam} + W_{Secondary\ beam} + W_{Masonry} + W_{FF} + W_{Live\ Load} + W_{shear\ wall} + W_{staircase}$

Now,

FirstFloor Load = 7404.709KN

2nd Floor Load = 5200.008KN

3rd Floor Load = 5659.256KN

4th Floor Load =4915.506KN

Top Floor Load = 4192.114KN

Total = 2321.3528 KN

J) TOTAL LUMP LOAD CALCULATION

Total lump load (W) = $W_1 + W_2 + W_3 + W_4 + W_5 + W_6 + W_7 + W_8$

= 27371.59 KN

Base Shear Calculation

According to **IS 1893 (Part I): 2016 Cl. No. 6.4.2** the design horizontal seismic coefficient A_h for a structure shall be determined by the following expression:

$$A_h = \frac{Z\,I\,S_a}{2\,R\,g}$$

where,

Z = Zone factor **[IS 1893 (Part I): 2016 Table 2]**

For Zone V, Z = 0.36

I = Importance Factor, depending upon the functional use of the structure. [**IS 1893 (Part I): 2016 Table 6]**

= 1.5

R = Response reduction factor [**IS 1893 (Part I): 2016 Table 7**]

= 5.0 (SMRF)

S_a/g = Average response acceleration coefficient which depends on Fundamental natural period of vibration (T_a).

According to **IS 1893 (Part I): 2016 Cl. No. 7.6**

$T_a = 0.075h^{0.75}$ sec

where, h = height of building in m, h = 19.6m

Therefore, $T_a = 0.075*19.6^{.75}$

= 0.698 sec

For T_a = 0.698 sec and medium soil type Sa/g = 1.94

Now,

$$A_h = \frac{0.36x1x1.94}{2x5} = 0.105$$

According to **IS 1893 (Part I): 2016 Cl. No. 7.5.3** the total design lateral force or design seismic base shear (V_B) along any principle direction is given by

$V_B = A_h$ x Wdd

Where, W = Seismic weight of the building

= 27371.59 KN

Hence, $\mathbf{V_B}$ =0.105*27371.59

= 2874.017 KN

According to **IS 1893 (Part I): 2016 Cl. No. 7.7.1** the design base shear (V_B) computed above shall be distributed along the height of the building as per the following expression:

$$Q_i = V_B \frac{W_i\,h_i^2}{\sum_{j=1}^{n} W_j\,h_j^2}$$ (For relatively flexible structure)

Lateral Force Distribution					
Story	**W_i (kN)**	**H_i (m)**	$\boldsymbol{W_i h_i^2}$	$\boldsymbol{\frac{W_i h_i^2}{\sum W_i h_i^2}}$	**Q_i (kN)**
Ground	7404.71	3.9	112625	0.02808	80.6922
First	5200.01	7.8	316368	0.07887	226.668
Second	5659.26	11.7	775695	0.19337	555.761
Third	4915.51	15.6	1196238	0.29821	857.066
Roof	4192.11	19.6	1610443	0.40147	1153.83
		Sum	**4011368**		**2874.02**

Table 3.4 Table for Calculation of Lateral loads of Block IV

where,

V_B= Base Shear = 2874.017 KN

Q_i = Design lateral force at floor i

W_i = Seismic weight of floor i

h_i = Height of floor I measured from base

n = No. of story in the building

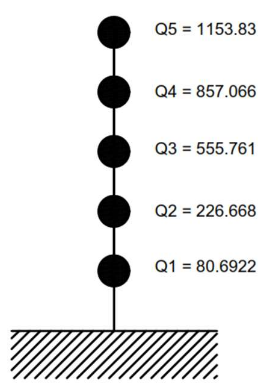


Figure 3.5 Visualization of Qi for Block IV

4 Modeling and structural analysis

4.1 Introduction

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

The structural analysis of the model is must to get the approximate response of structure under the prevailing loading conditions. This response is then used to derive the element internal forces and stresses, upon which the design of the element is based.

4.2 Analysis

I. Manual calculation

Since static-indeterminacy of multi storied building is very high, manual calculation is practically impossible; more tedious, time consuming and possibility of errors.

II. Use of standard software.

Two of the following analysis can be done by using standard software

- 2D-analysis
- 3D- analysis

Among various software, **ETABS 19** has been used for our purpose.

4.3 Modeling and Analysis Tool

ETABS version 19 was used as a tool for modeling and analysis of the building. ETABS is the most sophisticated and user-friendly structural analysis program. Creation modification of models, execution of analysis and checking and optimization of the design can be done through this single interface. Graphical displays of the results including real time, display of time history displacements are easily produced in it. The analysis is done based on the principle of finite element method. In this project the beams and columns were modeled as 3D frame elements and slab & shear walls as 3D shell elements.

4.4 Analysis Process

The finite element analysis program in ETABS involves of three stages of activities: **preprocessing, processing, post processing**.

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

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

The post processing stage deals with the presentation of results. Typically, deformed configuration, mode shapes, temperature and stress distributions are computed and displayed at this stage.

5 Idealization and analysis of structure

5.1 Idealization of Structure

Structural idealization involves incorporating necessary constraints in real structures to ensure that the design conforms to available theories and provides the required level of performance based on a probabilistic measure.

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

1. Idealization of Load
2. Idealization of Structure

The idealization of a structure is a crucial aspect that involves imposing restraints or constraints on variables that may not be properly addressed otherwise. To understand these idealizations, we must begin at the elemental level, which involves idealizing supports, slab elements, staircase elements, beam and column elements, and the overall structural system.

5.1.1 Idealization of Supports

In general, idealization of supports deals with the assessment of fixity of structure at the foundation level.

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

5.1.2 Idealization of Slab

Idealization of slab element is done in earthquake resistant design to perform as a rigid floor diaphragm. This idealization is done for the slab to behave as a thin shell element subjected to out-of-plane bending only under the action of gravity loads. Due to infinite in-plane stiffness of the shell element, lateral loads are not taken by the floor slab and hence resisted completely by the columns. Hence, such an idealized slab is then modeled in ETABS19 program for analysis.

5.1.3 Idealization of Staircase

The open-well staircase incorporated in the building is simplified to act as simply supported slabs, which are supported by beams at floor and landing levels. This simplification facilitates the analysis of the staircase slab as strips exposed to distributed loading on the landing strip and the going of the slab.

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

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

5.1.4 Idealization of Beam and Column

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

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

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

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

5.1.5 Idealization of Structural System

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

The building is idealized as unbraced space frame. This 3D space framework is modeled in ETABS 19 for analysis. Loads are modeled into structure in several load cases and load combinations.

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

5.2 Structural Analysis

Analysis of Structure

The analysis of structure is carried out in a commercial computer software ETABS 19, the salient features of which are already explained in detail in methodology.The results of analysis are used according to our necessities in designing representative beams and columns sections. A detailed manual design of these sample representive sections are presented later.

The size of members was entered as obtained from preliminary design. All members passed with the preliminary size of the elements with slab of thickness 130 mm, main beam of size 600 mm x 400 mm and 650 mm x 500, secondary beam of size 450mm x 300 mm and column of size 750*750 mm.

5.2.1 Story Drift Computation from ETABS 19 Analysis

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

The relative inter-story horizontal displacement is referred to as story drift. A limitation on story drift is necessary to avoid discomfort to occupants of the building and to save non-structural elements from damage. A drift of 0.004 times or 0.4% the story height in the elastic range is imposed by IS 1893:2016.

Table 5.1Story Drifts (BLOCK I)

Story	Drift ratio in X direction	Drift ratio in Y direction	Result
Top	0.001133	0.000843	Regular
Fourth	0.001163	0.00083	Regular
Third	0.001169	0.000819	Regular
Second	0.001098	0.000756	Regular
First	0.000939	0.000628	Regular
Ground	0.000666	0.000424	Regular
Basement	0.000148	0.000112	Regular

Table 5.2 Story Drifts (BLOCK II)			
Story	**Drift ratio in X direction**	**Drift ratio in Y direction**	**Result**
Top	0.000799	0.002137	Regular
Fourth	0.000737	0.00236	Regular
Third	0.000703	0.002315	Regular
Second	0.000703	0.002297	Regular
First	0.000686	0.002058	Regular
Ground	0.000672	0.001523	Regular
Basement	0.000135	0.000389	Regular
Base	0	0	Regular

Table 5.3 Story Drift (BLOCK III)			
Story	**Drift ratio in X direction**	**Drift ratio in Y direction**	**Result**
Top	0.001545	0.004022	Regular
Fourth	0.000856	0.001033	Regular
Third	0.000892	0.001113	Regular
Second	0.000717	0.000926	Regular
First	0.000415	0.000504	Regular
Ground	0.000081	0.000064	Regular
Basement	0	0	Regular

Table 5.4 Story Drifts (BLOCK IV)			
Story	**Drift ratio in X direction**	**Drift ratio in Y direction**	**Result**
Top	0.002021	0.002477	Regular
Third	0.002823	0.003378	Regular
Second	0.003289	0.003821	Regular
First	0.003011	0.003216	Regular
Ground	0.002395	0.002126	Regular
Basement	0.000233	0.000135	Regular

As maximum drift ratio in our model is less than 0.004 in both the cases except roof, whose drift comes out high which can't be controlled, it is under the permissible limit. Thus, it is okay.

5.2.2 Stiffness Irregularity Computation from ETABS 19 Analysis

A soft story is the one in which the lateral lateral stiffness is less than that of the story above. Lateral stiffness in the open story(s) is less than 80% of that in the sstory above. Computation of soft story is shown below from the story stiffness data obtained from ETABS.

Table 5.5 Story Stiffness (BLOCK I)

Story	Stiffness X (KNm)	Ki/Ki+1	Check	Stiffness Y (KNm)	Ki/Ki+1	Check
Top	188205.679	0	-	265880.632	0	-
Fourth	551736.724	2.932	Regular	856670.629	3.222	Regular
Third	912768.574	1.654	Regular	1430680.133	1.670	Regular
Second	1211855.204	1.328	Regular	1926434.335	1.347	Regular
First	1571851.38	1.297	Regular	2567903.982	1.333	Regular
Ground	2324373.81	1.479	Regular	4022996.185	1.567	Regular

Table 5.6 Story Stiffness (BLOCK II)

Story	Stiffness X (KNm)	Ki/Ki+1	Check	Stiffness Y (KNm)	Ki/Ki+1	Check
Top	569764.433	0	-	355324.477	0	-
Fourth	2115433.952	3.713	Regular	1171673.537	3.297	Regular
Third	4353175.643	2.058	Regular	2184224.149	1.864	Regular
Second	5671471.913	1.303	Regular	2910989.176	1.333	Regular
First	6337854.41	1.117	Regular	3824559.983	1.314	Regular
Ground	6637633.975	1.047	Regular	5052226.962	1.321	Regular

Table 5.7 Story Stiffness (BLOCK III)

Story	Stiffness X (KNm)	Ki/Ki+1	Check	Stiffness Y (KNm)	Ki/Ki+1	Check
Roof	181954.498	0	-	164216.348	0	-
Top	758219.626	4.167	Regular	568792.079	3.464	Regular
Fourth	1288478.369	1.699	Regular	985992.202	1.733	Regular
Third	1815739.143	1.409	Regular	1370797.749	1.390	Regular
Second	2549965.99	1.404	Regular	2032020.107	1.482	Regular
First	5321616.762	2.087	Regular	5278609.152	2.598	Regular
Ground	18325562.55	3.444	Regular	20636133.53	3.909	Regular

Table 5.8 Story Stiffness (BLOCK IV)						
Story	**Stiffness X (KNm)**	**Ki/Ki+1**	**Check**	**Stiffness Y (KNm)**	**Ki/Ki+1**	**Check**
Top	137807.656	0	-	109831.542	0	-
Third	181462.924	1.317	Regular	148635.619	1.353	Regular
Second	195618.396	1.078	Regular	164472.904	1.107	Regular
First	225060.148	1.151	Regular	202144.608	1.229	Regular
Ground	618282.906	2.747	Regular	708320.83	3.504	Regular

5.2.3 Mass Irregularity Computation from ETABS 19 Analysis

Mass irregularity shall be considered to exist where the seismic weight of any story is more than 150 percent of that of floors below. The irregularity need not be considered in case of roof. **(IS 1893-2016)**

Table: Mass Summary by Story (BLOCK I)			
Story	**Seismic Mass along X and Y direction**	**Mi/Mi-1**	**Check**
Top	141079.7	0.408	Regular
Fourth	346187.6	0.669	Regular
Third	517328.8	1.006	Regular
Second	514229.1	0.999	Regular
First	514668.5	1.001	Regular
Ground	514210.5	0.828	Regular
Basement	620818.7	0.000	Regular

Table: Mass Summary by Story (BLOCK II)			
Story	**Seismic Mass along X and Y direction**	**Mi/Mi-1**	**Check**
Top	278566.76	0.319	Regular
Fourth	873214.3	0.472	Regular
Third	1848634.05	0.906	Regular
Second	2041514.45	0.998	Regular
First	2045691.13	1.003	Regular
Ground	2039926.93	0.908	Regular
Basement	2245571	0	Regular

Table: Mass Summary by Story (BLOCK III)			
Story	**Seismic Mass along X and Y direction**	**Mi/Mi-1**	**Check**
Roof	181288.24	-	Regular
Top	658076.14	0.275	Regular
Fourth	1256920.87	0.524	Regular
Third	1447155.74	0.869	Regular
Second	1431354.79	1.011	Regular
First	1465736.81	0.977	Regular
Ground	1687023.92	0.869	Regular

Table: Mass Summary by Story (BLOCK IV)			
Story	**Seismic Mass along X and Y direction**	**Mi/Mi-1**	**Check**
Top	722651.04	0.808	Regular
Third	894759.92	0.947	Regular
Second	944480.25	1.057	Regular
First	893513.16	0.708	Regular
Ground	1262254.7	0.712	Regular
Basement	1772092.9	0.000	Regular

5.2.4 Torsion Irregularity Computation from ETABS 19 Analysis

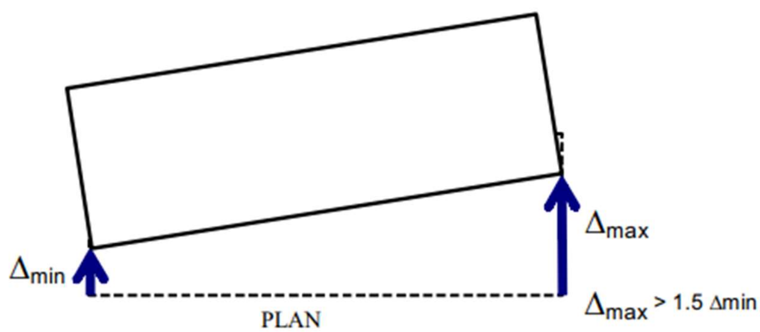
A three-dimensional building has series of frames in orthogonal direction X & Y to resist gravity loads and lateral loads. A floor is generally quite rigid in its own plane and each frame may have different stiffness distribution and mass distribution.

The earthquake force acts through center of mass and is resisted by the building through its center of rigidity. This leads to horizontal twisting of building and is called torsion. The floor generally rotates as a rigid body. The magnitude of the torsional moment depends on the distance between center of mass and center of rigidity which is referred as eccentricity.

A three-dimensional analysis of building using general purpose matrix analysis computer programs is able to take care of eccentricity but without displaying its magnitude. However, there is no general-purpose computer which is able to account for the design eccentricity because there is no direct method to compute center of rigidity or shear center of each floor/story. This is the main reason why most of designers adopt approximate methods for the torsional analysis of building. The design eccentricity of 5% for the applied lateral load is used.

Several studies made of structural damages during past wind and earthquakes reveal that the torsion is the most critical factor causing partial structural damages or complete collapse of buildings. This needs to be considered when the floor diaphragms are rigid in their own plane in relation to the vertical structural elements that resist the lateral forces.

A building is said to be torsionally irregular when the maximum horizontal displacement of any floor in the direction of the lateral force at one end of the floor is more than 1.5 times its minimum horizontal displacement at the far end of the same floor in that direction. **(IS 1893-2016)**



3A TORSIONAL IRREGULARITY

Table: Story Max Over Min Displacements (BLOCK I)								
	X-DIRECTION				Y-DIRECTION			
Story	**Maximum (mm)**	**Minimum (mm)**	**Ratio**	**Check**	**Maximum (mm)**	**Minimum (mm)**	**Ratio**	**Check**
Lift Cover	26.318	25.616	1.027	Regular	18.696	17.816	1.049	Regular
Roof	23.934	19.84	1.206	Regular	17.256	14.526	1.188	Regular
Top	19.955	15.195	1.313	Regular	14.198	9.638	1.473	Regular
Fourth	15.413	11.827	1.303	Regular	10.827	7.241	1.495	Regular
Third	10.949	8.417	1.301	Regular	7.584	5.252	1.444	Regular
Second	6.744	5.136	1.313	Regular	4.593	3.385	1.357	Regular
First	3.14	2.276	1.380	Regular	2.11	1.446	1.459	Regular
Ground	0.58	0.3884	1.493	Regular	0.389	0.287	1.355	Regular

Table: Story Max Over Min Displacements (BLOCK II)								
	X-DIRECTION				Y-DIRECTION			
Story	**Maximum (mm)**	**Minimum (mm)**	**Ratio**	**Check**	**Maximum (mm)**	**Minimum (mm)**	**Ratio**	**Check**
Top	17.018	15.34	1.109	Regular	48.705	46.257	1.053	Regular
Fourth	14.127	11.617	1.216	Regular	42.645	38.423	1.110	Regular
Third	11.275	9.427	1.196	Regular	33.451	30.713	1.089	Regular
Second	8.533	7.075	1.206	Regular	24.428	22.392	1.091	Regular
First	5.819	4.707	1.236	Regular	15.473	14.129	1.095	Regular
Ground	3.146	2.386	1.319	Regular	7.453	6.757	1.103	Regular
Basement	0.526	0.204	2.578	Regular	1.515	1.319	1.149	Regular

Table: Story Max Over Min Displacements (BLOCK III)								
	X-DIRECTION				Y-DIRECTION			
Story	**Maximum (mm)**	**Minimum (mm)**	**Ratio**	**Check**	**Maximum (mm)**	**Minimum (mm)**	**Ratio**	**Check**
Roof	19.13	17.17	1.114	Regular	21.593	20.735	1.041	Regular
Top	15.665	12.198	1.284	Regular	18.016	15.554	1.158	Regular
Fourth	12.107	9.437	1.283	Regular	14.281	11.439	1.248	Regular
Third	8.44	6.431	1.312	Regular	10.088	7.274	1.387	Regular
Second	4.993	3.709	1.346	Regular	5.472	4.023	1.36	Regular
First	2.168	1.565	1.385	Regular	2.1	1.416	1.483	Regular

Table: Story Max Over Min Displacements (BLOCK IV)								
	X-DIRECTION				Y-DIRECTION			
Story	Maximum (mm)	Minimum (mm)	Ratio	Check	Maximum (mm)	Minimum (mm)	Ratio	Check
Top	49.251	49.031	1.004	Regular	53.611	53.341	1.005	Regular
Third	42.26	41.168	1.027	Regular	45.055	43.703	1.031	Regular
Second	33.004	30.158	1.094	Regular	34.053	30.529	1.115	Regular
First	21.835	17.331	1.260	Regular	21.203	15.629	1.357	Regular
Ground	10.104	7.156	1.412	Regular	8.672	6.044	1.435	Regular
Basement	0.909	0.623	1.459	Regular	0.528	0.368	1.435	Regular

5.2.5 Drift Check (Response Spectra)

Table: Drift check (Response Spectrum) BLOCK I				
Story	X-Dir	Result	Y-Dir	Result
Top	0.00093	Regular	0.00066	Regular
Fourth	0.00095	Regular	0.00086	Regular
Third	0.00097	Regular	0.00079	Regular
Second	0.00092	Regular	0.00079	Regular
First	0.00081	Regular	0.00075	Regular
Ground	0.00058	Regular	0.00064	Regular
Basement	0.00013	Regular	0.00045	Regular

Table: Drift check (Response Spectrum) BLOCK II				
Story	X-Dir	Result	Y-Dir	Result
Top	0.00064	Regular	0.00080	Regular
Fourth	0.00063	Regular	0.00091	Regular
Third	0.00062	Regular	0.00095	Regular
Second	0.00061	Regular	0.00096	Regular
First	0.00059	Regular	0.00088	Regular
Ground	0.00055	Regular	0.00067	Regular
Basement	0.00013	Regular	0.00016	Regular

Table: Drift check (Response Spectrum) BLOCK III				
Story	X-Dir	Result	Y-Dir	Result
Roof	0.00086	Regular	0.00092	Regular
Top Floor	0.00087	Regular	0.00107	Regular
Fourth Floor	0.00089	Regular	0.00106	Regular
Third Floor	0.00084	Regular	0.00117	Regular
Second Floor	0.00072	Regular	0.00097	Regular
First Floor	0.00040	Regular	0.00042	Regular
Ground Floor	0.00013	Regular	0.00012	Regular
Basement	0	Regular	0	Regular

Table: Drift check (Response Spectrum) BLOCK IV				
Story	X-Dir	Result	Y-Dir	Result
Fourth	0.00163	Regular	0.00204	Regular
Third	0.00223	Regular	0.00264	Regular
Second	0.00268	Regular	0.00301	Regular
First	0.00279	Regular	0.00268	Regular
Ground	0.00197	Regular	0.00132	Regular
Basement	0.00018	Regular	0.00006	Regular

Torsional irregularity (Response Spectra)

Table: Story Max Over Min Displacements (BLOCK I)								
	X-DIRECTION				Y-DIRECTION			
Story	Maximum (mm)	Minimum (mm)	Ratio	Check	Maximum (mm)	Minimum (mm)	Ratio	Check
Lift	22.537	22.231	1.014	Regular	16.619	15.903	1.045	Regular
Roof	20.133	18.813	1.070	Regular	16.691	12.053	1.385	Regular
Top	16.893	15.407	1.096	Regular	13.713	9.177	1.494	Regular
Fourth	13.152	12.012	1.095	Regular	10.584	7.296	1.451	Regular
Third	9.432	8.612	1.095	Regular	7.512	5.048	1.488	Regular
Second	5.868	5.332	1.101	Regular	4.606	3.272	1.199	Regular
First	2.752	2.44	1.128	Regular	2.114	1.542	1.371	Regular
Ground	0.502	0.36	1.394	Regular	0.38	0.272	1.397	Regular

Table: Story Max Over Min Displacements (BLOCK II)								
	X-DIRECTION				Y-DIRECTION			
Story	**Maximum (mm)**	**Minimun (mm)**	**Ratio**	**Check**	**Maximum (mm)**	**Minimum (mm)**	**Ratio**	**Check**
Top	14.612	10.486	1.393	Regular	18.745	17.619	1.064	Regular
Fourth	12.104	8.098	1.495	Regular	17.374	14.63	1.188	Regular
Third	9.53	6.642	1.435	Regular	14.06	11.688	1.203	Regular
Second	7.123	5.049	1.411	Regular	10.365	8.631	1.201	Regular
First	4.109	4.071	1.009	Regular	6.629	5.547	1.195	Regular
Ground	2.237	2.067	1.082	Regular	3.196	2.718	1.176	Regular
Basement	0.297	0.385	0.771	Regular	0.605	0.561	1.078	Regular

Table: Story Max Over Min Displacements (BLOCK III)								
	X-DIRECTION				Y-DIRECTION			
Story	**Maximum (mm)**	**Minimum (mm)**	**Ratio**	**Check**	**Maximum (mm)**	**Minimum (mm)**	**Ratio**	**Check**
Roof	17.042	14.054	1.213	Regular	18.987	17.23	1.102	Regular
Top	14.149	10.658	1.328	Regular	16.06	12.888	1.246	Regular
Fourth	11.007	8.29	1.328	Regular	12.967	9.579	1.354	Regular
Third	7.711	5.714	1.349	Regular	9.372	6.215	1.508	Regular
Second	4.623	3.336	1.386	Regular	5.219	3.488	1.496	Regular
First	2.049	1.407	1.456	Regular	2.01	1.252	1.605	Regular

Table: Story Max Over Min Displacements (BLOCK IV)								
	X-DIRECTION				Y-DIRECTION			
Story	**Maximum (mm)**	**Minimum (mm)**	**Ratio**	**Check**	**Maximum (mm)**	**Minimum (mm)**	**Ratio**	**Check**
Fourth	42.752	32.468	1.317	Regular	43.534	34.984	1.244	Regular
Third	36.932	27.612	1.338	Regular	36.192	29.412	1.231	Regular
Second	28.91	20.928	1.381	Regular	26.5	22.046	1.202	Regular
First	18.875	12.735	1.482	Regular	15.043	13.187	1.141	Regular
Ground	8.236	5.708	1.443	Regular	5.348	4.648	1.151	Regular
Basement	0.701	0.489	1.434	Regular	0.207	0.139	1.489	Regular

5.2.6 Modal Data (Time Period and Mass participation Ratio)

The number of modes to be used in the analysis for the earthquake shaking along a considered direction, should be such that the sum total of modal masses of the modes is at least 90% of the total seismic mass. If modes with natural frequencies beyond 33 Hz are to be considered, the modal combination shall be carried out only for modes with natural frequency less than 33 Hz; the effect of modes with natural frequency greater than 33 Hz shall be included by missing mass correction procedure. The first three modes together contribute at least 65% mass participation factor in each principal plan direction for building located in zone V and the fundamental lateral natural periods of the building in two principal plan directions are away from each other by at least 10% of the larger value. **(IS 1893-2016)**

Table: Modal Participating Mass Ratios (BLOCK I)				
Mode	**Period (sec)**	**SumUX**	**SumUY**	**SumRZ**
1	0.578	0.6279	0.0031	0.0035
2	0.461	0.633	0.6635	0.0798
3	0.356	0.6342	0.6718	0.6887
4	0.141	0.8178	0.6718	0.6937
5	0.125	0.8186	0.7337	0.7229
6	0.104	0.8224	0.8033	0.7302
7	0.083	0.8349	0.8102	0.7499
8	0.074	0.8355	0.8509	0.7781
9	0.07	0.8834	0.8523	0.7941
10	0.051	0.9119	0.8553	0.7991
11	0.048	0.9139	0.9015	0.7991
12	0.042	0.9281	0.9023	0.8372
13	0.039	0.9418	0.9024	0.85
14	0.035	0.9419	0.928	0.8504
15	0.033	0.9697	0.928	0.8507
16	0.032	0.9697	0.9296	0.8513
17	0.031	0.989	0.9301	0.8551
18	0.029	0.9979	0.9415	0.8679
19	0.028	0.9985	0.9483	0.8728
20	0.026	0.9985	0.9818	0.8736
21	0.023	0.9988	0.9866	0.8866
22	0.021	1	0.9999	0.9998
23	0.018	1	0.9999	0.9998

Table: Modal Participating Mass Ratios (BLOCK II)				
Mode	Period (sec)	SumUX	SumUY	SumRZ
1	0.61	0.0456	0.7282	0.0002
2	0.49	0.7838	0.7762	0.0334
3	0.445	0.8224	0.7764	0.8456
4	0.169	0.8235	0.9611	0.8528
5	0.152	0.8235	0.9616	0.8531
6	0.149	0.8291	0.9796	0.9794
7	0.129	0.9902	0.9797	0.9919
8	0.126	0.9904	0.98	0.9919
9	0.125	0.9939	0.9802	0.9919
10	0.123	0.9943	0.9803	0.9923
11	0.119	0.9947	0.9803	0.9925
12	0.118	0.9947	0.9803	0.9925
13	0.117	0.9958	0.9804	0.9925
14	0.114	0.9958	0.9804	0.9925
15	0.113	0.9959	0.9804	0.9925
16	0.112	0.9961	0.9804	0.9926
17	0.111	0.9961	0.9805	0.9926
18	0.111	0.9962	0.9808	0.9926
19	0.111	0.9964	0.9809	0.9926
20	0.11	0.9964	0.9809	0.9926

Table: Modal Participating Mass Ratios (BLOCK III)				
Mode	Period (sec)	SumUX	SumUY	SumRZ
1	1.014	0.055	0.284	0.2849
2	0.908	0.5791	0.2896	0.4434
3	0.89	0.648	0.6587	0.6576
4	0.304	0.6481	0.6884	0.7208
5	0.268	0.7109	0.7514	0.7455
6	0.251	0.7879	0.7903	0.7621
7	0.171	0.7903	0.7946	0.7943
8	0.143	0.7956	0.8374	0.7992
9	0.125	0.8307	0.8384	0.8053
10	0.113	0.8468	0.8388	0.8258
11	0.1	0.8482	0.8618	0.826
12	0.09	0.8555	0.8655	0.8339
13	0.075	0.8576	0.8827	0.8342
14	0.075	0.868	0.8828	0.8447
15	0.07	0.8803	0.8858	0.847
16	0.057	0.8804	0.9041	0.849
17	0.054	0.8931	0.9044	0.8494
18	0.05	0.8931	0.9044	0.8494
19	0.047	0.8932	0.9047	0.8494
20	0.046	0.8973	0.9299	0.8496

21	0.044	0.8974	0.93	0.8496
22	0.042	0.9351	0.9763	0.8601
23	0.039	0.9354	0.9763	0.8604
24	0.039	0.9362	0.9765	0.8605
25	0.038	0.9363	0.9782	0.8622
26	0.038	0.9385	0.9785	0.8658
27	0.038	0.9386	0.9786	0.8658
28	0.033	0.9386	0.9786	0.8658
29	0.03	0.9388	0.9786	0.8659
30	0.029	0.9917	0.9901	0.9869

Table: Modal Participating Mass Ratios (BLOCK IV)				
Mode	**Period (sec)**	**SumUX**	**SumUY**	**SumRZ**
1	1.61	0.0029	0.6721	0.0029
2	1.519	0.6736	0.6735	0.0342
3	1.215	0.6892	0.6756	0.6981
4	0.507	0.693	0.8496	0.7443
5	0.48	0.8612	0.8528	0.7493
6	0.416	0.8631	0.8533	0.8353
7	0.274	0.8675	0.9163	0.8469
8	0.257	0.9248	0.9267	0.8478
9	0.226	0.9351	0.931	0.9149
10	0.159	0.936	0.9651	0.9187
11	0.155	0.9647	0.967	0.9191
12	0.137	0.968	0.9684	0.9459
13	0.124	0.9688	0.9688	0.9462
14	0.121	0.9688	0.9688	0.9466
15	0.12	0.9689	0.9688	0.9467
16	0.12	0.9689	0.9688	0.9467
17	0.119	0.9689	0.9688	0.9467
18	0.114	0.969	0.9701	0.9539
19	0.109	0.9857	0.9783	0.969
20	0.105	0.9954	0.995	0.9692
21	0.094	0.9962	0.9955	0.997
22	0.086	0.9967	0.9992	0.9973
23	0.084	0.9998	0.9997	0.9974
24	0.073	1	1	1

6. Design of structural element and their detailing

Altogether, five structural elements; slab, beam, column, footing and staircase have been designed. The design procedure for each structural element adopted is given below.

6.1 Design Procedure

6.1.1 Slab

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

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

1. Clear size, L_x and L_y.
2. Effective depth is taken from preliminary design
3. Calculation of effective span:

   $l_x = L_x + d_{eff}$ (slab) or width of support

   (Less of the above two values are taken.)

4. Calculation of the load (Dead load and Live load)
5. If $\frac{L_y}{L_x} <= 2$, Two-way slab is designed.
6. Calculation of bending moments:

   Positive and negative moments (α_x and α_y) are taken from **IS:456-2000**, Table 26, p.9 according to $\frac{L_y}{L_x}$ ratio.

   Bending moment is calculated using following formula:

   $$M_x = \alpha_x * W * l_x^2$$

   $$M_y = \alpha_y * W * l_x^2$$

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

a_x and a_y are bending moment coefficients,

l_x and l_y are the length of short and long span respectively

7. Effective depth from moment criteria is calculated to check the required effective depth for moment criteria using following formula:

$$M_{max} = 0.133\ f_{ck}\ bd^2$$

8. Area of steel required for negative moment at continuous edge and positive moment at mid span:

For short span, it is calculated using following formula,

$$M_x = 0.87 f_y A_{stx} d \left\{1 - \frac{f_y A_{stx}}{dbf_{ck}}\right\}$$

For long span, it is calculated using following formula,

$$M_y = 0.87 f_y A_{sty} d \left\{1 - \frac{f_y A_{sty}}{dbf_{ck}}\right\}$$

9. Check for minimum steel from codes:

For Fe 500, Minimum area of steel =0.12% of bD

10. Maximum spacing:

Spacing $\leq 3d$

$\leq 300mm$

11. Minimum area of steel required is provided in edge strip.
12. Corner steels (torsion steel):

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

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

$$Vu = \frac{Wulx}{2}$$

$$\tau v = \frac{v}{bd1}$$

Percentage of steel, p % = $(A_{st})/bd_1$

For p% & M20 grade concrete, τ_c is taken from IS **456:2000 Table 19 p.73** and k is taken as per **IS 456:2000, Cl.40.2.1.1, p.72.**

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

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

$$L_d \leq (1.3M_1/V) + L_o$$

$$\frac{0.87fy\phi}{4\tau bd} \leq \frac{1.3M1}{V} + Lo$$

15. Deflection is checked at mid span of short span according to **IS 456:2000, Cl.23.2.1, p.37:**

$$\frac{L}{d} \leq \alpha\beta\gamma\delta\lambda$$

6.1.2 Beam

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

Beam is designed as rectangular beam with doubly reinforced section. Dimension of the beam was fixed from preliminary design. Design procedure of the beam is as follows:

1. Size of the beam from preliminary design
2. Factored bending moment (M_u) and factored shear force (V_u) from ETABS analysis
3. Assuming diameter of reinforcement bars, with 30 mm clear cover, effective depth is calculated as

$$d = D - (\emptyset/2) - cc$$

4. Determination of limiting bending moment is calculated using following formula:

$$M_{lim} = 0.36 * f_{ck} * b * X_m (d - 0.4 X_m)$$

If $M_u > M_{lim}$, Doubly reinforced section is designed

If $M_u < M_{lim}$, singly reinforced section is designed

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

$$M_{lim} = 0.87*fy*A_{st1}*(d - (f_y*A_{st1})/(f_{ck}*b))$$

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

$$M_u - M_{lim} = (f_{sc} - f_{cc}) A_{sc} (d - d')$$

Where, f_{sc} for $\frac{d'}{d}$ is taken from SP-16,

Table F, p.13

$f_{cc} = 0.446f_{ck}$

Area of compression steel required for additional bending moment $M_u — M_{lim}$ is calculated as

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

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

7. Check for minimum area of tension steel from IS 456:2000, Cl.26.5.1.1 (a), p.46-47.

$$A_o/bd = 0.85/f_y$$

8. Check for maximum area of tension steel from IS 456:2000, Cl.26.5.1.1 (a), p. 47,

$$A_o= 0.04\ bD$$

9. Check for shear

Permissible shear stress τ_c is taken from IS 456:2000, Table 19, p.73 for designed p_t.

Nominal Shear stress,

$$\tau v = \frac{Vu}{bd}$$

$\tau_{c,max}$is taken from IS 456:2000, Table 20, p.73 for designed grade of concrete. If

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

Design shear force,

$$V_{us} = V_u - \tau_c\ bd$$

$V_{us} = (0.87f_y*A_{sv}*d)/x$

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

10. Check for ductility

Percentage of minimum and maximum area of tension reinforcements according to IS 13920:2016,

$$\rho min = (0.24\sqrt{fck}) / fy)$$

$$\rho_{max} = 2.5\%$$

11. Ductility check for shear

Spacing $x \leq d/4$

$\leq 8\ \phi \qquad \geq 100$ mm

6.1.3 Column

The design of column is governed by limit state of axial compression and bending moments about two axes. Shear in column is small and shear stress work out to be safe. Stirrups in column are provided mainly for holding column bars in place and making them strong against bulking and bursting as these bars come under direct compression. Moments in column change sign in each story, so that, we generally provide symmetrical bar arrangement in a column section and the steel area is kept constant throughout a given story.

The following steps are following in design of axially loaded column with biaxial bending.

1. Size of column
2. If slenderness ratio = (le/LLD) < 12, Case: Short column

   (Where, LLD = least lateral dimension)

3. Check for eccentricity

   e_{min} = L/500 +D/30

   Therefore, Moment due to eccentricity = P_u x e_{min}

   Reinforcement is equally distributed on four sides

4. Axial force (P_u), moments (M_{ux} &M_{uy}) are taken from ETABS analysis.
5. Assume percentage of reinforcement, p, and carried out trial for this percentage.
6. Find p/f_{ck}, d/D and $p_u/f_{ck}bd$
7. Find ($M_u/f_{ck}bD^{2)}$ Referring to chart as per the value obtain from step 6 from SP-16.
8. Calculation of Puz using

   $P_{uz} = 0.446f_{ck}BD + 0.75f_y*A_s$

9. Find (p_u/p_{uz}), (M_{uy}/M_{uy1}), (M_{uz}/M_{uz1}) and (α_n)
10. Find $(M_{uy}/M_{uy1})^{\alpha n}+ (M_{uz}/M_{uz1})^{\alpha n}$

11. If $(M_{uy}/M_{uy1})^{\alpha n} + (M_{uz}/M_{uz1})^{\alpha n} < 1.0$, Then O.K., If not , go for next trial increasing p.
12. Design of diameter and pitch of lateral ties:

    - The diameter of lateral ties should not be less than one forth diameter of largest longitudinal bar

$$\Phi_T \geq \phi/4$$

$$\geq 6mm$$

    - Pitch of ties:

      Pitch of ties $\leq$ least lateral dimension

      $\leq 16\ \phi_L$

      $\leq 300mm$

13. Check for ductility criteria

- The spacing of hoops shall not exceed half the LLD of column.
- Special confining reinforcement shall be provided over a length l_0 from each joint face

  > LLD

  >1/6 of clear span of member

  >450mm

- Lap splices shall be provided only in the central half of the member
- Hoops shall be provided over the entire splices spacing of which should be less than 150mm c/c at splices.

6.1.4 Foundation

Foundations are the structural elements that transfer loads from the buildings or individual columns to the earth. If these loads are to be properly transmitted, foundations must be designed to prevent excessive settlement or rotation, to minimize differential settlement and to provide adequate safety against sliding and overturning.

Most foundations may be classified as:

i. **Isolated footings**
ii. **Strip foundation and wall footings**
iii. **Combined footings**
iv. **Raft or mat foundation**
v. **Pile foundation**

The choice of type of foundations to be used in any given situation depends on a number of factors, such as:

- Soil strata
- Type of structure
- Type of loads
- Economy
- Bearing capacity and standard penetration test value N of soil.
- Permissible differential settlement, etc.

The choice is usually made from experience but it is advisable to carry out a comparative study of different designs to determine the most economical.

6.1.4.1 Depth of foundation

Depth of foundation is governed by the following objectives:

- to secure safe bearing capacity,
- to penetrate below the zone where seasonal weather changes are likely to cause significant movement due to swelling and shrinkage of soils, and
- to penetrate below the zone that may be affected by frost.

For footings with moments or eccentricity about both axes, the soil pressure at any point is given by the equation:

$$\sigma = \frac{P}{A} \pm \frac{My}{Iy}x \pm \frac{Mx}{Ix}y$$

Where,

I_x =M.O.I. of footing about X-axis

I_y =M.O.I of footing about X-axis

x = distance from Y-axis to the point of considerations

y = distance from X-axis to the point of considerations

There are situations where a footing must be built with a hole or notch and is thus unsymmetrical in plan about both axes. The soil pressure distribution in such rigid footings can be obtained from the principles of mechanics assuming linear distribution. The desired equation is as follows:

$$\sigma = \frac{P}{A} \pm \frac{MyIx - MxIxy}{IxIy - Ixy^2}(x) \pm \frac{MxIy - MyIxy}{IxIy - Ixy^2}(y)$$

Where,

I_x = M.O.I. of footing about X-axis

I_y = M.O.I. of footing about X-axis

x = distance from Y -axis to the point of considerations

y = distance from X-axis to the point of considerations

M_x = moment about X-axis

M_y = moment about Y -axis

I_{xy} = product of inertia, may be +ve or –ve

6.1.4.2 Raft foundation

In cases where individual footings would cover more than half of the area or the loads transmitted by the columns are too heavy, a continuous footing called a mat or raft foundation may be more suitable. These foundations are also used to reduce settlement in structures built on highly compressible deposits. When designing for earthquake forces along with other normal design forces, the permissible stress in material and allowable bearing pressure of the foundation soil can be increased by 33% and 50% respectively using the elastic method of design for mat foundations (as per Clause no.6.3.5 in IS: 1893-2016).

In case the columns are not equally spaced or their loads are not equal, moments of the loads can be taken about the same center of the base to determine the pressure distribution using the formula. However, these equations were derived from a rigid member and may lead to errors in pressure and resulting internal stresses if the eccentricity is significant since a raft is not a rigid member.

Raft weight is not considered in design as it's assumed to be carried by the sub-soil directly.

Mat foundation is preferred when the individual footings cover more than 50% of the plinth area. The depth of the footing is determined by considering bending moment, one-way shear, and two-way shear. Depth satisfying bending moment and two-way shear is chosen for economy, and the deficiency in capacity to resist one-way shear is compensated by providing shear reinforcement to the mat design.

The raft foundation is designed by dividing it into continuous strips centered on the column rows in both directions. The shear and bending moment are calculated using continuous beam analysis with moment distribution method.

Considering the reversal of seismic force, the maximum value of B.M. & shear force is taken for all strips and uniform thickness of raft is taken and reinforcement is provided uniformly throughout the entire mat.

Following are the steps are followed to design a mat foundation

1. Axial load for designed footing for load case 1.5 (Dead load +Live load) is taken from ETABS.
2. Find approximate area of footing
3. Find type of foundation
4. For raft foundation, find eccentricity.
5. Find soil pressure intensity at different points
6. Divide raft in different strips equivalent to beams.
7. Find maximum moments

   In X Direction $M = wl^2/10$

   In Y Direction $M = wl^2/10$

8. Depth of foundation slab required from moment criteria and punching shear criteria $d = Vu\tau v/bo$
9. Calculation of area of steel required

   $$M = 0.87 * f_y * Ast\ (d - (f_y * A_{st}/f_{ck} * b))$$

10. Check for minimum percentage of steel (0.12% of bD)

6.1.5 Staircase

Following steps are followed to design a staircase.

1. Assume thickness of waist slab
2. Calculation of load
3. Calculation of reactions
4. Calculation of maximum bending moment

   $B.M._{max}$= at distance X where shear force is zero.

5. Calculation of area of steel

$$M = 0.87*f_y*Ast(d–(f_y*A_{st}/f_{ck}*b))$$

6. Check for shear

$$Vu = W_uL_x / 2$$

$$\tau v = v/bd1$$

Percentage of steel,

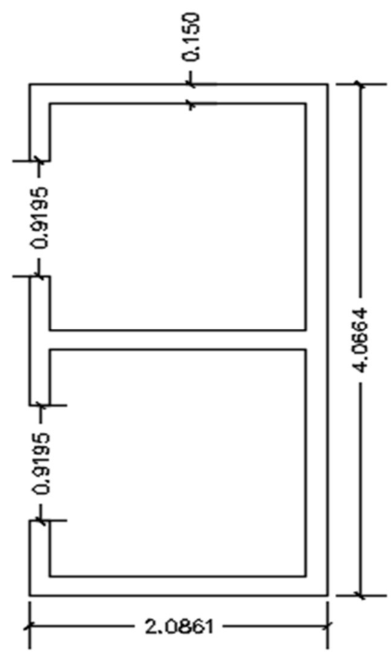
$$p \% = (A_{st}) / bd_1$$

For p % & M30 grade concrete, τ_c is taken from IS 456:2000, table 19 p.73 and k is taken as per IS 456:2000, Cl.40.2.1.1, p.72.

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

6.1.6 Lift Wall

The design of lift wall has been designed as the reinforced wall monolithic to the other structural members which is subjected to direct compression. They are designed as per the empirical procedure given in the IS 456-2000 Clause 32.2. The minimum thickness of the wall should be 100mm. the design of the wall shall account of the actual eccentricity of the vertical force subject to the min value of 0.05t. The vertical load transmitted to the wall by a discontinuous concrete floor or roof shall be assumed to act at one-third the depth of bearing area measured from the span face of the wall. Where there's an in-situ continuous concrete floor over the wall, load shall be assumed to act at the center of the wall. The resultant eccentricity of the total vertical load on a braced wall at any level between horizontal lateral supports shall be calculated on the assumption that the resultant eccentricity of all the vertical loads above the upper support is zero.



6.2 Detail design of Structural members

6.2.1 Detailed Slab Calculations

6.2.1.1 Interior Panel

Assumed overall depth of slab: 150 mm

C/C dimensions of selected slab panel: $l_{CX} = 3.375$m; $l_{CY} = 3.375$m

Step 1: Effective depth

From Preliminary design,

Overall Thickness = 130 mm

Provide clear cover= 15 mm and rebar diameter = 10 mm

Effective depth along short span (d) = 130-15-10/2 =110mm

Step 2: Effective span

In shorter span;

Lx= clear span + effective depth = (3.375 - 0.465/2 – 0.285/2) +0.11=3.11m < c/c length

In longer span;

Ly= clear span + effective depth = (6.553 - 0.43/2 – 0.285/2) +0.11=3.1275m < c/c length

So, Lx=3.11 m & Ly=3.1275m

Step 3: Slab type

Since, Ly/Lx = 1.005< 2 (So, it is a two-way slab)

Step 4: Design Load

- Floor finish = 1 kN/m^2
- Total dead load = 25 kN/m^3 *0.13= 3.25 kN/m^2
- Total live load = 3 kN/m^2
- Design load = 1.5 (DL+LL) = 10.875 kN/m^2

For unit width of slab, design load = 10.875 kN/m

Step 5: Bending Moment

-ve Bending Moment coefficient at continuous end

$\alpha_x = 0.03225$ $\alpha_y = 0.032$

+ve Bending Moment coefficient at mid span

$\alpha_x = 0.0242$ $\alpha_y = 0.024$

For short span,

Support Moment, Ms = - α_x w Lx^2 = -3.39 kNm

Mid Span Moment, Mm = α_x w Lx^2 = 2.545 kNm

For Long Span,

Support Moment, Ms = -α_y w Lx^2 = -3.365 kNm

Mid Span Moment, Mm = α_y w Lx^2 =2.5244 kNm

Step 6: Check for depth from moment consideration

For Fe 500 steel we have,

$$Mu, lim = \ 0.133\ fck\ bd^2$$

And for, Mu,lim = Mmax = 3.39×10^6 Nmm

d = 29.14 mm

Step 7: Area of steel

We have,

Ast_{min} = 0.12% of bD

= 0.12×1000×130/100

= 156 mm^2

Area of steel can be calculated solving the following equation,

$$Mu = \ 0.87\ fy\ Ast\ d\ (1 - \frac{fy\ Ast}{fck\ b\ d})$$

Area of steel along short span,

Area of steel along short span,

a. At supports

Ast =71.6236< Ast_{min}

Spacing = 1000 × (As/ Ast_{min}) = $1000\times(\pi\times10^2/4)/156$ = 503.460mm

Since, Spacing ≤ i. 3d

ii. 300 mm

Provide, 10 mm bars @ 150 mm c/c

Hence, $Ast_{,provided} = 1000\times(As/Spacing) = 523.598\ mm^2$

b. At mid span

$Ast = 53.62 < Ast_{min}$

Spacing $= 1000 \times (As/\ Ast_{min}) = 1000\times(\pi\times10^2/4)/156 = 503.460mm$

Since, Spacing ≤ i. 3d

ii. 300 mm

Provide, 10 mm bars @ 150 mm c/c

Hence, $Ast_{,provided} = 1000\times(As/Spacing) = 523.598\ mm^2$

Area of steel along long span,

a. At supports

$Ast = 71.089 < Ast_{min}$

Spacing $= 1000 \times (As/\ Ast_{min}) = 1000\times(\pi\times10^2/4)/156 = 503.460mm$

Since, Spacing ≤ i. 3d

ii. 300 mm

Provide, 10 mm bars @ 150 mm c/c

Hence, $Ast_{,provided} = 1000\times(As/Spacing) = 523.598\ mm^2$

b. At mid span

$Ast = 53.185 < Ast_{min}$

Spacing $= 1000 \times (As/\ Ast_{min}) = 1000\times(\pi\times10^2/4)/156 = 503.460mm$

Since, Spacing ≤ i. 3d

ii. 300 mm

Provide, 10 mm bars @ 150 mm c/c

Hence, $Ast_{,provided} = 1000\times(As/Spacing) = 523.598\ mm^2$

Step 8: Check for shear

For longer span

Shear force at the face of the support, $V = w\ Lx/2$

$= 10.875\times3.11/2$

$= 16.91\ kN$

Nominal Shear Stress $(\tau_v) = V/bd = 16.91\times1000/1000\times110$

$= 0.1537\ N/mm^2$

Design shear strength of concrete (τ_c)= 1.30×0.449 (for P_t = 0.4027%)

= 0.5837 N/mm^2

Maximum shear strength in for M30 grade concrete(τc(max)) =3.5 N/mm2

Thus, $\tau_v < \tau_c < \tau_{c(max)}$

So, it is safe in shear.

Step 9: Check for deflection

L/d ≤ $\alpha\beta\gamma\delta\lambda$
Basic Value, α = 26 (for continuous slab)
Span correction factor, β = 1
For tension reinforcement correction factor, γ
fs = 0.58×fy×Ast, required / Ast, provided
= 86.402
For, fs =79.338 and Pt =0.4027%
γ = 2
Compressive reinforcement modification factor, δ = 1

Reduction factors in for span to effective depth for flanged section, λ = 1

(L/d) max = 26*1*2 *1*1=52

(L/d) provided =28.277 < (L/d) max
Hence, ok.

Step 10: Check for development length

Ld =Φ* 0.87 fy / 4*τbd = 453.25mm

$$Mu = 0.87\, fy\, Ast\, d\, (1 - \frac{fy\, Ast}{fck\, b\, d})$$

Assume, Lo=8 Φ=80mm

1.3 M/V = 924.83 mm >Ld
No anchorage/bend required

Provide clear cover= 15 mm and rebar diameter = 10 mm
Effective depth along short span (d) = 130-15-10/2 =110mm

distance of 900 mm.

6.2.2 Beam Design

BEAM DETAILS

Width (b) =	400	mm
Overall Depth (D) =	600	mm
Effective Clear Cover (d') =	55	mm
Effective Depth (d) =	545	mm
Grade of Concrete (f_{ck}) =	30	N/mm^2
Yield Strength of Steel (f_y) =	500	N/mm^2
Rebar Diameter (Ø) =	20	mm
Beam Name :	B37	basement floor
Length of Beam =	6.75	m

Minimum Area Reqd(A_{st},min)

IS 456:2000 26.5.1.1

A_{st}, min =

A_{st}, min= 370.60 mm^2

IS 13920 6.2.1

A_{st}, min *b*D

A_{st}, min= 631.0 mm^2

So adopt maximum of above A_{st},min

A_{st}, min= 631.0 mm^2

Maximum Area Required (A_{st},max)

$A_{st,max}$ = 0.04*b*D

A_{st}, max= 9600 mm^2

Maximum Area Required (A_{sc},max)

$A_{sc,max}$ = 0.04*b*D

A_{sc}, max= 9600 mm^2

IS 456:2000 Cl 38

Formula

=

= 0.46

where $X_{u,max}$ is the limiting value of the depth of neutral axis for given grade of steel.

IS 456:2000, Cl G-1.1 c)

Limiting Moment ($M_{u,lim}$)

$$M_{u,lim}\ b\ d^2 f_{ck}$$

$M_{u,lim}$ = 478.921 KN-m

Position	***I (top)***
Governing Combination =	Envelope

From ETABS

Factored Moment (M_u) = 187.39087 KN-m (negative)

Factored Torsion (T_u) = 0.2863 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u$$

M_t = 0.421 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 187.8119 KN-m

Since M<M_{ul}, design as singly reinforced section

$$e_{sc} = (x_{ul} - d')$$

Where,

x_{ul}= 248.874612 mm

d'= 55 mm

We get,

e_{sc}= 0.0027

SP16 (Table A)

Interpolating,

e_{sc}	f_{sc}
0.00277	413
0.0027	?
0.00312	423.9

We,get

f_{sc} = 411.64585 N/mm^2

From IS 456:2000 Cl G-1.1 b)

$$A_{sc}$$

A_{sc}= 0 mm^2

M_{ul} =0.87 * f_y * A_{st1} *(d-)

Sloving,

A_{st1}= 847.515204 mm^2

A_{st2}

= 0 mm^2

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

= 847.515204 mm^2

Area of Steel (A_{st}) = 847.515 mm2 Top

Percentage of Steel (%) = 0.353 %

Area of compression steel(A_{sc}) = 630.976 mm^2 Bottom

% of compression steel= 0.263 %

Position ***Middle (Bottom)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 167.4829 KN-m (positive)

Factored Torsion (T_u) = -0.1994 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$M_t = T_u$

M_t= -0.293 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 167.7761 KN-m

Since M<M_{ul}, design as singly reinforced section

e_{sc} = (x_{ul} - d')

Where,

x_{ul}= 248.874612 mm

d'= 55 mm

We get,

e_{sc}= 0.0027

SP16 (Table A)

Interpolating,

e_{sc}	f_{sc}
0.00277	413
0.0027	?
0.00312	423.9

We,get

f_{sc} = 411.64585 N/mm^2

From IS 456:2000 Cl G-1.1 b)

A_{sc}

A_{sc}= 0 mm^2

M_{ul} =0.87 * f_y * A_{st1} *(d-)

Solving,

A_{st1}= 751.186881 mm^2

A_{st2}

A_{st2} = 0 mm^2

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

= 751.186881 mm^2

Area of Steel (A_{st}) =	751.187	mm^2	Bottom
:centage of Steel (%) =	0.313	%	
ımpression steel(A_{sc}) =	375.593	mm^2	Top
of compression steel=	0.156	%	
Position	***J (top)***		
/erning Combination =	Envelope		

From ETABS

:tored Moment (M_u) =	209.3316	KN-m	(negative)
'actored Torsion (T_u) =	0.3976	KN-m	

IS 456:2000, Cl 41.4.2

ɔment due to Torsion (M_t)

M_t =T_u

M_t = 0.585 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t =$ 209.9163 KN-m

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

$e_{sc} = (x_{ul} - d')$

Where,

$x_{ul}=$ 248.874612 mm

$d'=$ 55 mm

We get,

$e_{sc}=$ 0.0027

SP16 (Table A)

Interpolating,

e_{sc}	f_{sc}
0.00277	413
0.0027	?
0.00312	423.9

We,get

$f_{sc} =$ 411.64585 N/mm^2

From IS 456:2000 Cl G-1.1 b)

A_{sc}

$A_{sc}=$ 0 mm^2

$M_{ul} = 0.87 * f_y * A_{st1} *(d-)$

Solving,

$A_{st1}=$ 955.716705 mm^2

A_{st2}

$=$ 0 mm^2

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

$=$ 955.716705 mm^2

Area of Steel (A_{st}) = 955.717 mm^2 Top

Percentage of Steel (%) = 0.398 %

Area of compression steel(A_{sc}) = 630.976 mm^2 Bottom

% of compression steel= 0.263 %

Check for Deflection

IS 456-2000 cl.23.2.1

$\le \alpha\beta\gamma\delta\lambda$

clear span=	6750	mm
width of support=	750	mm
1/12 of clear span=	562.50	mm

Since,

width of support > 1/12 of clear span so L_x is taken as clear span

effective length(L_x)=	6750	mm
α=	26	
β=	1	span less than 10 m
λ=	1	not a flanged section

For γ

A_{sc} provided=	630.976	mm^2
% A_{sc} provided=	0.57	%

IS 456-2000 cl.23.2.1 fig 5

So,

δ= 1.15

For λ

fs= $\boxed{0.58\, f_y \dfrac{Area\ of\ Steel\ \mathrm{Re}quired}{Area\ of\ Steel\ \mathrm{Pr}ovided}}$

A_{st} required=	955.717	mm^2
A_{st} provided=	981.7	mm^2

So,

fs=	282.324	N/mm^2
%st=	0.481	%

IS 456-2000 cl.23.2.1 fig 4

γ= 1.4

So,

αβγδλ= 41.86

= 16.463 ≤ αβγδλ **(OK)**

Check for Development Length:

IS 456-2000 cl.26.2.1

$\boxed{Ld = \dfrac{\Phi\sigma_s}{4\ x\ \tau_{bd}} =}$ 906.25 mm for tension

$$\boxed{Ld = \frac{\Phi \sigma_s}{4 \ x \ \tau_{bd}} =}$$ 725 mm for compression

φ=	25	mm	(nominal diameter of bar)
σ_s=0.87*f_y=	435	N/mm^2	(stress in bars)
τ_{bd}=	1.5*1.6	N/mm^2	(design bond stress for tension)
τ_{bd}=	1.5*1.6*1.25	N/mm^2	(design bond stress for compression)

Also,

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

Where, M =0.87 * f_y * A_{stprvd} *(d-)

A_{st} provided=	981.7	mm^2	
M_1=	144032515	N-mm	(MOR offered by tension steel provided)
V=	354440	KN	(maximum shear force at that face)

-377.97 **<Ld Not OK**

No anchorage is provided

10 mm bar @ 100c/c spacing is provided over length 2d ie 1100 from either side of beam joints (IS 13920 cl 6.3.5)

BASEMENT **B38**

1. Input Details

Grade of Concrete (fck)	Strength of Steel (fy)	Beam Width (B)	Beam Depth (D)	Beam Length (L)	Slab Width	Effective Cover
30 N/mm2	500 N/mm2	400.0 mm	600.0 mm	6750.0 mm	130.0 mm	55.0 mm

Bar Diameter	Effective Depth(d)	Effective Cover (d')	B/D ratio	L/D ratio	Remarks
25.0 mm	545.0 mm	55.0 mm	0.67	11.25	IS 13920:2016
			Okay	Okay	Clause 6.1.1 & 6.1.3

Maximum Reinforcement	Minimum Reinforcement	Limiting Moment	(Pt lim)	Remarks
6000.0 mm2	631.0 mm2	476.19 kNm	1.132%	IS 13920:2016
0.025*B*d	bd			Clause 6.2.1-b & Clause 6.2.2

2. Design for Flexure

At left end

For hogging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
390.5 kNm	0.2 kNm	390.767 kNm	Singly Reinforced	-	0 mm2	1935 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 968 mm2

For sagging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
105.9 kNm	0.3 kNm	106.4 kNm	Singly Reinforced	-	0 mm2	631 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 631 mm2

At middle

For hogging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
0.0 kNm	0.0 kNm	0.046 kNm	Singly Reinforced	-	0 mm2	631 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 631 mm2

For sagging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
165.3 kNm	0.0 kNm	165.3 kNm	Singly Reinforced	-	0 mm2	740 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 631 mm2

At right end

For hogging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
390.9 kNm	0.3 kNm	391.333 kNm	Singly Reinforced	-	0 mm2	1938 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 969 mm2

For sagging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
125.0 kNm	0.2 kNm	125.3 kNm	Singly Reinforced	-	0 mm2	631 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 631 mm2

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	390.5 kNm	0.0 kNm	390.9 kNm
Torsional moment(kNm),Tu	0.2 kNm	0.0 kNm	0.3 kNm
Mt	0.247 kNm	0.046 kNm	0.433 kNm
Me1=Mt + Mu	390.8 kNm	0.0 kNm	391.3 kNm
Ast at top required (mm2)	1935 mm2	631 mm2	1938 mm2
Asc at bottom required	968 mm2	631 mm2	969 mm2
	Moment (kNm)		
Sagging moment(kNm),Mu	105.9 kNm	165.3 kNm	125.0 kNm
Torsional moment(kNm),Tu	0.3 kNm	0.0 kNm	0.2 kNm
Mt	0.429 kNm	0.064 kNm	0.280 kNm
Me1=Mt + Mu	106.4 kNm	165.3 kNm	125.3 kNm
Ast at bottom required (mm2)	631 mm2	740 mm2	631 mm2
Asc at top required	631 mm2	631 mm2	631 mm2

Summary of required reinforcement			
	Left	**Mid**	**Right**
Top	1935 mm2	631 mm2	1938 mm2
Bottom	968 mm2	740 mm2	969 mm2

Summary of reinforcement bars						
Top						
	Left		**Mid**		**Right**	
Bar diameter	25 mm	20 mm		25 mm	25 mm	20 mm
Number	4	0		2	4	0
Total area	1963.5 mm2		981.7 mm2		1963.5 mm2	
	Pt	0.818%	Pc	0.409%	Pt	0.818%
	Okay		Okay		Okay	
Bottom						
	Left		**Mid**		**Right**	
Bar diameter	25 mm	20 mm	25 mm		25 mm	20 mm
Number	2	0	3		2	0
Total area	981.7 mm2		1472.6 mm2		981.7 mm2	
	Pc	0.409%	Pt	0.614%	Pc	0.409%
	Okay		Okay		Okay	

Design for shear

Left		Mid		Right	
Shear Force	**Torsion**	**Shear Force**	**Torsion**	**Shear Force**	**Torsion**
-288.88 kN	0.3 kNm	-79.15 kN	0.0 kNm	288.87 kN	0.3 kNm
Equivalent Shear	290.05 kN	Equivalent Shear	79.32 kN	Equivalent Shear	290.05 kN

Shear force due to formation of plastic hinge at both ends		
VaD+L	-222.28 kN	Shear at end A, due to dead and live load with partial safety factor of 1.2 on loads
VbD+L	223.03 kN	Shear at end B,due to dead and live load with partial safety factor of 1.2 on loads

Mu,limAs	Sagging moment of resistance at left end					
Mu,limAh	Hogging moment of resistance at left end					
Section type	Pt	Pc	Mu,limAh/bd2	Mu,limAh	Mu,limAs/bd2	Mu,limAs
Singly reinforced	0.818%	0.409%	3.05	362.4 kNm	1.657142857	197 kNm

Table 4(Sp-16)

Mu,limBs	Sagging moment of resistance at right end					
Mu,limBh	Hogging moment of resistance at right end					
Section type	Pt	Pc	Mu,limBh/bd2	Mu,limBh	Mu,limBs/bd2	Mu,limBs
Singly reinforced	0.818%	0.409%	3.05	362.4 kNm	1.657142857	197 kNm

Sway to right		Sway to left	
Vu,a	Vu,b	Vu,a	Vu,b
-352.77 kN	353.52 kN	-352.77 kN	353.52 kN

Clear span
6.000 m

Hence design shear force are

At left	At middle	At right
352.77 kN	79.32 kN	353.52 kN

At ends, provide 2 legged vertical stirrups of 10mm dia @100 mm c-c.

At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

Check for Development length:		
	Tension	Compression
τbd	1.5 N/mm2	1.5 N/mm2
Diameter	25 mm	25 mm
Development length	1133 mm	906 mm
	Tension	Compression
Moment	362.4 kNm	197 kNm
Shear	353.52 kN	353.52 kN
Anchorage Length	**200 mm**	**0 mm**

BEAM DESIGN

BASEMENT B17

1. Input Details

Grade of Concrete (fck)	Strength of Steel (fy)	Beam Width (B)	Beam Depth (D)	Beam Length (L)	Slab Width	Effective Cover
30 N/mm2	500 N/mm2	400.0 mm	600.0 mm	6750.0 mm	130.0 mm	55.0 mm

Bar Diameter	Effective Depth(d)	Effective Cover (d')	B/D ratio	L/D ratio	Remarks
25.0 mm	545.0 mm	55.0 mm	0.67	11.25	IS 13920:2016
			Okay	Okay	Clause 6.1.1 & 6.1.3

Maximum Reinforcement	Minimum Reinforcement	Limiting Moment	(Pt lim)	Remarks
6000.0 mm2	631.0 mm2	476.19 kNm	1.132%	IS 13920:2016
0.025*B*d	bd			Clause 6.2.1-b & Clause 6.2.2

2. Design for Flexure

At left end

For hogging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
366.2 kNm	0.3 kNm	366.551 kNm	Singly Reinforced	-	0 mm2	1792 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 896 mm2

For sagging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
151.0 kNm	0.2 kNm	151.4 kNm	Singly Reinforced	-	0 mm2	674 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 631 mm2

At middle

For hogging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
0.0 kNm	0.0 kNm	0.054 kNm	Singly Reinforced	-	0 mm2	631 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 631 mm2

For sagging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
141.2 kNm	0.0 kNm	141.2 kNm	Singly Reinforced	-	0 mm2	631 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 631 mm2

At right end

For hogging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
366.1 kNm	0.3 kNm	366.5 kNm	Singly Reinforced	-	0 mm2	1792 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 896 mm2

For sagging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
143.8 kNm	0.2 kNm	144.1 kNm	Singly Reinforced	-	0 mm2	639 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 631 mm2

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	366.2 kNm	0.0 kNm	366.1 kNm
Torsional moment(kNm),Tu	0.3 kNm	0.0 kNm	0.3 kNm
Mt	0.401 kNm	0.054 kNm	0.401 kNm
Me1=Mt + Mu	366.6 kNm	0.1 kNm	366.5 kNm
Ast at top required (mm2)	1792 mm2	631 mm2	1792 mm2
Asc at bottom required	896 mm2	631 mm2	896 mm2
	Moment (kNm)		
Sagging moment(kNm),Mu	151.0 kNm	141.2 kNm	143.8 kNm
Torsional moment(kNm),Tu	0.2 kNm	0.0 kNm	0.2 kNm
Mt	0.346 kNm	0.069 kNm	0.335 kNm
Me1=Mt + Mu	151.4 kNm	141.2 kNm	144.1 kNm
Ast at bottom required (mm2)	674 mm2	631 mm2	639 mm2
Asc at top required	631 mm2	631 mm2	631 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	1792 mm2	631 mm2	1792 mm2
Bottom	896 mm2	631 mm2	896 mm2

Summary of reinforcement bars						
Top						
	Left		Mid		Right	
Bar diameter	25 mm	20 mm		25 mm	25 mm	20 mm
Number	4	0		2	4	0
Total area	1963.5 mm2		981.7 mm2		1963.5 mm2	
	Pt	0.818%	Pc	0.409%	Pt	0.818%
	Okay		Okay		Okay	
Bottom						
	Left		Mid		Right	
Bar diameter	25 mm	20 mm	25 mm		25 mm	20 mm
Number	2	0	2		2	0
Total area	981.7 mm2		981.7 mm2		981.7 mm2	
	Pc	0.409%	Pt	0.409%	Pc	0.409%
	Okay		Okay		Okay	

Design for shear

Left		Mid		Right	
Shear Force	Torsion	Shear Force	Torsion	Shear Force	Torsion
-241.77 kN	0.3 kNm	-75.36 kN	0.0 kNm	261.02 kN	0.3 kNm
Equivalent Shear	242.86 kN		75.55 kN		262.11 kN

Shear force due to formation of plastic hinge at both ends		
VaD+L	-143.54 kN	Shear at end A, due to dead and live load with partial safety factor of 1.2 on loads
VbD+L	164.26 kN	Shear at end B,due to dead and live load with partial safety factor of 1.2 on loads

Mu,limAs	Sagging moment of resistance at left end					
Mu,limAh	**Hogging moment of resistance at left end**					
Section type	**Pt**	**Pc**	**Mu,limAh/bd2**	**Mu,limAh**	**Mu,limAs/bd2**	**Mu,limAs**
Singly reinforced	0.818%	0.409%	3.06875	364.6 kNm	1.657142857	197 kNm

Table 4(Sp-16)

Mu,limBs	Sagging moment of resistance at right end					
Mu,limBh	**Hogging moment of resistance at right end**					
Section type	**Pt**	**Pc**	**Mu,limBh/bd2**	**Mu,limBh**	**Mu,limBs/bd2**	**Mu,limBs**
Singly reinforced	0.818%	0.409%	3.06875	364.6 kNm	1.657142857	197 kNm

Sway to right		Sway to left	
Vu,a	**Vu,b**	**Vu,a**	**Vu,b**
-274.55 kN	295.27 kN	-274.55 kN	295.27 kN

Clear span
6.000 m

Hence design shear force are

At left	At middle	At right
274.55 kN	75.55 kN	295.27 kN

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.

At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

Check for Development length:		
	Tension	**Compression**
τbd	1.5 N/mm2	1.5 N/mm2
Diameter	25 mm	25 mm
Development length	1133 mm	906 mm
	Tension	**Compression**
Moment	364.6 kNm	197 kNm
Shear	295.27 kN	295.27 kN
Anchorage Length	**472 mm**	**0 mm**

BEAM DESIGN

BASEMENT SECONDARY B58

1. Input Details

Grade of Concrete (fck)	Strength of Steel (fy)	Beam Width (B)	Beam Depth (D)	Beam Length (L)	Slab Width	Effective Cover
30 N/mm2	500 N/mm2	300.0 mm	450.0 mm	6750.0 mm	130.0 mm	50.0 mm

Bar Diameter	Effective Depth(d)	Effective Cover (d')	B/D ratio	L/D ratio	Remarks
25.0 mm	400.0 mm	50.0 mm	0.67	15.00	IS 13920:2016
			Okay	Okay	Clause 6.1.1 & 6.1.3

Maximum Reinforcement	Minimum Reinforcement	Limiting Moment	(Pt lim)	Remarks
3375.0 mm2	354.9 mm2	192.38 kNm	1.132%	IS 13920:2016
0.025*B*d	bd			Clause 6.2.1-b & Clause 6.2.2

2. Design for Flexure

At left end

For hogging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
3.8 kNm	0.0 kNm	3.794 kNm	Singly Reinforced	-	0 mm2	355 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 355 mm2

For sagging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
0.0 kNm	0.0 kNm	0.0 kNm	Singly Reinforced	-	0 mm2	355 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 355 mm2

At middle

For hogging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
0.0 kNm	0.0 kNm	0.000 kNm	Singly Reinforced	-	0 mm2	355 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 355 mm2

For sagging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
19.0 kNm	0.0 kNm	19.1 kNm	Singly Reinforced	-	0 mm2	355 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 355 mm2

At right end

For hogging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
2.7 kNm	0.1 kNm	2.8 kNm	Singly Reinforced	-	0 mm2	355 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 355 mm2

For sagging moment

Moment (Mu)	Tu	Me1	Section Type	Fsc	Asc Min	Ast Min
0.0 kNm	0.1 kNm	0.1 kNm	Singly Reinforced	-	0 mm2	355 mm2

Area of Steel in Compression (Asc) must be at least 50% of Area of Steel in Tension (Ast)

Area of Steel in Compression (Asc) = 355 mm2

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	3.8 kNm	0.0 kNm	2.7 kNm
Torsional moment(kNm),Tu	0.0 kNm	0.0 kNm	0.1 kNm
Mt	0.000 kNm	0.000 kNm	0.117 kNm
Me1=Mt + Mu	3.8 kNm	0.0 kNm	2.8 kNm
Ast at top required (mm2)	355 mm2	355 mm2	355 mm2
Asc at bottom required	355 mm2	355 mm2	355 mm2
	Moment (kNm)		
Sagging moment(kNm),Mu	0.0 kNm	19.0 kNm	0.0 kNm
Torsional moment(kNm),Tu	0.0 kNm	0.0 kNm	0.1 kNm
Mt	0.000 kNm	0.058 kNm	0.117 kNm
Me1=Mt + Mu	0.0 kNm	19.1 kNm	0.1 kNm
Ast at bottom required (mm2)	355 mm2	355 mm2	355 mm2
Asc at top required	355 mm2	355 mm2	355 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	355 mm2	355 mm2	355 mm2
Bottom	355 mm2	355 mm2	355 mm2

Summary of reinforcement bars						
Top						
	Left		Mid		Right	
Bar diameter	20 mm	20 mm		20 mm	20 mm	20 mm
Number	2	0		2	2	0
Total area	628.3 mm2		628.3 mm2		628.3 mm2	
	Pt	0.465%	Pc	0.465%	Pt	0.465%
	Okay		Okay		Okay	
Bottom						
	Left		Mid		Right	
Bar diameter	20 mm	20 mm	20 mm		20 mm	20 mm
Number	2	0	2		2	0
Total area	628.3 mm2		628.3 mm2		628.3 mm2	
	Pc	0.465%	Pt	0.465%	Pc	0.465%
	Okay		Okay		Okay	

Design for shear

Left		Mid		Right	
Shear Force	**Torsion**	**Shear Force**	**Torsion**	**Shear Force**	**Torsion**
11.06 kN	0.0 kNm	-2.68 kN	0.0 kNm	6.21 kN	0.1 kNm
Equivalent Shear	11.06 kN		2.89 kN		6.63 kN

Shear force due to formation of plastic hinge at both ends		
VaD+L	14.86 kN	Shear at end A, due to dead and live load with partial safety factor of 1.2 on loads
VbD+L	9.33 kN	Shear at end B,due to dead and live load with partial safety factor of 1.2 on loads

Mu,limAs	Sagging moment of resistance at left end					
Mu,limAh	Hogging moment of resistance at left end					
Section type	Pt	Pc	Mu,limAh/bd2	Mu,limAh	Mu,limAs/bd2	Mu,limAs
Singly reinforced	0.465%	0.465%	1.1871	57.0 kNm	1.1871	57 kNm

Table 4(Sp-16)

Mu,limBs	Sagging moment of resistance at right end					
Mu,limBh	Hogging moment of resistance at right end					
Section type	Pt	Pc	Mu,limBh/bd2	Mu,limBh	Mu,limBs/bd2	Mu,limBs
Singly reinforced	0.465%	0.465%	1.1871	57.0 kNm	1.1871	57 kNm

Sway to right		Sway to left	
Vu,a	Vu,b	Vu,a	Vu,b
-10.26 kN	34.46 kN	-10.26 kN	34.46 kN

Clear span
6.350 m

Hence design shear force are

At left	At middle	At right
11.06 kN	2.89 kN	34.46 kN

At ends, provide 2 legged vertical stirrups of 10mm dia @200mm c-c.

At mid point, Provide 2-legged , 10mm Φ stirrups @ 200 mm c/c.

Check for Development length:		
	Tension	Compression
τbd	1.5 N/mm2	1.5 N/mm2
Diameter	25 mm	25 mm
Development length	1133 mm	906 mm
	Tension	Compression
Moment	57.0 kNm	57 kNm
Shear	34.46 kN	34.46 kN
Anchorage Length	**400 mm**	**50 mm**

BEAM DESIGN SUMMARY

FIRST FLOOR B39

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	206.0404	0.0 kNm	206.7 kNm
Torsional moment(kNm),Tu	0.0 kNm	0.0 kNm	0.0 kNm
Mt	0.056 kNm	0.1 kNm	0.037 kNm
Me1=Mt + Mu	206.1 kNm	0.1 kNm	206.8 kNm
Ast at top required (mm2)	937 mm2	631 mm2	940 mm2
Asc at bottom required	631 mm2	631 mm2	631 mm2
	Moment (kNm)		
Sagging moment(kNm),Mu	0.0 kNm	253.6 kNm	0.0 kNm
Torsional moment(kNm),Tu	0.0 kNm	0.0 kNm	0.0 kNm
Mt	0.037 kNm	0.037 kNm	0.057 kNm
Me1=Mt + Mu	0.0 kNm	253.7 kNm	0.1 kNm
Ast at bottom required (mm2)	631 mm2	1176 mm2	631 mm2
Asc at top required	631 mm2	631 mm2	631 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	937 mm2	631 mm2	940 mm2
Bottom	631 mm2	1176 mm2	631 mm2

Summary of reinforcement provided			
	Left	Mid	Right
Top	2-25mm	2-25mm	2-25mm
Bottom	2-25mm	3-25mm	2-25mm

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.
At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

FIRST FLOOR B15

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	277.5 kNm	0.0 kNm	262.1 kNm
Torsional moment(kNm),Tu	0.0 kNm	0.0 kNm	0.0 kNm
Mt	0.015 kNm	0.063 kNm	0.063 kNm
Me1=Mt + Mu	277.5 kNm	0.1 kNm	262.1 kNm
Ast at top required (mm2)	1300 mm2	631 mm2	1220 mm2
Asc at bottom required	650 mm2	631 mm2	631 mm2

	Moment (kNm)		
Sagging moment(kNm),Mu	0.0 kNm	337.0 kNm	0.0 kNm
Torsional moment(kNm),Tu	0.0 kNm	0.0 kNm	0.0 kNm
Mt	0.061 kNm	0.018 kNm	0.018 kNm
Me1=Mt + Mu	0.1 kNm	337.1 kNm	0.0 kNm
Ast at bottom required (mm2)	631 mm2	1624 mm2	631 mm2
Asc at top required	631 mm2	812 mm2	631 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	1300 mm2	812 mm2	1220 mm2
Bottom	650 mm2	1624 mm2	631 mm2

Summary of reinforcement provided			
	Left	Mid	Right
Top	3-25mm	2-25mm	3-25mm
Bottom	2-25mm	4-25mm	2-25mm

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.
At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

SECOND FLOOR B27

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	430.2 kNm	0.0 kNm	370.2 kNm
Torsional moment(kNm),Tu	0.0 kNm	0.0 kNm	0.1 kNm
Mt	0.069 kNm	0.000 kNm	0.120 kNm
Me1=Mt + Mu	430.2 kNm	0.0 kNm	370.4 kNm
Ast at top required (mm2)	1897 mm2	735 mm2	1598 mm2
Asc at bottom required	949 mm2	735 mm2	799 mm2
	Moment (kNm)		
Sagging moment(kNm),Mu	0.0 kNm	333.1 kNm	0.0 kNm
Torsional moment(kNm),Tu	0.0 kNm	0.0 kNm	0.0 kNm
Mt	0.049 kNm	0.025 kNm	0.008 kNm
Me1=Mt + Mu	0.0 kNm	333.1 kNm	0.0 kNm
Ast at bottom required (mm2)	735 mm2	1419 mm2	735 mm2
Asc at top required	735 mm2	735 mm2	735 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	1897 mm2	735 mm2	1598 mm2
Bottom	949 mm2	1419 mm2	799 mm2

Summary of reinforcement provided			
	Left	Mid	Right
Top	4-25mm	2-25mm	4-25mm
Bottom	2-25mm	3-25mm	2-25mm

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.
At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

SECOND FLOOR B43

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	396.6 kNm	0.0 kNm	395.2 kNm
Torsional moment(kNm),Tu	0.0 kNm	0.3 kNm	0.1 kNm
Mt	0.000 kNm	0.477 kNm	0.101 kNm
Me1=Mt + Mu	396.6 kNm	0.5 kNm	395.3 kNm
Ast at top required (mm2)	1727 mm2	735 mm2	1720 mm2
Asc at bottom required	864 mm2	735 mm2	860 mm2
	Moment (kNm)		
Sagging moment(kNm),Mu	183.9 kNm	128.6 kNm	204.3 kNm
Torsional moment(kNm),Tu	0.1 kNm	0.0 kNm	0.0 kNm
Mt	0.191 kNm	0.000 kNm	0.000 kNm
Me1=Mt + Mu	184.1 kNm	128.6 kNm	204.3 kNm
Ast at bottom required (mm2)	748 mm2	735 mm2	835 mm2
Asc at top required	735 mm2	735 mm2	735 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	1727 mm2	735 mm2	1720 mm2
Bottom	864 mm2	735 mm2	860 mm2

Summary of reinforcement provided			
	Left	Mid	Right
Top	4-25mm	2-25mm	4-25mm
Bottom	2-25mm	2-25mm	2-25mm

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.
At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

THIRD FLOOR B39

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	213.0 kNm	0.0 kNm	216.3 kNm
Torsional moment(kNm),Tu	0.1 kNm	0.1 kNm	0.0 kNm
Mt	0.088 kNm	0.088 kNm	0.068 kNm
Me1=Mt + Mu	213.1 kNm	0.1 kNm	216.4 kNm
Ast at top required (mm2)	972 mm2	631 mm2	988 mm2
Asc at bottom required	631 mm2	631 mm2	631 mm2

	Moment (kNm)		
Sagging moment(kNm),Mu	0.0 kNm	237.4 kNm	0.0 kNm
Torsional moment(kNm),Tu	0.0 kNm	0.0 kNm	0.1 kNm
Mt	0.069 kNm	0.069 kNm	0.086 kNm
Me1=Mt + Mu	0.1 kNm	237.5 kNm	0.1 kNm
Ast at bottom required (mm2)	631 mm2	1094 mm2	631 mm2
Asc at top required	631 mm2	631 mm2	631 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	972 mm2	631 mm2	988 mm2
Bottom	631 mm2	1094 mm2	631 mm2

Summary of reinforcement provided			
	Left	Mid	Right
Top	3-25mm	2-25mm	3-25mm
Bottom	2-25mm	3-25mm	2-25mm

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.
At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

THIRD FLOOR B15

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	266.9 kNm	0.0 kNm	246.1 kNm
Torsional moment(kNm),Tu	0.0 kNm	0.0 kNm	0.0 kNm
Mt	0.062 kNm	0.050 kNm	0.050 kNm
Me1=Mt + Mu	267.0 kNm	0.1 kNm	246.1 kNm
Ast at top required (mm2)	1245 mm2	631 mm2	1137 mm2
Asc at bottom required	631 mm2	631 mm2	631 mm2
	Moment (kNm)		
Sagging moment(kNm),Mu	0.0 kNm	305.9 kNm	0.0 kNm
Torsional moment(kNm),Tu	0.0 kNm	0.0 kNm	0.0 kNm
Mt	0.045 kNm	0.063 kNm	0.063 kNm
Me1=Mt + Mu	0.0 kNm	306.0 kNm	0.1 kNm
Ast at bottom required (mm2)	631 mm2	1452 mm2	631 mm2
Asc at top required	631 mm2	726 mm2	631 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	1245 mm2	726 mm2	1137 mm2
Bottom	631 mm2	1452 mm2	631 mm2

Summary of reinforcement provided			
	Left	Mid	Right
Top	3-25mm	2-25mm	3-25mm
Bottom	2-25mm	4-25mm	2-25mm

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.
At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

FOURTH FLOOR B38

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	247.2 kNm	0.0 kNm	243.9 kNm
Torsional moment(kNm),Tu	1.7 kNm	0.3 kNm	1.7 kNm
Mt	2.561 kNm	0.402 kNm	2.545 kNm
Me1=Mt + Mu	249.8 kNm	0.4 kNm	246.4 kNm
Ast at top required (mm2)	1156 mm2	631 mm2	1139 mm2
Asc at bottom required	631 mm2	631 mm2	631 mm2
	Moment (kNm)		
Sagging moment(kNm),Mu	12.3 kNm	128.3 kNm	92.8 kNm
Torsional moment(kNm),Tu	1.7 kNm	0.3 kNm	1.7 kNm
Mt	2.561 kNm	0.414 kNm	2.523 kNm
Me1=Mt + Mu	14.9 kNm	128.7 kNm	95.3 kNm
Ast at bottom required (mm2)	631 mm2	631 mm2	631 mm2
Asc at top required	631 mm2	631 mm2	631 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	1156 mm2	631 mm2	1139 mm2
Bottom	631 mm2	631 mm2	631 mm2

Summary of reinforcement provided			
	Left	Mid	Right
Top	3-25mm	2-25mm	3-25mm
Bottom	2-25mm	2-25mm	2-25mm

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.
At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

FOURTH FLOOR B20

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	251.8 kNm	0.0 kNm	255.0 kNm
Torsional moment(kNm),Tu	2.3 kNm	0.2 kNm	1.3 kNm
Mt	3.358 kNm	0.319 kNm	1.886 kNm
Me1=Mt + Mu	255.2 kNm	0.3 kNm	256.9 kNm
Ast at top required (mm2)	1184 mm2	631 mm2	1193 mm2
Asc at bottom required	631 mm2	631 mm2	631 mm2

	Moment (kNm)		
Sagging moment(kNm),Mu	125.3 kNm	167.6 kNm	118.4 kNm
Torsional moment(kNm),Tu	1.1 kNm	0.1 kNm	2.2 kNm
Mt	1.588 kNm	0.142 kNm	3.240 kNm
Me1=Mt + Mu	126.9 kNm	167.8 kNm	121.7 kNm
Ast at bottom required (mm2)	631 mm2	751 mm2	631 mm2
Asc at top required	631 mm2	631 mm2	631 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	1184 mm2	631 mm2	1193 mm2
Bottom	631 mm2	751 mm2	631 mm2

Summary of reinforcement provided			
	Left	Mid	Right
Top	3-25mm	2-25mm	3-25mm
Bottom	2-25mm	2-25mm	2-25mm

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.
At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

TOP FLOOR B61

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	290.5 kNm	0.0 kNm	319.9 kNm
Torsional moment(kNm),Tu	9.3 kNm	0.7 kNm	3.1 kNm
Mt	13.789 kNm	1.074 kNm	4.652 kNm
Me1=Mt + Mu	304.3 kNm	1.1 kNm	324.6 kNm
Ast at top required (mm2)	1283 mm2	735 mm2	1378 mm2
Asc at bottom required	735 mm2	735 mm2	735 mm2
	Moment (kNm)		
Sagging moment(kNm),Mu	123.1 kNm	221.8 kNm	123.9 kNm
Torsional moment(kNm),Tu	0.9 kNm	2.5 kNm	4.8 kNm
Mt	1.322 kNm	3.711 kNm	7.025 kNm
Me1=Mt + Mu	124.4 kNm	225.5 kNm	130.9 kNm
Ast at bottom required (mm2)	735 mm2	928 mm2	735 mm2
Asc at top required	735 mm2	735 mm2	735 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	1283 mm2	735 mm2	1378 mm2
Bottom	735 mm2	928 mm2	735 mm2

Summary of reinforcement provided			
	Left	Mid	Right
Top	3-25mm	2-25mm	3-25mm
Bottom	2-25mm	3-25mm	2-25mm

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.
At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

TOP FLOOR B54

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	189.7 kNm	0.0 kNm	210.4 kNm
Torsional moment(kNm),Tu	18.6 kNm	7.9 kNm	6.4 kNm
Mt	27.427 kNm	11.684 kNm	9.444 kNm
Me1=Mt + Mu	217.1 kNm	11.7 kNm	219.9 kNm
Ast at top required (mm2)	891 mm2	735 mm2	903 mm2
Asc at bottom required	735 mm2	735 mm2	735 mm2
	Moment (kNm)		
Sagging moment(kNm),Mu	44.5 kNm	168.9 kNm	31.2 kNm
Torsional moment(kNm),Tu	6.0 kNm	2.0 kNm	17.4 kNm
Mt	8.826 kNm	2.962 kNm	25.725 kNm
Me1=Mt + Mu	53.3 kNm	171.8 kNm	56.9 kNm
Ast at bottom required (mm2)	735 mm2	735 mm2	735 mm2
Asc at top required	735 mm2	735 mm2	735 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	891 mm2	735 mm2	903 mm2
Bottom	735 mm2	735 mm2	735 mm2

Summary of reinforcement provided			
	Left	Mid	Right
Top	2-25mm	2-25mm	2-25mm
Bottom	2-25mm	2-25mm	2-25mm

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.
At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

GROUND FLOOR SECONDARY B67

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	105.6 kNm	0.0 kNm	202.2 kNm
Torsional moment(kNm),Tu	1.2 kNm	0.3 kNm	0.1 kNm
Mt	1.798 kNm	0.444 kNm	0.081 kNm
Me1=Mt + Mu	107.4 kNm	0.4 kNm	202.2 kNm
Ast at top required (mm2)	588 mm2	434 mm2	1193 mm2
Asc at bottom required	434 mm2	434 mm2	597 mm2

	Moment (kNm)		
Sagging moment(kNm),Mu	0.0 kNm	94.8 kNm	0.0 kNm
Torsional moment(kNm),Tu	0.0 kNm	1.2 kNm	0.2 kNm
Mt	0.011 kNm	1.724 kNm	0.248 kNm
Me1=Mt + Mu	0.0 kNm	96.5 kNm	0.2 kNm
Ast at bottom required (mm2)	434 mm2	524 mm2	434 mm2
Asc at top required	434 mm2	434 mm2	434 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	588 mm2	434 mm2	1193 mm2
Bottom	434 mm2	524 mm2	597 mm2

Summary of reinforcement provided			
	Left	Mid	Right
Top	2-20mm	2-20mm	2-20mm
Bottom	2-20mm	2-20mm	2-20mm

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.
At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

FIRST FLOOR SECONDARY B61

Summary of Fexural Design of Beam			
	Moment (kNm)		
	Left	Mid	Right
Hogging moment(kNm),Mu	82.8 kNm	0.0 kNm	0.0 kNm
Torsional moment(kNm),Tu	3.3 kNm	4.4 kNm	2.6 kNm
Mt	4.845 kNm	6.350 kNm	3.792 kNm
Me1=Mt + Mu	87.7 kNm	6.3 kNm	3.8 kNm
Ast at top required (mm2)	603 mm2	315 mm2	315 mm2
Asc at bottom required	315 mm2	315 mm2	315 mm2
	Moment (kNm)		
Sagging moment(kNm),Mu	0.0 kNm	58.9 kNm	0.7 kNm
Torsional moment(kNm),Tu	0.5 kNm	1.5 kNm	2.6 kNm
Mt	0.790 kNm	2.198 kNm	3.792 kNm
Me1=Mt + Mu	0.8 kNm	61.1 kNm	4.5 kNm
Ast at bottom required (mm2)	315 mm2	406 mm2	315 mm2
Asc at top required	315 mm2	315 mm2	315 mm2

Summary of required reinforcement			
	Left	Mid	Right
Top	603 mm2	315 mm2	315 mm2
Bottom	315 mm2	406 mm2	315 mm2

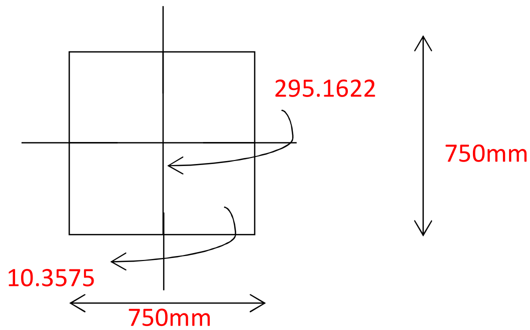
Summary of reinforcement provided			
	Left	Mid	Right
Top	2-20mm	2-20mm	2-20mm
Bottom	2-20mm	2-20mm	2-20mm

At ends, provide 2 legged vertical stirrups of 10mm dia @100mm c-c.
At mid point, Provide 2-legged , 10mm Φ stirrups @ 150 mm c/c.

6.2.3 Design of Column

FLEXURAL ANALYSIS OF COLUMN

For column 11(basement)



Unsupported Length(L) =	3.3	m
Depth of column: D=	750	mm
Width of column: B=	750	mm
Bar Dia(φ)=	20	mm
d'=	55	mm
Concrete Grade=	M30	
Steel Grade=	Fe500	
f_{ck}=	30	N/mm^2
f_y=	500	N/mm^2
P_u=	9401.974	KN

From ETABS

M_{uy}=	10.3575	KN-m
M_{ux}=	295.1622	KN-m

Check for Axial Stress:

IS 13920:1993 cl.7.1.1

$0.08*f_{ck}$=	2.4	N/mm^2
Factored Axial Load(P_u) =	9401.974	KN

Factored Axial Stress

$$\frac{P_u}{B*D} = 16.715 \ N/mm^2 > 2.5 \ N/mm^2$$

Hence, design as Column Member

$$\frac{l_{effx}}{l} = 0.6780$$

$$\frac{l_{effy}}{l} = 0.6780$$

So,

l_{effx}= 2.2374 m

l_{effy}= 2.2374 m

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l} =$ 2.98 **<12 Short Column**

$\lambda_x = \frac{l_{effy}}{l} =$ 2.98 **<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

ex 31.6 mm **>20 OK**

ey 31.6 mm **>20 OK**

Also,

0.05*D = 37.5 mm **>e_{minx}**

0.05*B = 37.5 mm **>e_{miny}**

M_{minx}= P_u*e_{minx}

= 297.1024 KN-m

M_{miny}= P_u*e_{miny}

= 297.1024 KN-m

So,

M_{ux}= 297.1024 KN-m (maximum of M_{ux} and M_{minx})

M_{uy}= 297.1024 KN-m (maximum of M_{uy} and M_{miny})

Since, (e_{minx} and e_{miny}) < (0.05*D and 0.05*B) respectively the column is desi as Biaxial short column

Now,

Assume p%= 1.40%

$\frac{d'}{D} =$ 0.073333 $\frac{d'}{B} =$ 0.0733333

$$\frac{p}{f_{ck}} = 0.046667$$

$$\frac{P_u}{f_{ck}BD} = 0.56$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = 0.025$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = 0.025$$

Thus,

$M_{ux,1} =$ 316.4063 KN-m

$M_{uy,1} =$ 316.4063 KN-m

IS 456 : 2000 cl.39.6

$\mathbf{P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}}$

We get,

$P_{uz} =$ 10440.56 KN

$$\frac{P_u}{P_{uz}} = 0.901$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.901	2.168

Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = 0.861 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 7875 mm^2

From Etabs

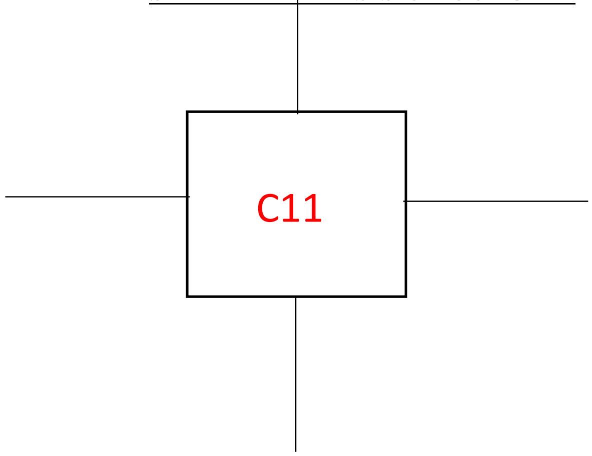
Area of Steel (A_{sc})= 7280 mm^2

		30	28
Provide 6-28mm and 6-30 mm dia	No of bars	6	6

% A_{sc} provided = 1.41 % Range = 0.8% - 6%

A_{sc} provided = 7934.166 mm^2

SHEAR ANALYSIS OF COLUMN



Unsupported Length(L) =	3.3	m
Depth of column: D=	750	mm
Width of column: B=	750	mm
Bar Dia(φ)=	20	mm
d'=	55	mm
Concrete Grade=	M30	
Steel Grade=	Fe500	
f_{ck}=	30	N/mm^2
f_y=	500	N/mm^2

End moment of the beams along X andY-axis adjoining the column is tabulated b

		M_R^{LS}	M_R^{LH}	M_R^{RS}	M_R^{RH}
X-AXIS	Moment from ETABS KNm	222.235	425.901	254.090	483.600
Y-AXIS	Moment from ETABS KNm	265.530	504.093	211.8027	406.8285

Shear force due to formation of plastic hinges

IS 13920 :1993 clause 7.3.4

$$V_P=\left(\frac{M_R^{LS}+M_R^{LH}}{H}\ OR\ \frac{M_R^{RS}+M_R^{RH}}{H}\right)$$

which ever is more

$$V_p = 1.4 * \frac{\text{Sum of MOR of opposite signs of beams framing into column from opposite faces}}{\text{storey height}}$$

V_{px} = 253.3764 KN

V_{py} = 256.988 KN

Factored Shear Force (V_u)

From ETABS Analysis

V_{ux} = 4.439 KN

V_{uy} = 24.1572 KN

Design Shear Force V_e= Max of $|V_p$ or $V_u|$

V_{ex} = 253.376 KN

V_{ey} = 256.988 KN

Shear Force Carried by Concrete

IS 456: 2000, clause 40.2

$$V_c = \tau_{cd} * A_{cv}$$

where,

τ_{cd} = Design shear strength of concrete

= $k*\delta*\tau_c$

for,

A_{st} provided = 7934.166 mm^2

$$p = \frac{A_{st} * 100}{B * d}$$

p= 1.410518 %

τ_c= 0.78

IS 456: 2000, table 19

k= 1

IS 456: 2000, Cl 40.2.2

$$\delta = 1 + \frac{3 * Pu}{A_g * Fck}$$

δ= 2.67 >1.5

so,

δ_x= 1.5

δ_y= 1.5

we get,

τ_{cdx} = 1.170942 N/mm^2

τ_{cdy} = 1.170942 N/mm^2

A_{cv} = Effective Area Under Shear

= b * dc

= 521250 mm^2

so,

V_{cx} = 610353.6 N

V_{cy} = 610353.6 N

Maximum shear stress:

τ_{cmax} = 3.5 N/mm^2 (For M30)

Nominal shear stress:

IS 456: 2000 cl 40.1

$$\tau_{vx} = \frac{V_{ex}}{A_{cv}}$$

= 0.486 N/mm^2

$$\tau_{vy} = \frac{V_{ey}}{A_{cv}}$$

= 0.493 N/mm^2

Check Shear Reinforcement

Comparision:

	Along X	Along Y	Summary	
τ_{cd} =	1.170942	1.17094	Along X	Use case 2
$\tau_{cd}/2$ =	0.585471	0.58547	Along Y	Use case 2
τ_v =	0.486	0.493		
τ_{cmax} =	3.5	3.5		

case: ***IS 456: 2000 Clause 40.2.3***

1) If $\tau_{cd} < \tau_V < \tau_{Cmax}$ design of Shear Reinforcement is necessary
2) If $\tau_{cd} > \tau_V$ also $\tau_{cd}/2 > \tau_V$ theoretically shear reinforcement is not require
3) If $\tau_{cd} > \tau_V$ also $\tau_{cd}/2 < \tau_V$ minimum shear reinforcement is provided
4) If $\tau_V > \tau_{Cmax}$ failure condition is declared; redesign the section

Minimum Shear reinforcement

IS 456: cl 40.3, 26.5.1.6

$$\frac{A_{svmin}}{s_v} = \frac{0.4*b}{0.87*fy}$$

$= 0.690$ mm^2/mm

Max shear force carried by shear reinforcement along any-axis

(y-axis)

$V_{usy} = V_{ey} - \tau_{cdy} A_{cv}$

$= 353365.5$ N

Shear area required per milimeter spacing along Y-axis

$A_{sv}/S_v = V_{usy}/0.87*fy*d$ 1.168827 mm^2/mm > A_{svmin}/S_v

so,

A_{sv}/S_v 1.169 mm^2/mm

Diameter,

$\varphi_t \geq$ 6 mm

$\phi_t \geq \frac{1}{4}*\phi_l =$ 5 mm

Adopt φ_t = 8 mm

so, s_{vmin} = 172.0895 mm (4 legged)

Pitch should not be more than

1)	750	mm	least lateral dimension
2)	320	mm	$16*\varphi_l$
3)	300	mm	300mm

Adopt pitch as 125mm $< s_{vmin}$ OK

Spacing of longitudinal bars

In X-direction,

Space between end bars = 640 mm $> 48*\varphi_t(384)$

Space between bars = 213.3333 mm >75mm

In Y-direction,

Space between end bars = 640 mm $>48*\varphi_t(384)$

Space between bars = 213.3333 mm >75mm

Since, longitudinal bar are spaced more than 75mm on either side so transverse reinforcement should go around corner bars as well as alternate bars with open type

Special confining reinforcement

IS 13920:1993 cl 7.4,7.4.1

Special confining reinforcement has to be provided over a length lo from each joint face towards mid span, and on either side of any section, where flexural yielding may occur under the effect of earthquake.

L_o should be more than

1)	750	mm	larger lateral dimension
2)	550.00	mm	(clear span of member)/6
3)	450	mm	450mm

so,

Provide L_o as 800mm

Spacing of special confinement

IS 13920:1993 cl 7.4.6

spacing should be

1)	187.5	mm	1/4th of minimum lateral dimension
2)	100	mm	should not be less than 75mm nor more than 100mm

so,

Provide 100 mm spacing

Development Length:

IS 456-2000 cl.26.2.1

$$Ld = \frac{\Phi \sigma_s}{4 \ x \ \tau_{bd}} = 906.25 \text{ mm}$$ for compression

φ=	25	mm	(nominal diameter of bar)
$\sigma_s=0.87*f_y$=	435	N/mm^2	(stress in bars)
τ_{bd}=	1.5*1.6*1.25	N/mm^2	(design bond stress for compression)

Thus,

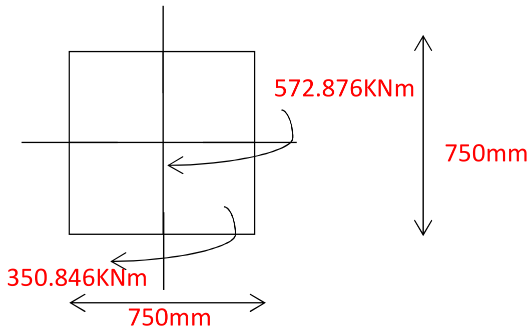
For special confining region i.e. 800 mm adopting 8mm diameter bars at 100 mm spacing

For other regions provide 8mm diameter bars at 125 mm spacing.

Design of Column

FLEXURAL ANALYSIS OF COLUMN

For column 21(basement) BLOCK IV



Unsupported Length(L) =	3.3	m
Depth of column: D=	750	mm
Width of column: B=	750	mm
Bar Dia(φ)=	25	mm
d'=	55	mm
Concrete Grade=	M30	
Steel Grade=	Fe500	
f_{ck}=	30	N/mm^2
f_y=	500	N/mm^2
P_u=	991.142	KN

From ETABS

M_{uy}=	350.8461	KN-m
M_{ux}=	572.8757	KN-m

Check for Axial Stress:

IS 13920:1993 cl.7.1.1

$0.1*f_{ck}$=	3	N/mm^2
Factored Axial Load(P_u) =	991.142	KN

Factored Axial Stress

$\frac{P_u}{B*D}$ = 1.762 N/mm^2 < 2.5 N/mm^2

Hence, design as Column Member

$\frac{l_{effx}}{l} =$ 0.9698

$\frac{l_{effy}}{l} =$ 0.9428

So,

$l_{effx} =$ 3.2003 m

$l_{effy} =$ 3.1112 m

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l} =$ 4.27 **<12 Short Column**

$\lambda_x = \frac{l_{effy}}{l} =$ 4.15 **<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

ex	31.6	mm	**>20 OK**
ey	31.6	mm	**>20 OK**

Also,

0.05*D =	37.5	mm	**>e_{minx}**
0.05*B =	37.5	mm	**>e_{miny}**

$M_{minx} = P_u * e_{minx}$

= 31.3200904 KN-m

$M_{miny} = P_u * e_{miny}$

= 31.3200904 KN-m

So,

$M_{ux} =$ 350.8461 KN-m (maximum of M_{ux} and M_{minx})

$M_{uy} =$ 572.8757 KN-m (maximum of M_{uy} and M_{miny})

Since, (e_{minx} and e_{miny}) < (0.05*D and 0.05*B) respectively the column is designed as Biaxial short column

Now,

Assume p%= 1.00%

$$\frac{d'}{D} = 0.07333333$$

$$\frac{d'}{B} = 0.07333333$$

$$\frac{p}{f_{ck}} = 0.03333333$$

$$\frac{P_u}{f_{ck}BD} = 0.06$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = 0.08$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = 0.08$$

Thus,

$M_{ux,1} =$ 1012.5 KN-m

$M_{uy,1} =$ 1012.5 KN-m

IS 456 : 2000 cl.39.6

$\mathbf{P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}}$

We get,

$P_{uz} =$ 9627.1875 KN

$$\frac{P_u}{P_{uz}} = 0.103$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.103	1.000

Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = \quad 0.912 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5625 mm^2

Area of each Bar (A_b) = 490.88 mm^2

From Etabs

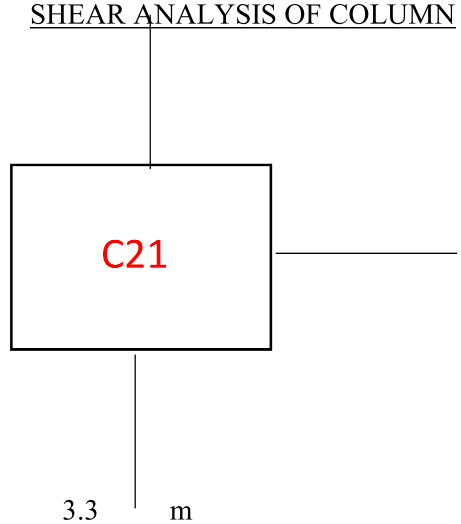
Area of Steel (A_{sc})= 4500 mm^2

Provide 12-25mm dia rebars No of bars 12

% A_{sc} provided = 1.05 % Range = 0.8% - 6%

A_{sc} provided = 5890.5 mm^2

SHEAR ANALYSIS OF COLUMN



Unsupported Length(L) = 3.3 m

Depth of column: D= 750 mm

Width of column: B= 750 mm

Bar Dia(φ)= 25 mm

d'= 55 mm

Concrete Grade= M30

Steel Grade= Fe500

f_{ck}= 30 N/mm^2

f_y= 500 N/mm^2

End moment of the beams along X andY-axis adjoining the column is tabulated below

		M_R^{LS}	M_R^{LH}	M_R^{RS}	M_R^{RH}
X-AXIS	Moment from ETABS KNm	190.173	364.869	0.000	0.000
Y-AXIS	Moment from ETABS KNm	155.862	301.483	148.6726	288.0567

Shear force due to formation of plastic hinges

IS 13920 :1993 clause 7.3.4

$$V_P=\left(\frac{M_R^{LS}+M_R^{LH}}{H}\ OR\ \frac{M_R^{RS}+M_R^{RH}}{H}\right)$$ which ever is more

$$V_p = 1.4 * \frac{\text{Sum of MOR of opposite signs of beams framing into column from opposite faces}}{\text{storey height}}$$

V_{px} = 130.978651 KN

V_{py} = 161.594 KN

Factored Shear Force (V_u)

From ETABS Analysis

V_{ux} = 160.850 KN

V_{uy} = 161.5943 KN

Design Shear Force V_e= Max of $|V_p$ or $V_u|$

V_{ex} = 160.850 KN

V_{ey} = 161.594 KN

Shear Force Carried by Concrete

IS 456: 2000, clause 40.2

$$V_c = \tau_{cd} * A_{cv}$$

where,

τ_{cd} = Design shear strength of concrete

$= k*\delta*\tau_c$

for,

A_{st} provided = 5890.5 mm^2

$$p=\frac{A_{st} * 100}{B * d}$$

p= 1.13007194 %

$\tau_c =$ 0.69

IS 456: 2000, table 19

k= 1

IS 456: 2000, Cl 40.2.2

$$\delta = 1 + \frac{3 * Pu}{A_g * Fck}$$

$\delta =$ 1.18 >1.5

so,

$\delta_x =$ 1.17620304

$\delta_y =$ 1.17620304

we get,

$\tau_{cdx} =$ 0.80689221 N/mm^2

$\tau_{cdy} =$ 0.80689221 N/mm^2

$A_{cv} =$ Effective Area Under Shear

= b * dc

= 521250 mm^2

so,

$V_{cx} =$ 420592.564 N

$V_{cy} =$ 420592.564 N

Maximum shear stress:

$\tau_{cmax} =$ 3.5 N/mm^2 (For M30)

Nominal shear stress:

IS 456: 2000 cl 40.1

$$\tau_{vx} = \frac{V_{ex}}{A_{cv}}$$

= 0.309 N/mm^2

$$\tau_{vy} = \frac{V_{ey}}{A_{cv}}$$

= 0.310 N/mm^2

Check Shear Reinforcement

Comparision:

	Along X	Along Y	Summary	
τ_{cd} =	0.80689221	0.80689	Along X	Use case 2
$\tau_{cd}/2$ =	0.4034461	0.40345	Along Y	Use case 2
τ_v =	0.309	0.310		
τ_{cmax} =	3.5	3.5		

case: ***IS 456: 2000 Clause 40.2.3***

1) If $\tau_{cd} < \tau_V < \tau_{Cmax}$ design of Shear Reinforcement is necessary
2) If $\tau_{cd} > \tau_V$ also $\tau_{cd}/2 > \tau_V$ theoretically shear reinforcement is not required
3) If $\tau_{cd} > \tau_V$ also $\tau_{cd}/2 < \tau_V$ minimum shear reinforcement is provided
4) If $\tau_V > \tau_{Cmax}$ failure condition is declared; redesign the section

Minimum Shear reinforcement

IS 456: cl 40.3, 26.5.1.6

$$\frac{A_{svmin}}{s_v} = \frac{0.4*b}{0.87*fy}$$

$= 0.690$ mm^2/mm

Max shear force carried by shear reinforcement along any-axis

(y-axis)

$Vusy = Vey - \tau cdy\ Acv$

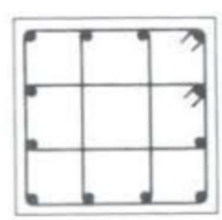
$= 258998.264$ N

Shear area required per milimeter spacing along Y-axis

$A_{sv}/Sv = V_{usy}/0.87*fy*d$ 0.85668821 mm^2/mm > A_{svmin}/S_v

so,

A_{sv}/Sv 0.857 mm^2/mm



Diameter,

$\varphi_t \geq$ 6 mm

$\phi_t \geq \frac{1}{4}*\phi_l =$ 6.25 mm

	Adopt φ_t =	**8**	**mm**	
so,	s_{vmin}=	234.791204	mm	(4 legged)

Pitch should not be more than

1)	750	mm	least lateral dimension
2)	400	mm	16*φ_l
3)	300	mm	300mm

Adopt pitch as 200mm $<s_{vmin}$ OK

Spacing of longitudinal bars

In X-direction,

Space between end bars =	640	mm	> 48*φ_t(384)
Space between bars =	213.333333	mm	>75mm

In Y-direction,

Space between end bars =	640	mm	>48*φ_t(384)
Space between bars =	213.333333	mm	>75mm

Since, longitudinal bar are spaced more than 75mm on either side so transverse reinforcement should go around corner bars as well as alternate bars with open type

Special confining reinforcement

IS 13920:1993 cl 7.4,7.4.1

Special confining reinforcement has to be provided over a length lo from each joint face towards mid span, and on either side of any section, where flexural yielding may occur under the effect of earthquake.

L_o should be more than

1)	750	mm	larger lateral dimension
2)	550.00	mm	(clear span of member)/6
3)	450	mm	450mm

so,

Provide L_o as 500mm

Spacing of special confinement

IS 13920:1993 cl 7.4.6

spacing should be

1)	187.5	mm	1/4th of minimum lateral dimension
2)	100	mm	should not be less than 75mm nor more than 100mm

so,

Provide 100 mm spacing

<u>Development Length:</u>

IS 456-2000 cl.26.2.1

$$Ld = \frac{\Phi \sigma_s}{4 \; x \; \tau_{bd}} = 906.25 \text{ mm} \quad \text{for compression}$$

$\varphi=$	25	mm	(nominal diameter of bar)
$\sigma_s=0.87*f_y=$	435	N/mm^2	(stress in bars)
$\tau_{bd}=$ 3		N/mm^2	(design bond stress for compression)

Thus,

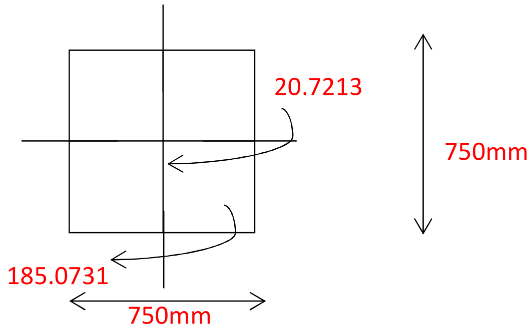
For special confining region i.e. 500 mm adopting 8mm diameter bars at 100 mm spacing

For other regions provide 8mm diameter bars at 200 mm spacing.

Design of Column

FLEXURAL ANALYSIS OF COLUMN

For column 11(first)



Unsupported Length(L) =	3.3	m
Depth of column: D=	750	mm
Width of column: B=	750	mm
Bar Dia(φ)=	20	mm
d'=	55	mm
Concrete Grade=	M30	
Steel Grade=	Fe500	
f_{ck}=	30	N/mm^2
f_y=	500	N/mm^2
P_u=	5875.337	KN

From ETABS

M_{uy}=	20.7213	KN-m
M_{ux}=	185.0731	KN-m

Check for Axial Stress:

IS 13920:1993 cl.7.1.1

$0.08*f_{ck}$=	2.4	N/mm^2
Factored Axial Load(P_u) =	5875.337	KN

Factored Axial Stress

$$\frac{P_u}{B*D} = 10.445 \ N/mm^2 > 2.4 \ N/mm^2$$

Hence, design as Column Member

$\frac{l_{effx}}{l}$ = 0.9500

$\frac{l_{effy}}{l}$ = 0.9500

So,

l_{effx}= 3.1350 m

l_{effy}= 3.1350 m

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l}$ = 4.18 **<12 Short Column**

$\lambda_x = \frac{l_{effy}}{l}$ = 4.18 **<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

ex 31.6 mm **>20 OK**

ey 31.6 mm **>20 OK**

Also,

0.05*D = 37.5 mm **>e_{minx}**

0.05*B = 37.5 mm **>e_{miny}**

M_{minx}= P_u*e_{minx}

= 185.6606618 KN-m

M_{miny}= P_u*e_{miny}

= 185.6606618 KN-m

So,

M_{ux}= 185.6606618 KN-m (maximum of M_{ux} and M_{minx})

M_{uy}= 185.6606618 KN-m (maximum of M_{uy} and M_{miny})

Since, (e_{minx} and e_{miny}) < (0.05*D and 0.05*B) respectively the column is designed as Biaxial short column

Now,

Assume p%= 0.80%

$$\frac{d'}{D} = 0.073333333$$

$$\frac{d'}{B} = 0.073333333$$

$$\frac{p}{f_{ck}} = 0.026666667$$

$$\frac{P_u}{f_{ck}BD} = 0.35$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = 0.065$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = 0.065$$

Thus,

$M_{ux,1}$ = 822.65625 KN-m

$M_{uy,1}$ = 822.65625 KN-m

IS 456 : 2000 cl.39.6

$P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}$

We get,

P_{uz} = 9220.5 KN

$$\frac{P_u}{P_{uz}} = 0.637$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.637	1.729

Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = \quad 0.078 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 4500 mm^2

<u>From Etabs</u>

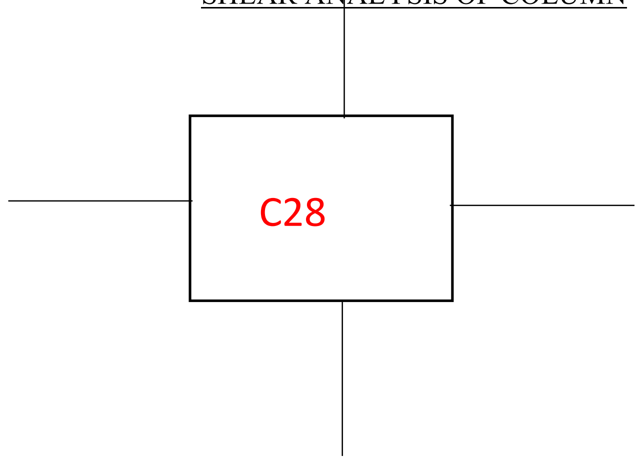
Area of Steel (A_{sc})= 4500 mm^2

			25	22
Provide 6-22mm and 6-25 mm dia reb	No of bars		6	6

% A_{sc} provided = 1.00 % Range = 0.8% - 6%

A_{sc} provided = 5225.0535 mm^2

<u>SHEAR ANALYSIS OF COLUMN</u>



Unsupported Length(L) =	3.3	m
Depth of column: D=	750	mm
Width of column: B=	750	mm
Bar Dia(φ)=	20	mm
d'=	55	mm
Concrete Grade=	M30	
Steel Grade=	Fe500	
f_{ck}=	30	N/mm^2
f_y=	500	N/mm^2

End moment of the beams along X andY-axis adjoining the column is tabulated below

		M_R^{LS}	M_R^{LH}	M_R^{RS}	M_R^{RH}
X-AXIS	Moment from ETABS KNm	196.095	377.879	235.720	450.439
Y-AXIS	Moment from ETABS KNm	241.604	461.094	180.1264	348.2521

Shear force due to formation of plastic hinges

IS 13920 :1993 clause 7.3.4

$$V_P = \left(\frac{M_R^{LS} + M_R^{LH}}{H} \; OR \; \frac{M_R^{RS} + M_R^{RH}}{H} \right)$$ which ever is more

$$V_p = 1.4 * \frac{\text{Sum of MOR of opposite signs of beams framing into column from opposite faces}}{\text{storey height}}$$

V_{px} = 232.0890923 KN

V_{py} = 230.182 KN

Factored Shear Force (V_u)

From ETABS Analysis

V_{ux} = 25.296 KN

V_{uy} = 13.2885 KN

Design Shear Force V_e= Max of $|V_p$ or $V_u|$

V_{ex} = 232.089 KN

V_{ey} = 230.182 KN

Shear Force Carried by Concrete

IS 456: 2000, clause 40.2

$$V_c = \tau_{cd} * A_{cv}$$

where,

τ_{cd} = Design shear strength of concrete

= $k*\delta*\tau_c$

for,

A_{st} provided = 5225.0535 mm^2

$$p = \frac{A_{st} * 100}{B*d}$$

$b*d$

p= 1.002408345 %

τ_c= 0.66

IS 456: 2000, table 19

k= 1

IS 456: 2000, Cl 40.2.2

$$\delta = 1 + \frac{3*Pu}{A_g * Fck}$$

δ= 2.04 >1.5

so,

δ_x= 1.5

δ_y= 1.5

we get,

τ_{cdx} = 0.990722504 N/mm^2

τ_{cdy} = 0.990722504 N/mm^2

A_{cv} = Effective Area Under Shear

= b * dc

= 521250 mm^2

so,

V_{cx}= 516414.105 N

V_{cy}= 516414.105 N

Maximum shear stress:

τ_{cmax} = 3.5 N/mm^2 (For M30)

Nominal shear stress:

IS 456: 2000 cl 40.1

$$\tau_{vx} = \frac{V_{ex}}{A_{cv}}$$

= 0.445 N/mm^2

$$\tau_{vy} = \frac{V_{ey}}{A_{cv}}$$

= 0.442 N/mm^2

<u>Check Shear Reinforcement</u>

Comparision:

	Along X	Along Y	Summary

τ_{cd} =	0.990722504	0.99072	Along X	Use case 2
$\tau_{cd}/2$ =	0.495361252	0.49536	Along Y	Use case 2
τ_v =	0.445	0.442		
τ_{cmax} =	3.5	3.5		

case: ***IS 456: 2000 Clause 40.2.3***

1) If $\tau_{cd} < \tau_V < \tau_{Cmax}$ design of Shear Reinforcement is necessary
2) If $\tau_{cd} > \tau_V$ also $\tau_{cd}/2 > \tau_V$ theoretically shear reinforcement is not required
3) If $\tau_{cd} > \tau_V$ also $\tau_{cd}/2 < \tau_V$ minimum shear reinforcement is provided
4) If $\tau_V > \tau_{Cmax}$ failure condition is declared; redesign the section

Minimum Shear reinforcement

IS 456: cl 40.3, 26.5.1.6

$$\frac{A_{svmin}}{s_v} = \frac{0.4*b}{0.87*fy}$$

= 0.690 mm^2/mm

Max shear force carried by shear reinforcement along any-axis

(x-axis)

Vusx= Vex - Ʈcdx Acv

= 284325.0127 N

Shear area required per milimeter spacing along Y-axis

A_{sv}/Sv= $V_{usy}/0.87*fy*d$ 0.940461466 mm^2/mm > A_{svmin}/S_v

so,

A_{sv}/Sv 0.940 mm^2/mm

Diameter,

$\varphi_t \geq$ 6 mm

$\phi_t \geq \frac{1}{4}*\phi_l$ = 5 mm

Adopt φ_t = 8 mm

so, s_{vmin}= 213.8767663 mm (4 legged)

Pitch should not be more than

1)	750	mm	least lateral dimension
2)	320	mm	$16*\varphi_l$
3)	300	mm	300mm

Adopt pitch as 200mm $<s_{vmin}$ OK

Spacing of longitudinal bars

In X-direction,

Space between end bars =	640	mm	$> 48*\varphi_t(384)$
Space between bars =	213.3333333	mm	>75mm

In Y-direction,

Space between end bars =	640	mm	$>48*\varphi_t(384)$
Space between bars =	213.3333333	mm	>75mm

Since, longitudinal bar are spaced more than 75mm on either side so transverse reinforcement should go around corner bars as well as alternate bars with open type

Special confining reinforcement

IS 13920:1993 cl 7.4,7.4.1

Special confining reinforcement has to be provided over a length lo from each joint face towards mid span, and on either side of any section, where flexural yielding may occur under the effect of earthquake.

L_o should be more than

1)	750	mm	larger lateral dimension
2)	550.00	mm	(clear span of member)/6
3)	450	mm	450mm

so,

Provide L_o as 800mm

Spacing of special confinement

IS 13920:1993 cl 7.4.6

spacing should be

1)	187.5	mm	1/4th of minimum lateral dimension
2)	100	mm	should not be less than 75mm nor more than 100mm

so,

Provide 100 mm spacing

Development Length:

IS 456-2000 cl.26.2.1

$$Ld = \frac{\Phi \sigma_s}{4 \ x \ \tau_{bd}} = \quad 906.25 \text{ mm} \quad \text{for compression}$$

$\varphi=$	25	mm	(nominal diameter of bar)
$\sigma_s=0.87*f_y=$	435	N/mm^2	(stress in bars)
$\tau_{bd}=$	$1.5*1.6*1.25$ N/mm^2		(design bond stress for compression)

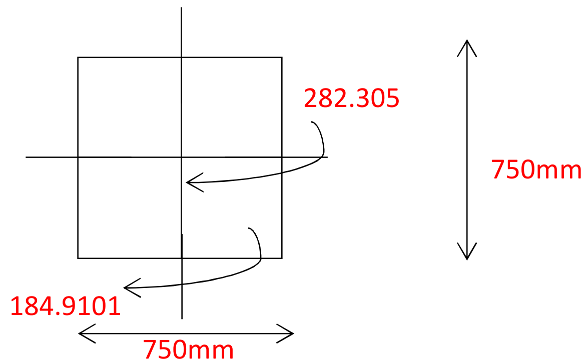
Thus,

For special confining region i.e. 800 mm adopting 8mm diameter bars at 100 mm spacing
For other regions provide 8mm diameter bars at 200 mm spacing.

Design of Column

FLEXURAL ANALYSIS OF COLUMN

For column 29(third)



Unsupported Length(L) =	3.3	m
Depth of column: D=	750	mm
Width of column: B=	750	mm
Bar Dia(φ)=	20	mm
d'=	55	mm
Concrete Grade=	M30	
Steel Grade=	Fe500	
f_{ck}=	30	N/mm^2
f_y=	500	N/mm^2
P_u=	676.227	KN

From ETABS

M_{uy}=	282.305	KN-m
M_{ux}=	184.9101	KN-m

Check for Axial Stress:

IS 13920:1993 cl.7.1.1

$0.08*f_{ck}$=	2.4	N/mm^2
Factored Axial Load(P_u) =	676.227	KN

Factored Axial Stress

$\frac{P_u}{B*D}$ = 1.202 N/mm^2 < 2.4 N/mm^2

Hence, design as Column Member

$\frac{l_{effx}}{l} =$ 0.9740

$\frac{l_{effy}}{l} =$ 0.9740

So,

$l_{effx} =$ 3.2142 m

$l_{effy} =$ 3.2142 m

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l} =$ 4.29 **<12 Short Column**

$\lambda_x = \frac{l_{effy}}{l} =$ 4.29 **<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

ex 31.6 mm **>20 OK**

ey 31.6 mm **>20 OK**

Also,

0.05*D = 37.5 mm **>e_{minx}**

0.05*B = 37.5 mm **>e_{miny}**

$M_{minx} = P_u * e_{minx}$

= 21.3687574 KN-m

$M_{miny} = P_u * e_{miny}$

= 21.3687574 KN-m

So,

$M_{ux} =$ 282.305 KN-m (maximum of M_{ux} and M_{minx})

$M_{uy} =$ 184.9101 KN-m (maximum of M_{uy} and M_{miny})

Since, (e_{minx} and e_{miny}) < (0.05*D and 0.05*B) respectively the column is designed as Biaxial short column

Now,

Assume p%= 0.90%

$$\frac{d'}{D} = 0.073333333$$

$$\frac{d'}{B} = 0.073333333$$

$$\frac{p}{f_{ck}} = 0.03$$

$$\frac{P_u}{f_{ck}BD} = 0.04$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = 0.06$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = 0.06$$

Thus,

$M_{ux,1}$ = 759.375 KN-m

$M_{uy,1}$ = 759.375 KN-m

IS 456 : 2000 cl.39.6

P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}

We get,

P_{uz} = 9423.84375 KN

$$\frac{P_u}{P_{uz}} = 0.072$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.072	1.000

Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = \quad 0.615 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 4691.25 mm^2

<u>From Etabs</u>

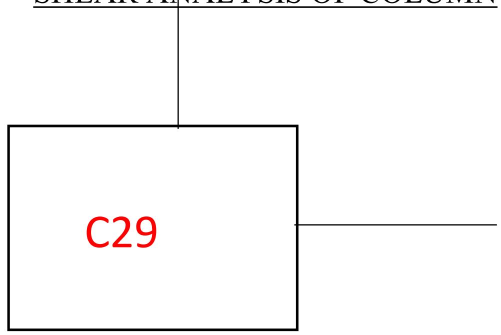
Area of Steel (A_{sc})= 4500 mm^2

		25	22
Provide 8-22mm and 4-25 mm dia reb	No of bars	4	8

% A_{sc} provided = 0.96 % Range = 0.8% - 6%

A_{sc} provided = 5003.613 mm^2

<u>SHEAR ANALYSIS OF COLUMN</u>



Unsupported Length(L) =	3.3	m
Depth of column: D=	750	mm
Width of column: B=	750	mm
Bar Dia(φ)=	20	mm
d'=	55	mm
Concrete Grade=	M30	
Steel Grade=	Fe500	
f_{ck}=	30	N/mm^2
f_y=	500	N/mm^2

End moment of the beams along X andY-axis adjoining the column is tabulated below

		M_R^{LS}	M_R^{LH}	M_R^{RS}	M_R^{RH}
X-AXIS	Moment from ETABS KNm	166.028	311.872	0.000	0.000
Y-AXIS	Moment from ETABS KNm	0.000	0.000	166.0281	166.0281

Shear force due to formation of plastic hinges

IS 13920 :1993 clause 7.3.4

$$V_P=\left(\frac{M_R^{LS}+M_R^{LH}}{H}\ OR\ \frac{M_R^{RS}+M_R^{RH}}{H}\right)$$ which ever is

$$V_p=1.4*\frac{\text{Sum of MOR of opposite signs of beams framing into column from opposite faces}}{\text{storey height}}$$

V_{px} =	111.9540513	KN
V_{py} =	59.600	KN

Factored Shear Force (V_u)

From ETABS Analysis

V_{ux} =	89.363	KN
V_{uy} =	51.1161	KN

Design Shear Force V_e= Max of $|V_p$ or $V_u|$

$V_{ex} =$ 111.954 KN

$V_{ey} =$ 59.600 KN

Shear Force Carried by Concrete

IS 456: 2000, clause 40.2

$V_c = \tau_{cd} * A_{cv}$

where,

$\tau_{cd} =$ Design shear strength of concrete

$= k*\delta*\tau_c$

for,

A_{st} provided = 5003.613 mm^2

$$p = \frac{A_{st} * 100}{B * d}$$

p= 0.959925755 %

$\tau_c =$ 0.65

IS 456: 2000, table 19

k= 1

IS 456: 2000, Cl 40.2.2

$$\delta = 1 + \frac{3 * Pu}{A_g * Fck}$$

$\delta =$ 1.12 >1.5

so,

$\delta_x =$ 1.120218044

$\delta_y =$ 1.120218044

we get,

$\tau_{cdx} =$ 0.730365531 N/mm^2

$\tau_{cdy} =$ 0.730365531 N/mm^2

$A_{cv} =$ Effective Area Under Shear

$= b * dc$

$=$ 521250 mm^2

so,

$V_{cx} =$ 380703.033 N

$V_{cy} =$ 380703.033 N

Maximum shear stress:

$\tau_{cmax} =$ 3.5 N/mm^2 (For M30)

Nominal shear stress:

IS 456: 2000 cl 40.1

$$\tau_{vx} = \frac{V_{ex}}{A_{cv}}$$

$=$ 0.215 N/mm^2

$$\tau_{vy} = \frac{V_{ey}}{A_{cv}}$$

$=$ 0.114 N/mm^2

Check Shear Reinforcement

Comparision:

	Along X	Along Y	Summary	
$\tau_{cd} =$	0.730365531	0.73037	Along X	Use case 2
$\tau_{cd}/2 =$	0.365182765	0.36518	Along Y	Use case 2
$\tau_v =$	0.215	0.114		
$\tau_{cmax} =$	3.5	3.5		

case: ***IS 456: 2000 Clause 40.2.3***

1) If $\tau_{cd} < \tau_V < \tau_{Cmax}$ design of Shear Reinforcement is necessary
2) If $\tau_{cd} > \tau_V$ also $\tau_{cd}/2 > \tau_V$ theoretically shear reinforcement is not required
3) If $\tau_{cd} > \tau_V$ also $\tau_{cd}/2 < \tau_V$ minimum shear reinforcement is provided
4) If $\tau_V > \tau_{Cmax}$ failure condition is declared; redesign the section

Minimum Shear reinforcement

IS 456: cl 40.3, 26.5.1.6

$$\frac{A_{svmin}}{s_v} = \frac{0.4*b}{0.87*fy}$$

$=$ 0.690 mm^2/mm

Max shear force carried by shear reinforcement along any-axis

(x-axis)

Vusx= Vex - Ʈcdx Acv

= 268748.9817 N

Shear area required per milimeter spacing along Y-axis

$A_{sv}/Sv=$ $V_{usy}/0.87*fy*d$ 0.888940649 mm^2 /mm > A_{svmin}/S_v

so,

A_{sv}/Sv 0.889 mm^2 /mm

Diameter,

$\varphi_t \geq$ 6 mm

$\phi_t \geq \frac{1}{4} * \phi_l =$ 5 mm

Adopt φ_t = 8 mm

so, s_{vmin}= 226.2725384 mm (4 legged)

Pitch should not be more than

1)	750	mm	least lateral dimension
2)	320	mm	$16*\varphi_l$
3)	300	mm	300mm

Adopt pitch as 200mm $<s_{vmin}$ OK

Spacing of longitudinal bars

In X-direction,

Space between end bars = 640 mm $> 48*\varphi_t(384)$

Space between bars = 213.3333333 mm >75mm

In Y-direction,

Space between end bars = 640 mm $>48*\varphi_t(384)$

Space between bars = 213.3333333 mm >75mm

Since, longitudinal bar are spaced more than 75mm on either side so transverse reinforcement should go around corner bars as well as alternate bars with open type

Special confining reinforcement

IS 13920:1993 cl 7.4,7.4.1

Special confining reinforcement has to be provided over a length lo from each joint face towards mid span, and on either side of any section, where flexural yielding may occur under the effect of earthquake.

L_o should be more than

1)	750	mm	larger lateral dimension
2)	550.00	mm	(clear span of member)/6
3)	450	mm	450mm

so,

Provide L_o as 800mm

Spacing of special confinement

IS 13920:1993 cl 7.4.6

spacing should be

1)	187.5	mm	1/4th of minimum lateral dimension
2)	100	mm	should not be less than 75mm nor more than 100mm

so,

Provide 100 mm spacing

Development Length:

IS 456-2000 cl.26.2.1

$$Ld = \frac{\Phi\ \sigma_s}{4\ x\ \tau_{bd}} = 906.25 \text{ mm} \quad \text{for compression}$$

φ=	25	mm	(nominal diameter of bar)
$\sigma_s=0.87*f_y$=	435	N/mm^2	(stress in bars)

τ_{bd}= 1.5*1.6*1.25 N/mm^2 (design bond stress for compression)

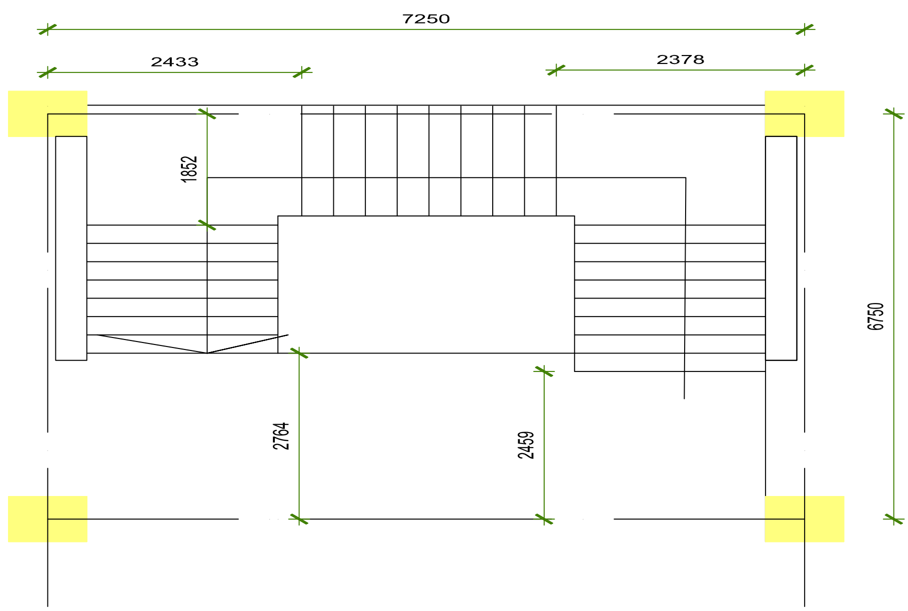
Thus,

For special confining region i.e. 800 mm adopting 8mm diameter bars at 100 mm spacing

For other regions provide 8mm diameter bars at 200 mm spacing.

6.2.4 Staircase Calculations

Design of Staircase – Open Well Staircase



1. Geometrical properties

- Floor height = 3.9 m
- Tread width (T) = 305 mm
- Riser height (R) = 150 mm
- Number of risers = 26
- Numbers of risers in
  - Flight 1 – 9
  - Flight 2 – 9
  - Flight 3 – 8

- Number of treads = 23
- Numbers of tread in
  - Flight 1 – 8
  - Flight 2 – 8
  - Flight 3 – 7

- Length of flights 1(Going Length) = 2.44 m
- Length of flights 2(Going Length) = 2.44m
- Length of flights 3 (Going Length) = 2.135 m
- Width of flights 1,2 and 3= 2m

For Flight 1

- L_{eff} = 2.459*0.5+2.44+1.852*0.5= 4.5955m
  $\text{Cos}\alpha = \frac{2.44}{(2.44^2+1.35^2)^{\frac{1}{2}}} = 0.875$

Some assumed data,

- Depth of waist slab (D) = 200mm
- Clear cover (cc) = 20mm
- Diameter of bar (ϕ) = 12 mm
- Effective depth = 200-20-12/2 = 174 mm

2. Load Calculation

2.1 Load calculation for landing

Self-weight of slab = 0.16m * 25kN/m^3 = 4 kN/m^2

Floor finish = 1kN/m2

Total dead load = 4 +1 = 5 kN/m^2

Imposed load = 4kN/m2

Total load = 5 + 4 = 9 kN/m^2

Factored loads on landing = 1.5*9= 13.5 kN/m^2

2.2 Load calculation for going

Wt. of waist slab on horizontal plane = $25*0.16*\frac{339.8896}{305}$ = 4.4575 kN/m^2

Wt. of steps = 25*0.5*0.15 = 1.875 kN/m^2

Floor finish = 1 kN/m^2

Total dead load = 4.4575+1.875+1 = 7.3325 kN/m^2

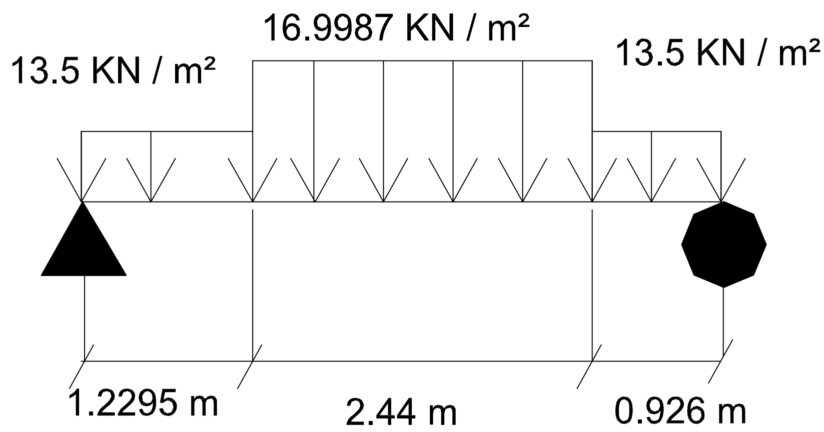
Imposed load = 4 kN/m^2

Total load = 11.3325 kN/m^2

Factored load on going = 1.5 * 11.3325 = 16.99875 KN/m^2

4. Analysis

Consider 1m width of flight



$R_A * 4.5955 = \{13.5*1.2295*(0.926 + 2.44 + \frac{1.2295}{2})\} + \{16.9987*2.44*(\frac{2.44}{2} + 0.926)\} + \{13.5*0.926*(\frac{0.926}{2})\}$

$R_A = 35.006$ kN

$R_B = 35.57$ kN

Location of zero shear

$x = \frac{35.006}{13.5} = 2.593\text{m} > 1.2295\text{m}$

So, it lies beyond the 13.5 kN/m UDL,

$x = \frac{35.006-(13.5*1.2295)}{16.9987} = 1.0829$ m

So, location of zero shear is (1.0829+1.2295)=2.3124 m from left end.

$M_{max} = R_A*(2.3124) - (13.5*1.2295*(1.2295*0.5+1.0829)) - (16.9987*1.0829*1.0829*0.5)$

$= 42.803$ kN-m

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

$= 0.133*30*1000*174^2 = 122.193$ kNm $> M_{max}$ (42.803 kNm)

Hence section can be designed as singly reinforced.

5. Design of reinforcement

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

$42.803*10^6 = 0.87*500*Ast*174*(1- \frac{Ast*500}{1000*174*30})$

Solving, $Ast_{required} = 596.115\ mm^2$

Use 12mm Ø bars then,

Spacing = $\frac{\pi*12^2/4}{817.417}*1000 = 189.724$ mm

Provide 12 dia bar at 150 mm c/c.

$Ast_{provided} = \frac{\pi*12^2/4}{150}*1000 = 753.982\ mm^2$

Distribution steel = 0.12% of bD = 240 mm^2 /m

Use 12mm Ø bars then,

Spacing = $\frac{\pi*12^2/4}{240}*1000 = 471.238$mm

Provide 12dia. bar at 150mm c/c.

6. Check for Shear

Vu = 35.006 kN

$\tau_v = \frac{V_u}{bd} = \frac{35.006*1000}{1000*174} = 0.2$ N/mm2

Pt = $\frac{753.982}{1000*174}*100 = 0.4308$` %

From **Table 19 IS 456:2000** for Pt = 0.4308% and M30 Concrete;

τ_c= 0.464 N/mm2

Again, from **Clause 40.2.1.1 of IS 456** for slab thickness = 160 mm; k = 1.2

Therefore, Permissible shear stress (τ'_c) = k× τ_c = 1.2×0.464 = 0.5568 N/mm2

Also, from **Table 20 of IS 456**; τc, max = 3.5 N/mm2 (for M30) i.e. $\tau v < \tau_c' < \tau c_{max}$ hence shear capacity is sufficient.

7. Check for deflection

$\frac{l}{d} = \frac{4595.5}{174} = 26.41$

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

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

fs = 0.58×fy×($Ast_{required}/Ast_{provided}$) = 0.58×500 × (596.115/753.982) = 229.28

Hence for fs = 229.28 and %$Ast_{provided}$ = 0.4308% From **Fig. 4 IS456:2000;** γ= 1.4; δ=1 (From **Fig. 5 IS456:2000);** λ=1 (From **Fig. 6 IS456:2000)**

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

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

8. Development length

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

$L_d = \frac{0.87 f_y \emptyset}{4\tau bd} = \frac{0.87*500*12}{4*1.5*1.6} = 543.75$ mm

Development length of 600mm is provided.

For Flight 2

- $L_{eff} = \frac{2.38}{2} + 2.44 + \frac{2.43}{2} = 4.845$ m (Transverse support for flight 2)
  $\cos\alpha = \frac{2.44}{(2.44^2+1.35^2)^{\frac{1}{2}}} = 0.875$

Some assumed data

- Depth of waist slab (D) = 200 mm
- Clear cover (cc) = 20mm
- Diameter of bar (ϕ) = 12 mm
- Effective depth = 200-20-12/2 = 174 mm

2. Load Calculation

2.1 Load calculation for landing

Self-weight of slab = 0.16m * 25 kN/m^3

= 4 kN/m^2

Floor finish = 1kN/m2

Total dead load = 4 + 1 = 5 kN/m^2

Imposed load = 4 kN/m2

Total load = 5 + 4 = 9 kN/m^2

Factored loads on landing = 1.5*9 = 13.5 kN/m^2

2.2 Load calculation for going

Wt. of waist slab on horizontal plane = 25*0.16*$\frac{339.889}{305}$ = 4.457 kN/m^2

Wt. of steps = 25*0.5*0.15 = 1.875 kN/m^2

Floor finish = 1 kN/m^2

Total dead load = 4.457+1.875+1 = 7.332 kN/m^2

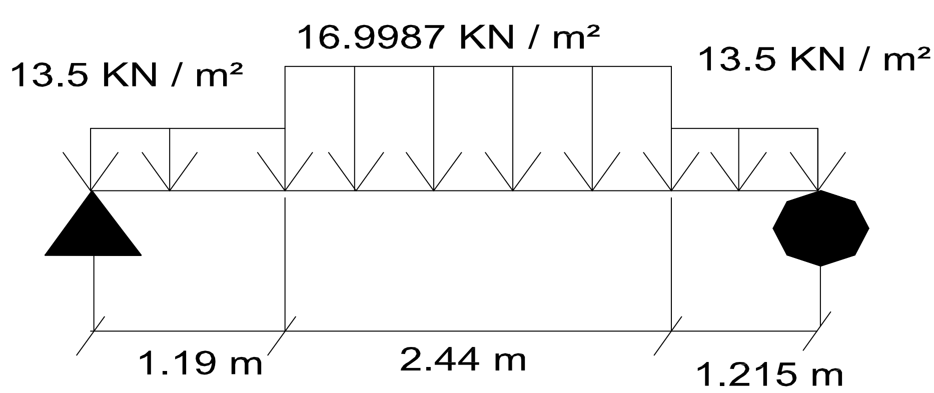
Imposed load = 4 kN/m^2

Total load = 11.332 kN/m^2

Factored load on going = 1.5 * 11.332= 16.998 KN/m^2

4. Analysis

Consider 1m width of flight



R_A *4.845= {13.5*1.19*(1.215 + 2.44 + $\frac{1.19}{2}$)}+ {16.998*2.44*(1.215+$\frac{2.44}{2}$)} + {13.5*1.215*$\frac{1.215}{2}$}

R_A =36.994 kN

R_B = 36.95 kN

Location of zero shear

x = $\frac{36.994}{13.5}$ = 2.74 m > 1.19 m

So, it lies beyond the 13.5 kN/m UDL,

x = $\frac{36.994-(13.5*1.19)}{16.998}$ = 1.231 m

So, location of zero shear is (1.19 + 1.231) m i.e., 2.421 m from left end.

M_{max} = R_A*(1.19+1.231) – (13.5*1.19*(1.231+$\frac{1.19}{2}$)) – 16.998*$\frac{1.231^2}{2}$

= 47.348 kN-m

Mu,lim = 0.133*fck*b*d^2

= 0.133*30*1000*174^2 = 120.8 kNm > M_{max} (47.348 kNm)

Hence section can be designed as singly reinforced.

5. Design of reinforcement

Mu = 0.87f_yAstd (1- $\frac{Astfy}{bdfck}$)

47.348*10^6 = 0.87*500*Ast*174*(1- $\frac{Ast*500}{1000*174*30}$)

Solving, Ast, required = 668.336 mm^2

Use 12mm Ø bars then,

Spacing = $\frac{\pi*12^2/4}{668.336}*1000$ = 169.22mm

Provide 12 dia. bar at 150 mm c/c.

Ast, provided = $\frac{\pi * 12^2/4}{150}$ $*1000 = 753.982$ mm^2

Distribution steel = 0.12% of bD = 240 mm^2 /m

Use 12mm Ø bars then,

Spacing = $\frac{\pi * 12^2/4}{240} * 1000 = 471.238$ mm

Provide 12 dia. bar at 150 c/c.

6. Check for Shear

Vu = 36.994 kN

$\tau_v = \frac{V_u}{bd} = \frac{36.994*1000}{1000*174} = 0.2126$ N/mm2

Pt = $\frac{753.982}{1000*174} * 100 = 0.433\%$

From **Table 19 IS 456:2000** for Pt = 0.433% and M30 Concrete;

$\tau_c = 0.4651$ N/mm2

Again, from **Clause 40.2.1.1 of IS 456** for slab thickness = 200 mm; k = 1.20

Therefore, Permissible shear stress (τ'_c) = k× τ_c = 1.20×0.4651 = 0.55812 N/mm2

Also, from **Table 20 of IS 456**; τc, max = 3.5 N/mm2 (for M30) i.e., $\tau v < \tau c' < \tau c_{max}$ hence shear capacity is sufficient.

7. Check for deflection

$\frac{l}{d} = \frac{4845}{174} = 27.844$

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

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

fs = 0.58×fy×(Astrequired/AstProvided) = 0.58×500 × (668.336 /753.982) = 257.058

Hence for fs = 257.058 and %AstProvided = 0.433% From **Fig. 4 IS456:2000;** γ= 1.25; δ=1 (From **Fig. 5 IS456:2000);** λ=1 (From **Fig. 6 IS456:2000)**

$\alpha\beta\gamma\delta\lambda$= 26×1×1.25×1×1 = 32.5

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

8. Development length

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

$L_d = \frac{0.87 f_y \emptyset}{4\tau} = \frac{0.87*500*12}{4*1.6*1.5} = 543.75$ mm

Development length of 600mm is provided.

For Flight 3

- L_{eff} = 1.852*0.5+2.135+2.764*0.5= 4.443m
  $\text{Cos}\alpha = \frac{2.745}{(2.745^2+1.5^2)^{\frac{1}{2}}} = 0.87752$

Some assumed data

- Depth of waist slab (D) = 200mm
- Clear cover (cc) = 20mm
- Diameter of bar (ϕ) = 12 mm
- Effective depth = 160-20-12/2 = 174 mm

2. Load Calculation

2.1 Load calculation for landing

Self-weight of slab = 0.16m * 25kN/m^3 = 4 kN/m^2

Floor finish = 1kN/m2

Total dead load = 4 +1 = 5 kN/m^2

Imposed load = 4kN/m2

Total load = 5 + 4 = 9 kN/m^2

Factored loads on landing = 1.5*9= 13.5 kN/m^2

2.2 Load calculation for going

Wt. of waist slab on horizontal plane = 25*0.16*$\frac{339.8896}{305}$ = 4.4575 kN/m^2

Wt. of steps = 25*0.5*0.15 = 1.875 kN/m^2

Floor finish = 1 kN/m^2

Total dead load = 4.4575+1.875+1 = 7.3325 kN/m^2

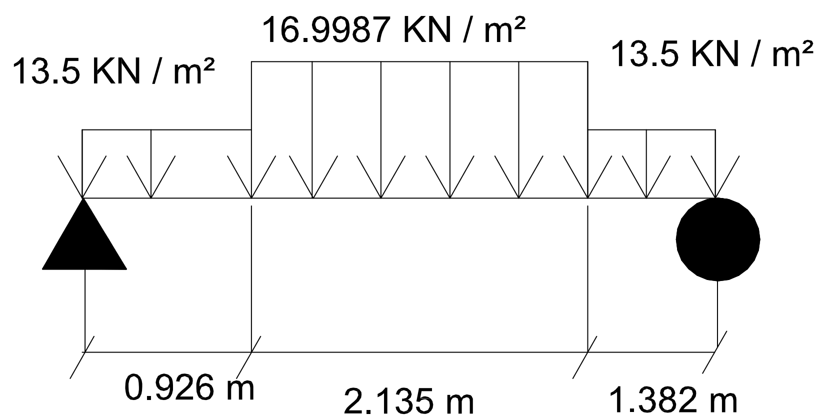
Imposed load = 4 kN/m^2

Total load = 11.3325 kN/m^2

Factored load on going = 1.5 * 11.3325 = 16.99875 KN/m^2

4. Analysis

Consider 1m width of flight



R_A * 4.443 = {13.5*0.926*($\frac{0.926}{2}$ + 2.135 + 1.382))}+{16.9987*2.135*(1.382 + $\frac{2.135}{2}$)} + {13.5*1.382*($\frac{1.382}{2}$)}

R_A = 34.108 kN

R_B = 33.342 kN

Location of zero shear

x = $\frac{34.108}{13.5}$ = 2.526m >0.926m

So, it lies beyond the 13.5 kN/m UDL,

$x = \frac{34.108-(13.5*0.926)}{16.9987} = 1.27$ m

So, location of zero shear is 3.796 m from left end.

$M_{max} = R_A*(0.926+1.27) - (13.5*0.926*(1.27+\frac{0.926}{2})) - 16.9987*\frac{1.27^2}{2}$

$= 39.528$ kN-m

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

$= 0.133*30*1000*174^2 = 120.8$ kNm $> M_{max}$ (39.528 kNm)

Hence section can be designed as singly reinforced.

5. Design of reinforcement

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

$39.528*10^6 = 0.87*500*Ast*174*(1-\frac{Ast*500}{1000*174*30})$

Solving, $Ast_{required} = 551.353\ mm^2$

Use 12mm Ø bars then,

Spacing $= \frac{\pi*12^2/4}{551.353}*1000 = 205.126$ mm

Provide 12 dia. bar at 150 mm c/c.

Ast, provided $= \frac{\pi*12^2/4}{150}*1000 = 753.982\ mm^2$

Distribution steel = 0.12% of bD = 240 mm^2 /m

Use 12mm Ø bars then,

Spacing $= \frac{\pi*12^2/4}{240}*1000 = 471.238$mm

Provide 12 dia. bar at 150 c/c.

6. Check for Shear

Vu = 34.108 kN

$\tau_v = \frac{V_u}{bd} = \frac{34.108*1000}{1000*174} = 0.196$ N/mm2

Pt = $\frac{753.982}{1000*174}*100 = 0.433$` %

From **Table 19 IS 456:2000** for Pt = 0.433% and M30 Concrete;

$\tau_c = 0.4651$ N/mm2

Again, from **Clause 40.2.1.1 of IS 456** for slab thickness = 200 mm; k = 1.2

Therefore, Permissible shear stress (τ'_c) = k× τ_c = 1.2×0.4651 = 0.5581 N/mm2

Also, from **Table 20 of IS 456**; τc, max = 3.5 N/mm2 (for M30) i.e., τv < τc' < τc_{max}, hence shear capacity is sufficient.

7. Check for deflection

$\frac{l}{d} = \frac{4443}{174} = 25.534$

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

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

fs = 0.58 × fy × ($Ast_{required}$ / $Ast_{provided}$) = 0.58×500 × (551.353/753.982) = 212.063
Hence for fs = 212.063 and %$Ast_{provided}$ = 0.433% From **Fig. 4 IS456:2000**
γ= 1.15; δ=1 (From **Fig. 5 IS456:2000);** λ=1.5 (From **Fig. 6 IS456:2000)**

$\alpha\beta\gamma\delta\lambda$= 26×1×1.5×1×1 = 39

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

8. Development length

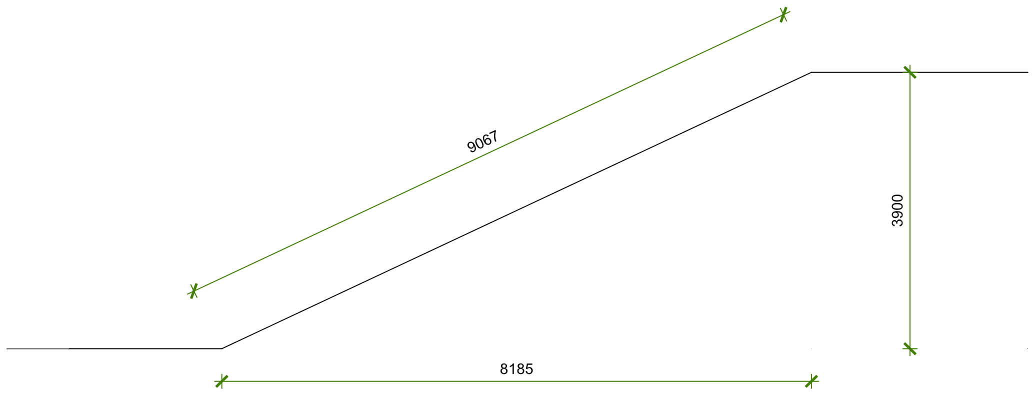
The development length (L_d) is given by **(IS 456: 2000, Cl. 26.2)**;

$L_d = \frac{0.87\ y\ \emptyset}{4\tau bd} = \frac{0.87*500*12}{4*1.5*1.6} = 543.75$ mm

Development length of 600mm is provided.

6.2.5 Ramp Slab calculations

Concrete Grade = M30
Steel Grade = Fe500



1. Known data

Height = 3.9m

Width, W = 4.75m

Length of the ramp = 8.185 m

2. Load Calculation

Slab thickness = 130 mm = 0.13m

Total thickness = 0.13m

Total area= 9.0666*0.13= 1.1786 m^2

Dead Load per meter length of plan= 25*1.786 = 29.466 KN/m **(γ = 25 KN/m^3)**

Dead load per m^2 of plan = $\frac{29.466}{9.066} = 3.25$

Live load = 4 KN/m^2

FF load = 1 KN/m^2

Total load = 8.25 KN/m^2

Factored load = 1.5*8.25 = 12.375 KN/ m^2

Taking 4.75 m width of slab = 4.75*12.375= 58.78125 KN/m.

(W_s= 58.78125 KN/m)

3. **Analysis**

Span of ramp = 4.75 m

Maximum bending moment = (58.78125*4.75^2)/8 = 165.7815 kNm

Maximum shear force = (4.75*58.78125) / 2 = 139.6054 kN

Checking for depth,

Take d = 105 mm

Effective cover = 25 mm

Overall depth D = 130 mm

Depth required = $\sqrt{\frac{Mu}{0.133bf_{ck}}} = \sqrt{\frac{165.7815\ X\ 10^6}{0.133\ X\ 4750\ X\ 30}} = 93.526 < 105\ mm$ **(OK)**

(i.e., $d_{required} < d_{provided}$)

So, take d = 105 mm; D =130 mm.

Design for main reinforcement,

For maximum moment,

$165.7815 * 10^6 = 0.87 * 500 * Ast * 105 (1- (Ast * 500) / (105 * 4750 * 30))$

$Ast_{required} = 4226.535\ mm^2 > Ast_{min}$ **(OK)**

($Ast_{min} = 0.12\% * bd = 598.5\ mm^2$)

Required spacing of 20 mm bars,

c/c spacing = $(4750/4226.535) * (3.14 * 10^2) = 352.889$ mm

Provide 20mm bars @ 200mm c/c

$Ast_{provided} = 7457.5\ mm^2$ ($> 4226.535\ mm^2$)

4. **Check for Shear (IS 456-2000, CL.40.1)**

Vu = 139.6054 KN

Nominal Shear, Tv = Vu/(bd)

= (139.6054 * 1000) (4750 * 105)

= 0.2799 N/mm^2

Percentage of tensile steel = (100 * Ast) / (bd) = 1.495%

Shear strength of M30 concrete for 1.495% steel,

(IS 456-2000, CL.40.2.1 T19)

Tc = 0.759 N/mm^2 > Tv

K = 1.3 (For l = 30mm) **(IS 456-2000, CL40.2.1.1)**

K*Tc = 1.3 * 0.759 = 0.9867 N/mm^2 > Tv

Hence, safe in shear.

5. **Check for Development Length (IS 456-2000, CL.26.2.1)**

Ld = (0.87 * fy * 20)/(4T_{bd})

= (20 * 0.87 * 500)/(4*2.4)

= 906.25 mm

Where, T_{bd} = 1.6 * 1.5 = 2.4 N/mm^2

The value of T_{bd} is increased by 60% for deformed bar in tension.

Mu = 0.133* fck *bd^2 = 208.95 KNm

1.3(Mu/Vu)= 1945.73 mm > Ld (OK)

6. **Temperature Reinforcement**

Provide 8 mm bars as temperature reinforcement

In waist slab, provide 12% steel

Ast,min = 0.0012*105*4750 = 598.5 mm^2

Required spcaing for 8mm bars,

c/c spacing = (4750/598.5) * (3.14*4^2) = 398.73 mm

Provide 8 mm bars @ 300 mm c/c

Ast, provided = 795.87 mm^2

7. **Deflection check**

IS 456-2000 CL.23.2.1

Lx/d < αβγδλ

Lx = 4.75m

α = 26

β = 1 (Span less than 10 m)

γ = 1 (No compression reinforcement)

δ = 1 (Not a flanged section)

For λ,

fs = $0.58 \text{ fv } \frac{Area\ of\ steel\ required}{Area\ of\ steel\ provided}$

Area of steel required = 4226.535 mm^2

Area of steel provided = 7457.5 mm^2

So, fs = 164.35 N/m^2; %st = 1.495%; **IS 456-2000 CL.23.2.1 Fig 4**

So, $\alpha\beta\gamma\delta\lambda$ = 31.2

Lx/d = 45.328 < 31.2 **(NOT OK)**

Adopt d = 175 mm

Conclusion

Adopt, d = 175 mm

D = 200 mm

6.2.6 Design of Lift Shear Wall

The design of lift wall has been designed as the reinforced wall monolithic to the other structural members which is subjected to the direct compression. They are designed as per the empirical procedure given in the **IS-13920, clause 9.1.2** The minimum thickness of the wall should be 150 mm. The design of a wall shall account of the actual eccentricity of the vertical force subjected to the minimum value of 0.05t. The vertical load transmitted to a wall by a discontinuous concrete floor or roof shall be assumed to act at one-third the depth of the bearing area measured from the span face of the wall. Where there is an in-situ continuous concrete floor over the wall, the load shall be assumed to act the center of the wall. The resultant eccentricity of the total vertical load on a braced wall at any level between horizontal lateral supports shall be calculated on the assumption that the resultant eccentricity of all the vertical loads above the upper support is zero.

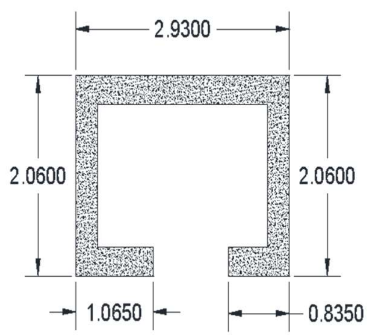


Figure 6.1 Lift Shear Wall

Design:

Known Data from drawing:

Total Length of Wall = 8.95 m

Floor Height (H) = 3.9 m

Assume, wall thickness (t) = 300 mm

Ratio of effective height to thickness:

Effective Height of the wall (h_e) = 0.75H = 0.75 x 3.9 = 2.925 m

Slenderness Ratio (h_e/t) = 2.925/0.30 = 9.75 < 30 (**OK**)

Minimum eccentricity:

e_{min} =0.05t = 0.05 x 300 = 15 mm

$e_a = H^2_{we}/(2500\ t) = 2925^2/(2500 \times 300)$ = 11.41 mm

1. Basement Floor:

a) Lift Wall:

Length = 8.95m

Characteristic load= 8.95 x 1.95 x 0.30 x 25 = 130.89 kN

Design load= 1.5 x 130.89 = 196.34 kN

2. Typical Floor (GF to 4th):

a) Lift Wall:

Length = 8.95m

Characteristic load = 8.95 x 3.9 x 0.30 x 25 = 261.788 kN

Design load = 1.5 x 126 = 392.681 kN

3. Roof:

a) Lift Wall:

Length = 8.95m

Characteristic load= 8.95 x 1.95 x 0.30 x 25 = 130.894 kN

Design load= 1.5 x 63 = 196.341 kN

Total Seismic Weight of Lift =196.34 + 6 x 392.681 + 196.341 = 2748.767 kN

h = 27.3 m

$A_h = \frac{ZIS_a}{2Rg}$ **(IS 1893 -2016, Cl.6.4.13)**

$$T_x = \frac{0.09h}{\sqrt{d_x}} = \frac{0.09 \times 27.3}{\sqrt{2.93}} = 1.435\ s$$

$\frac{S_a}{g} = 0.95$ (From graph)

Z = 0.36

I = 1.5

R = 5

Ah_X = 0.0513

Base shear (V_{bX}) = A_h.W = 0.0513 x 2748.767 kN = 141.012 kN

$A_h = \frac{ZIS_a}{2Rg}$ **(IS 1893 -2016, Cl.6.4.13)**

$$T_y = \frac{0.09h}{\sqrt{d_y}} = \frac{0.09 \times 27.3}{\sqrt{2.06}} = 1.712\ s$$

$\frac{S_a}{g} = 0.794$ (From graph)

Z = 0.36

I = 1.5

R = 5

$Ah_y = 0.0429$

Base shear (V_{by}) = A_{hy}.W = 0.0429 x 2748.767 kN = 117.856 kN

$$Q_i = \frac{(W_i {h_i}^2)}{\sum_{i=1}^{n}(W_i {h_i}^2)}$$

Additional eccentricity:

$e_a = H^2_{we}/(2500*t) = 2925^2/(2500*300) = 11.41$ mm < e_{min} (15mm)

Table 6.1 Lateral Distribution of Shear by Static Method								
Story	**Wi**	**hi**	**Wi*hi^2**	$\frac{W_i h_i^2}{\sum W_i h_i^2}$	**Lateral Force**		**Moment due to lateral force**	
					Qx (kN)	**Qy (kN)**	**Mux**	**Muy**
Roof	196.34	27.3	146330.239	0.212	29.912	25.000	0	0
Top	392.681	23.4	215016.408	0.312	43.952	36.734	171.4121	143.264
Fourth	392.681	19.5	149316.950	0.216	30.522	25.510	409.4844	342.2418
Third	392.681	15.6	95562.848	0.139	19.534	16.326	638.0339	533.2604
Second	392.681	11.7	53754.102	0.078	10.988	9.184	809.4459	676.5244
First	392.681	7.8	23890.712	0.035	4.884	4.082	904.6749	756.1155
Ground	392.681	3.9	5972.678	0.009	1.221	1.020	933.2436	779.9929
Basement	196.34	0	0.000	0.000	0.000	0.000	933.2436	779.9929
		Sum	689843.938		141.012	117.856		

Hence, adopt eccentricity = 15mm

Since, wall thickness is greater than 200mm, reinforcement bars should be provided in two curtains within the cross section.

Using 16 mm ϕ bar, Effective cover, d' = 40mm

When lateral load is acting along X–direction:

From ETABS:

M_{ux} = 5421.282/2 = 2710.641 kNm

V_{ux} = 1300.271/2 = 650.136 kN

P_{ux} = 11576.06/2 = 5788.03 kN

d'/Dx = 40/2930 = 0.0137 (< 0.05)

Hence, for this case we have to use sp16-chart 35

From P_u – M_u interaction curve

$\frac{Mux}{f_{ck}bd^2}$ = 2710.641 × 10^6/ (30 x 300 x 2930^2) = 0.035

$\frac{Pux}{f_{ck}bd}$ = 5788.03 x 10^3/ (30 x 300 x 2930) = 0.219

P/f_{ck} = 0.00

So, we provide minimum reinforcement.

According to IS 13920 (2016) CL 9.1.4, minimum reinforcement ratio is 0.0025bD

Minimum reinforcement:

$A_{st,min}$ = 0.12% of bD = 1054.8 mm^2

A_{st} = 0.0025 x 300 x 2930 = 2197.5 mm^2 **(ok)**

Using 16 mm - ϕ bar,

No. of bars = 2197.5/ (π x 16^2/4)

= 10.93

Adopt no. of bars = 11

Therefore, Spacing of Bars, S_v = (2930 – 80) / (11-1) = 285 mm

Check for Spacing:

Spacing of vertical steel reinforcement should not be greater than

i. 3 times thickness of web of wall
ii. 450mm
iii. 1/5 times horizontal length of wall = 586 mm

So, Provide 16 mm ϕ bars @ 250 mm c/c.

When lateral load is acting along Y – direction:

M_{uy} = 5575.75/2 = 2787.875 kNm

$V_{uy} =$ 924.266/2 = 462.133 kN

$P_{uy} =$ 11576.06/2 = 5788.03 kN

d'/b = 40/2060 = 0.019 (< 0.05)

Hence, for this case we have to use sp16-chart 35

From $P_u - M_u$ interaction curve,

$\frac{Muy}{f_{ck}bd^2}$ = 2787.875 × 10^6/ (30 x 300 x 2060^2) = 0.073

$\frac{Puy}{f_{ck}bd}$ = 5788.03 x 10^3/ (30 x 300 x 2060) = 0.312

P/f_{ck} = 0.00

So, we provide minimum reinforcement. i.e., 0.0025bD

Minimum reinforcement:

$A_{st,min}$ = 0.12% of bD = 741.6 mm^2

A_{st} = 0.0025 x 300 x 2060 = 1545 mm^2 **(ok)**

Using 16 mm - ϕ bar,

No. of bars = 1545 / (π x 16^2/4)

= 7.688

So, we adopt no. of bars = 9

Therefore, Spacing of Bars, S_v = (2060 – 80) / (9-1) = 247.5 mm

Check for Spacing:

Spacing of vertical steel reinforcement should not be greater than

i. 3 times thickness of web of wall
ii. 450mm
iii. 1/5 times horizontal length of wall = 412 mm

So, Provide 16 mm ϕ bars @ 230 mm c/c.

Calculation of Horizontal Steel Reinforcement:

Area of horizontal steel reinforcement = 0.25% of bH

= 0.0025 x 300 x 3900

= 2925 mm^2

Providing 12 mm ϕ bar,

No. of Bars = 2925 /2 x (π x 12^2/4) = 12.93, adopt 14

No. of bars per m = 14/3.9 = 3.58

But from ETABS, shear reinforcement required is 750mm2/m

i.e., Reinforcement in each curtain = 750/2 = 375mm2/m

Area of bar, As = 113.04 mm2

No. of bars per m = 375/113.04 = 3.317

So, adopt no. of bars = 14

Spacing of Bars, S_v = 3900 / (14-1) = 300 mm

According to IS13920 (2016) CL 9.2.6, the vertical reinforcement shall not be less than the horizontal reinforcement.

So, provide 12 mm ϕ bars @ 300 mm c/c both faces of the wall.

<u>Check for Shear:</u>

When lateral load is acting along X – direction:

Nominal Shear Stress,

τ_v = V_u/td **(IS456:2000, Cl.32.4.2)**

= V_u/(t x 0.8 xL_w)

= (650.136 x 10^3) / (300 x 0.8 x 2930)

= 0.924 N/mm^2

Allowable Shear Stress, $a_{llowable}$ = $0.17 f_{ck}$ = 0.17 x 30 = 5.1 $N/mm^2 > \tau_v$ **(IS456:2016,Cl.32.4.2.1)**

H_w/L_w = 3900 / 2930 = 1.33 (Intermediate Wall)

Now, to find Design Shear Strength of Concrete (τ_{cw}),

$$K_1 = 0.2;\ K_2 = 0.045;\ K_3 = 0.15$$

Then, for H_w/L_w >1, τ_{cw} will be: **(IS456:2016,Cl.32.4.3)**

$$\tau_{cw} = K_2\sqrt{f_{ck}}\,\frac{(\frac{H_w}{L_w}+1)}{(\frac{H_w}{L_w}-1)} = 0.045 \times \sqrt{30} \times \frac{1.33+1}{1.33-1} = 1.74 \text{ N/mm}^2$$

But shall not be less than,

$\tau_{cw} = K_3\sqrt{f_{ck}} = 0.15 \times \sqrt{30} = 0.822$ N/mm^2

$\therefore \tau_{cw} = 1.74$ N/mm^2 > τ_v

When lateral load is acting along Y – direction:

Nominal Shear Stress,

$\tau_v = V_u/td$ **(IS456:2016,Cl.32.4.2)**

$= V_u/ (t \times 0.8 \times L_w)$

$= (462.133 \times 10^{3)} / (300 \times 0.8 \times 2060)$

$= 0.934$ N/mm^2

Allowable Shear Stress, $\tau_{allowable} = 0.17 f_{ck} = 0.17 \times 30 = 5.1\ N/mm^2 > \tau_v$ **(IS456:2016, Cl.32.4.2.1)**

$H_w/L_w = 3900 / 2060 = 1.89$ (Intermediate Wall)

Now, to find τ_{cw},

$$K_1 = 0.2;\ K_2 =;\ K_3 = 0.15$$

Then, for $H_w/L_w > 1$, τ_{cw} will be: **(IS456:2016,Cl.32.4.3)**

$\tau_{cw} = K_2\sqrt{f_{ck}}\frac{(\frac{H_w}{L_w}+1)}{(\frac{H_w}{L_w}-1)} = 0.045 \times \sqrt{30} \times \frac{1.89+1}{1.89-1} = 0.80$ N/mm^2

But shall not be less than,

$\tau_{cw} = K_3\sqrt{f_{ck}} = 0.15 \times \sqrt{30} = 0.82$ N/mm^2

$\therefore \tau_{cw} = 0.82$ N/mm^2 < τ_v

Hence, the design is safe in X direction as the design shear strength of concrete is more than the shear stress. However, shear reinforcement shall be provided in Y direction.

Shear reinforcement in Y direction:

According to IS13920 (2016) CL 9.2.5,

$$V_{us} = \frac{0.87 f_y A_h d_w}{S_v}$$

$$S_v = \frac{0.87x500x1921.68x0.8x2060}{(456.133x10^3) - (0.82x300x0.8x2060)} = 3398.09\text{mm}$$

Here, A_h = Area of minimum horizontal reinforcement provided above.

Since, minimum horizontal rebars of 12 mm ϕ bars @ 230 mm c/c both faces of the wall is already provided. No additional reinforcement is required.

6.2.7 Design of Basement Wall

The purpose of a basement wall is to prevent moisture from entering the building and retain the earth. The mat foundation provides support to the basement wall, which only needs to be designed as a vertical stem for stability.

Basement walls can be either exterior walls of underground structures or retaining walls that need to withstand the pressure from surrounding earth and other loads. These walls are usually vertical slabs supported by floor framing at the basement and upper floor levels to resist lateral earth pressure. The forces in the floor structures are then balanced by either shear walls or the opposite side's lateral earth pressure.

The design of the basement wall assumes that the soil backfilling will be done after the construction of the ground floor. This approach is cost-effective as it allows the wall to be designed as cantilevered and supported by both the mat foundation and the soil pressure.s

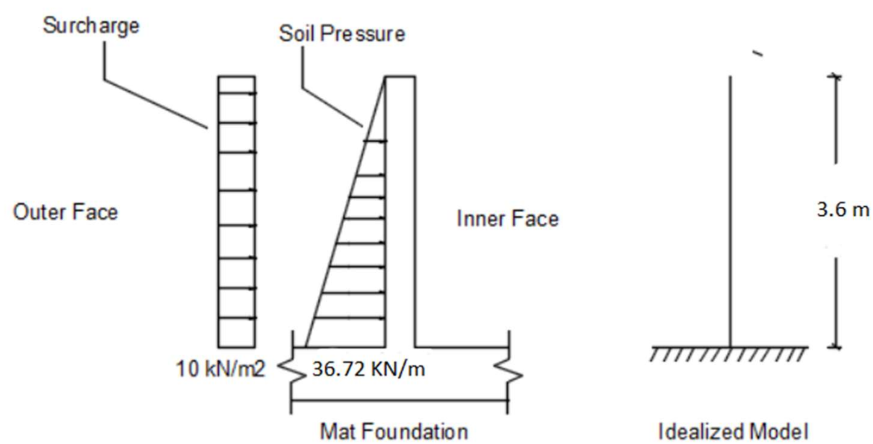


Figure 6.2 Basement Wall models

Known data:

Concrete Grade = M30

Steel Grade = Fe500

1. Design Constants

Floor to floor height, Basement height (h) = 3.9m

Unit weight of soil, $\gamma = 17\ kN/m^3$

Angle of internal friction of the soil, $\theta = 30^{\circ}$

Surcharge produced due to vehicular movement, $W_s = 20\ kN/m^3$

Safe bearing capacity of the soil, q_s = 140 kN/m^3

Height of the soil from base to ground level (h) = 3.6 m

2. Moment calculation

Coefficient of Earth Pressure, $K_a = \frac{1-Sin}{1+Sin\,\theta} = \frac{1-Sin\quad°}{1+Sin\quad°} = 0.333$

Lateral load due to soil pressure, $P_a = \frac{Ka*\gamma*h^2}{2} = \frac{0.333*17*3.6^2}{2} = 36.68 kN/m$

Lateral load due to surcharge load, P_s = Ka * W_s * h = 0.333*20*3.6 = 23.976 kN/m

Characteristics bending moment at the base of wall is calculated below. Since the weight of the wall gives insignificant moment, so this can be neglected in the design.

$M_c = \frac{Pa*h}{3} + \frac{Ps*h}{2} = \frac{36.68*3.6}{3} + \frac{23.976*3.6}{2} = 87.1728 kNm$/m

Design moment, M = 1.5 Mc = 130.7592 kNm/m

3. Approximate design of section

Let effective depth of wall = d

BM = $0.133 fckbd^2$ (For Fe500) **(IS 456:2000, Annex G)**

or, 130.7592*10^6 = 0.133*30*1000*d^2

Let clear cover is 40mm and bar size Φ is 16 mm.

Overall depth of wall, D = 181.023+40+8 = 229.023mm

Take D = 250mm

So, d = 150-40-8 = 202 mm

D = 250 mm > 200mm **(IS 456:2000, Cl. 32.5.1)**

So, double curtains of reinforcement need to be provided.

4. Calculation of main steel reinforcement

$M_u = 0.87 * fy * Ast * d\left(1 - \frac{Astfy}{bdfck}\right)$ **(IS 456:2000, Annex G)**

or, 130.7592*10^6$= 0.87 * 500 * Ast * 202\left(1 - \frac{Ast*500}{1000*202*30}\right)$

or, A_{st} = 1737.0556 mm^2

Minimum A_{st} = 0.0012*b*D = 0.0012*1000*250 = 300 mm^2 <A_{st} (OK) **(IS 456:2000, Cl. 26.5.2.1)**

Maximum diameter of bar = D/8 = 250/8 =31.25 mm > 16mm (OK)

Providing 20mm Φ bar,

Spacing of bars (S) = 1000*A_b/A_{st} = $\frac{\Pi*20*1000}{4*1737.0556}$ = 180.85 mm/m

Providing 20mm Φ bar @ 150mm c/c

Maximum spacing = 3 * wall thickness or 450 = 450 mm **(IS 456:2000, Cl. 32.5.b)**

So, Provided A_{st} = $\frac{\Pi*20*1000}{4*150}$= 2094.395 mm^2

$p_t = \frac{2094.395}{1000*202} * 100$ = 1.03% > 0.4% (OK)

5. Check for shear

The critical section for shear strength is taken at a distance 'd' from the face of the support. Thus, the cri tical section is at d = 0.202m from the top of mat foundation i.e. 3.9-0.202 = 3.698 m below the top edge of wall.

Shear force at critical section is

$Vu = 1.5*(Ka*W_s*z + K_a*\gamma*(z^2/2)$

$= 1.5*(0.333*20*3.698+0.333*17*(3.698^2/2))$

= 95.005 kN

Nominal shear stress, $\tau_v = \frac{Vu}{bd} = \frac{95.005*1000}{1000*202} = 0.470$N/$mm^2$ **(IS 456:2000, Cl. 31.6.2.1,40.3)**

Permissible shear stress, τ_c = 0.665N/mm^2 **(IS 456:2000, Table 19)**

Maximum shear stress, $\tau_{c,max}$ = 3.5 N/mm^2 **(IS 456:2000, Table 20)**

Here, $\tau_{c,max} > \tau_c > \tau_v$. Hence, Safe.

6. Check for deflection

L_{eff} = Clear span + d or c/c of support

= (3.9-0.55) + 0.202 = 3.552 m

Allowable deflection = L_{eff} /250 = 3552.8/250 =14.208mm

$$Actual\ deflection = \frac{Ps\qquad ^3}{8E} + \frac{Paleff^3}{30EI}$$ **(IS 456:2000, Cl. 23.2.a)**

$$= \frac{23.97*3552.0^3*12}{8*5000\sqrt{30}*1000*250^3} + \frac{33.68*3552.0^3*12}{30*5000\sqrt{30}*1000*25\ ^3}$$

= 5.17mm

= 5.17mm < 14.208mm (Safe)

7. Calculation of Horizontal Reinforcement Steel Bar

Min. reinforcement = 0.0012*1000*150=180 mm^2 **(IS 456:2000, Cl.32.5)**

As the temperature change occurs at the front face of the basement wall,

2/3rd of horizontal reinforcement is provided at the front face and

1/3rd of horizontal reinforcement is provided at the inner face face.

Temp. reinforcement at front face = (2*180)/3= 120 mm^2

Provide 10 mm bars spacing= $\frac{\pi*10^2*1000}{4*120}$= 654.5 mm

Max. Spacing = 3*d = 306 mm or 450 mm (whichever is small) **(IS 456:2000, Cl.32.5d)**

Hence, **Provide 10 mm bar @ 300 mm c/c at front face of the wall.**

Temp. reinforcement at inner face = 180/3= 60 mm^2

Provide 10 mm bars spacing= $\frac{\pi*10^2*1000}{4*60}$= 1309mm

Hence, **Provide 10 mm bar @ 300 mm c/c at inner face of the wall.**

8. Curtailment of Reinforcement

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

$L_d = \frac{\sigma s\phi}{1.6*4*\tau bd} = \frac{0.87*500*20}{1.6*4*1.5} = 906.25mm$ **(IS 456:2000, Cl.26.2.1)**

The curtailment of the bars can be done in two layers, at 1/3 and 2/3 heights of the stem above the base.

Let us curtail the bars only at 1/3 i.e., 1300 mm from the base (i.e., h'=2600mm distance from top)

Lateral load due to soil pressure,

$P_a = K_a * \gamma * (h'^2/2) = 0.333*17*(2600-300)^2/2) = 14.97$ kN/m

Lateral load due to surcharge load,

$P_s = K_a * W_s * h' = 0.333*20*(2600-300) = 15.318$ kN/m

Characteristics bending moment at the base of the wall is

$M_c = P_a * (h'/3) + P_s * (h/2) = 14.97 * (2.3/3) + 15.318 * (2.3/2) = 29.097$ kN-m/m

Design moment, $M = 1.5\ M_c = 1.5*29.097 = 43.6455$ kN-m/m

Since the moment is less than the half of the moment at the base of the stem, spacing of the vertical reinforcement can be doubled from 1300 mm from the base of the wall.

Provide 200mm bars @ 200 mm above 1300 mm from the base.

6.2.8 Design of RC Structural Wall

Design:

Known Data:

Total Length of Wall = 6.75 m

Floor Height (H) = 3.9 m

Assume, wall thickness (t) = 300 mm

Check for Slenderness Ratio:

Effective Height of the wall (h_e) = 0.75H = 0.75 x 3.9 = 2.925 m

Slenderness Ratio (h_e/t) = 2.925/0.30 = 9.75 < 30 **(OK)**

Minimum eccentricity:

$E_{min} = 0.05t = 0.05 \times 300 = 15$ mm

$e_a = H^2_{we}/(2500\ t) = 2925^2/(2500 \times 300) = 11.41$ mm $< E_{min}$

1. Basement Floor:

a) RC wall:

Length = 6.75m

Characteristic load= 6.75 x 1.95 x 0.30 x 25 = 98.718 kN

Design load= 1.5 x 98.718 = 148.078 kN

2. Typical Floor (GF to 5th):

a) RC wall:

Length = 6.75m

Characteristic load = 6.75 x 3.9 x 0.30 x 25 = 197.438 kN

Design load = 1.5 x 197.438 = 296.156 kN

3. Roof:

a) RC Wall:

Length = 6.75m

Characteristic load= 6.75 x 1.95 x 0.30 x 25 = 98.718 kN

Design load= 1.5 x 98.718 = 148.078 kN

Total Seismic Weight of Lift = 148.078 + 6 x 296.156 + 148.078 = 2073.092 kN

h = 27.3 m

$A_h = \frac{ZIS_a}{2Rg}$ **(IS 1893 -2016, Cl.6.4.13)**

$$T_x = \frac{0.09h}{\sqrt{d_x}} = \frac{0.09 \times 27.3}{\sqrt{6.75}} = 0.945\ s$$

$\frac{S_a}{g} = 1.438$ (From graph)

Z = 0.36

I = 1.5

R = 5

$Ah_X = 0.078$

Base shear (V_{bX}) = A_h.W = 0.078 x 2073.092 kN = 160.98 kN

$$Q_i = \frac{(W_i {h_i}^2)}{\sum_{i=1}^{n}(W_i {h_i}^2)}$$

Lateral Distribution of Shear By Static Method						
					Lateral Force	Moment
Story	Wi	hi	Wihi2	(Wihi2)/∑(Wihi2)	Qx	Muy
Roof	148.078	27.3	110361.1	0.2121	34.171	0.000
Top	296.156	23.4	162163.2	0.3117	50.210	133.267
Fourth	296.156	19.5	112613.3	0.2165	34.868	462.355
Third	296.156	15.6	72072.52	0.1385	22.316	927.430
Second	296.156	11.7	40540.79	0.0779	12.553	1479.536
First	296.156	7.8	18018.13	0.0346	5.579	2080.597
Ground	296.156	3.9	4504.533	0.0087	1.395	2703.417
Basement	148.078	0	0	0.0000	0.000	3331.675
		Sum	520273.5		161.092	

Using 12 mm ϕ bar, Effective cover, d' = 40mm

When lateral load is acting along X – direction:

M_{ux} = 3331.675/2 = 1665.837 kNm

V_{ux} = 161.092/2 = 80.546 kN

P_{ux} = 2073.092/2 = 1036.546 kN

d'/Dx = 40/6750 = 0.0059 (< 0.05)

Hence, for this case we have to use sp16-chart 35

From P_u – M_u interaction curve

$\frac{Mux}{f_{ck}bd^2}$ = 1665.837 × 10^6/ (25 x 300 x 6750^2) = 0.0048

$\frac{Pux}{f_{ck}bd}$ = 1036.546 x 10^3/ (25 x 300 x 6750) = 0.0205

P/f_{ck} = 0.000

Hence, provide minimum reinforcement i.e. 0.0025bD

Minimum reinforcement:

$(A_{st})_{min}$ = 0.0025 x 300 x 6750 = 5062.5 mm^2

Using 12 mm - ϕ bar,

No. of bars = 5062.5/ (π x 12^2/4)

= 44.78 adopt 45

Therefore, Spacing of Bars, S_v = (6750 – 80) / (45-1) = 151.59 mm

Check for Spacing:

Spacing of vertical steel reinforcement should be ≤ 3t or 450mm whichever is less.

To take account of the reversal effect,

Provide 12 mm ϕ bars @ 150 mm c/c.

Calculation of Horizontal Steel Reinforcement:

Minimum area of horizontal steel reinforcement = 0.25% of bH

= 0.0025 x 300 x 3900

= 2925 mm^2

Providing 12 mm ϕ bar,

No. of Bars = 2925 / (π x 12^2/4) = 25.87 , adopt 26

Spacing of Bars, S_v = 3900 / (26-1) = 156 mm

To take account of the reversal effect, **Provide 12 mm ϕ bars @ 150 mm c/c** both faces of the wall.

Check for Shear :

When lateral load is acting along X – direction :

Nominal Shear Stress,

τ_v = V_u/td **(IS456:2002,Cl.32.4.2)**

= V_u/(t x 0.8 xL_w)

$= (80.546 \times 10^3) / (300 \times 0.8 \times 2500)$
$= 0.134\ N/mm^2$

Allowable Shear Stress, $\tau_{allowable} = 0.17 f_{ck} = 0.17 \times 30 = 5.1\ N/mm^2 > \tau_v$
(IS456:2002,Cl.32.4.2.1)

$H_w/L_w = 27300 / 6750 = 4.04$ (Slender Wall)

Now, to find τ_{cw},

$K_1 = 0.2$

$K_2 = 0.045$

$K_3 = 0.15$

Then, τ_{cw} will be least of : **(IS456:2002,Cl.32.4.3)**

$$\tau_{cw} = K_2 \sqrt{f_{ck}} \frac{(\frac{H_w}{L_w}+1)}{(\frac{H_w}{L_w}-1)} = 0.045 \times \sqrt{30} \times \frac{4.04+1}{4.04-1} = 0.493\ \text{N/mm}^2$$

But shall not be less than,

$$\tau_{cw} = K_3 \sqrt{f_{ck}} = 0.15 \times \sqrt{30} = 0.822\ \text{N/mm}^2$$

$\therefore \tau_{cw} = 0.822$ N/mm^2 $> \tau_v$

Hence, the design is safe in both directions as the design shear strength of concrete is more than the shear stress.

6.2.9 Design of Seismic Gap

According to IS**1893: 2016, Cl.7.11.3**, two adjacent building or two adjacent units of same building with separation joint in between shall be separated by a distance equal to R times the sum of the calculated storey displacements as per **Cl.7.11.1** of each of them, to avoid damaging contact when two units deflect towards each other. When floor levels of two similar adjacent units or buildings are at same elevation levels, factor R in the IS requirement may be replaced by R/2.

A. **Seperation between Block I and Block II:**

Maximum displacement in Block I (Δ_1) = 26.318 mm

Maximum displacement in Block II (Δ_2) = 48.705

Since, floor levels of both blocks are same:

Seperation = R (Δ_1+Δ_2)/2 = (5x (26.318+48.705))/2 = 187.55mm

Adopt 190 mm.

B. Seperation Between Block I and Block III:

Maximum displacement in Block I (Δ_1) = 26.318 mm

Maximum displacement in Block III (Δ_2) = 9.16

Since, floor levels of both blocks are same:

Seperation = R (Δ_1+Δ_2)/2 = (5x (26.318+9.16))/2 = 88.695 mm

Adopt 90 mm.

C. Seperation Between Block IV and Block III:

Maximum displacement in Block IV (Δ_1) = 53.611 mm

Maximum displacement in Block III (Δ_2) = 9.16

Since, floor levels of both blocks are same:

Seperation = R (Δ_1+Δ_2)/2 = (5x (53.611+9.16))/2 = 156.93mm

Adopt 160 mm.

D. Seperation Between Block IV and Block II:

Maximum displacement in Block IV (Δ_1) = 53.611 mm

Maximum displacement in Block II (Δ_2) = 48.705

Since, floor levels of both blocks are same:

Seperation = R (Δ_1+Δ_2)/2 = (5x (53.611+48.705))/2 = 255.79 mm

Adopt 260 mm.

6.2.10 Design of Mat Foundation

Foundation are structural elements that transfer load from the building or individual column to the earth below. If these loads are to be transmitted properly, foundations should be designed to prevent excessive settlement and rotation, to minimize differential settlement and to provide adequate safety against sliding and overturning. Foundation can be classified as:

(1) Isolated footing under individual columns. These may be rectangular, square of circular in plan.

(2) Strip foundation or Wall foundation

(3) Combined footing supporting two or more column load.

(4) Mat or Raft foundation

(5) Pile Foundation

(6) Well Foundation.

Raft foundation is a sub structure supporting an arrangement of columns or walls in a row or rows and transmitting the load to the soil by means of a continuous slab with or without depressions or openings. Such types of foundations are found useful where soil has low bearing capacity.

Detail Designing of Raft Foundation:

Design Constants:

Unit weight of soil (γ) = 18 kN/m^3

Service Load (P) = 278718.4 kN

Service load includes the total axial forces of column, weight from lift wall, and load from basement walls.

Grade of Concrete = M30

Grade of steel = Fe500

Bearing Capacity (q) = 200 KN/m^2

Angle of Repose of soil (Ø) = 30 ^{o}

Depth of raft foundation shall generally be not less than 1m **(IS 2950 Part 1, Cl. 4.3)**

$D_f = q_u/\gamma_s \times \{(1-sin\emptyset)^2/(1+sin\emptyset)^2\}$

$= 200/18\times \{(1-sin30)^2/(1+sin30)^2\}$

= 1.234 m > 1 m.

However, the lower face of the designed footing will be placed at a level of 1m below which soil is free from seasonal volumetric change.

If above reaction from superstructure is taken as non-eccentric surcharge to the soil, area of foundation required for safe transmission of load is given by:

Maximum service load of column = 13775.9 kN

Area required for one footing = 13775.9/200 = 68.87m2

Total no. of columns = 38

Hence, total area for foundation = 38 x 68.87 = 2617.421m2 >> 50% of plinth area (1275m2)

Consider the raft having same shape of the superstructures with 2000 mm projection along the building periphery for critical shear section consideration.

∴ Area of Foundation provided = 1497.625 m^2

Location of geometric C.G.: X = 23.625m, Y = 14.238m

Calculation of Eccentricity:

SN	Column ID	Force (kN)	X	Y	Mx	My
1	C1	9461.82	0	6.75	-91.40	-97.99
2	C2	5508.82	0	13.5	-176.67	-184.74
3	C3	1600.38	0	20.25	-235.47	-202.08
4	C4	3152.87	0	27	-179.61	-42.42
5	C5	7486.79	6.75	6.75	-123.34	-57.30
6	C6	9543.33	6.75	13.5	-73.29	-9.63
7	C7	9615.02	6.75	20.25	-80.05	-17.25
8	C8	3332.60	6.75	27	-4.11	-42.33
9	C9	5928.92	13.5	6.75	-61.49	0.26

10	C10	8578.08	13.5	13.5	-150.69	-34.33
11	C11	9489.51	13.5	20.25	-64.95	-23.69
12	C12	1397.84	13.5	27	-24.19	-66.36
13	C13	11558.26	20.25	6.75	-220.58	-143.23
14	C14	11569.93	20.25	13.5	-145.16	-84.05
15	C15	8895.71	20.25	20.25	-63.88	-20.76
16	C16	1416.18	20.25	27	-39.47	-58.51
17	C17	11182.57	27	6.75	-109.51	-108.46
18	C18	7573.94	27	13.5	-160.41	-34.14
19	C19	9041.63	27	20.25	-81.24	-37.52
20	C20	1391.20	27	27	-37.54	-72.79
21	C21	10788.19	33.75	6.75	-85.90	-27.33
22	C22	3593.98	33.75	13.5	-65.29	-84.62
23	C23	3537.42	33.75	20.25	-130.29	-58.91
24	C24	3065.74	33.75	27	-38.88	-46.75
25	C25	2598.55	40.5	6.75	-223.37	-117.94
26	C26	1307.92	40.5	13.5	-40.27	-95.05
27	C27	900.39	40.5	20.25	-163.67	-70.09
28	C28	3257.31	40.5	27	-43.09	-32.53
29	C29	6152.88	47.25	6.75	-145.96	-110.40
30	C30	6029.32	47.25	13.5	-126.98	-70.03
31	C31	841.58	47.25	20.25	-229.68	-66.71
32	C32	715.18	47.25	27	-223.63	-59.50
33	C33	3416.75	6.75	0	-86.36	-74.12
34	C34	6147.12	13.5	0	-109.75	-42.84
35	C35	6779.80	20.25	0	-110.57	-65.05
36	C36	6706.32	27	0	-123.93	-61.10
37	C37	6843.26	33.75	0	-138.42	-71.70
38	C38	3720.98	40.5	0	-226.76	-55.05
39	PW2	3506.24	37.125	27	-8.53	-3246.23
40	PW4	5104.54	37.125	0	-58.21	-7435.57
41	PW3	3420.16	3.375	27	-26.93	-3249.85
42	PW5	7790.76	23.625	0	-35.51	-7670.69
43	PW6	5578.00	0	10.125	14.75	-10708.20
44	PW7	2596.52	47.25	10.125	-32.10	-10821.07
45	PW8	10764.84	20.25	10.125	-34.44	-13106.59
46	PW10	12053.29	27	9.125	1.53	-4534.49
48	PW9	13775.90	23.625	13.5	-44.67	-6424.85
	Sum	**278718.36**			**-4659.98**	**-69744.56**

Here, negative moment in finite element analysis indicates clockwise moment.

ex = 0.0167 m

ey = 0.25 m

M_{xx} = 4659.98 kNm

M_{yy} = 69744.6 kNm

I_{xx} = 112674.5 m^4

I_{yy} = 02287.4 m^4

Soil pressure calculation for different points, i.e., the point through which the load of superstructure is transmitted to the foundation.

$$\sigma = \frac{\sum P}{A} \pm \frac{Myy}{Iyy} \times X \pm \frac{Mxx}{Ixx} \times Y$$

Soil Pressure at different Points are as follows:

Column ID	P/A (kN/m2)	Ixx (m4)	Iyy (m4)	X-X̄ (m)	Y-Ȳ (m)	(Mxx/Ixx)xy (kN/m2)	(Myy/Iyy)x X (kN/m2)	Soil Pressure (kN/m2)
C1	186.107	112674.500	302287.400	-23.625	-7.488	-0.310	-5.451	180.346
C2	186.107	112674.500	302287.400	-23.625	-0.738	-0.031	-5.451	180.626
C3	186.107	112674.500	302287.400	-23.625	6.012	0.249	-5.451	180.905
C4	186.107	112674.500	302287.400	-23.625	12.762	0.528	-5.451	181.184
C5	186.107	112674.500	302287.400	-16.875	-7.488	-0.310	-3.893	181.904
C6	186.107	112674.500	302287.400	-16.875	-0.738	-0.031	-3.893	182.183
C7	186.107	112674.500	302287.400	-16.875	6.012	0.249	-3.893	182.462
C8	186.107	112674.500	302287.400	-16.875	12.762	0.528	-3.893	182.741
C9	186.107	112674.500	302287.400	-10.125	-7.488	-0.310	-2.336	183.461
C10	186.107	112674.500	302287.400	-10.125	-0.738	-0.031	-2.336	183.740
C11	186.107	112674.500	302287.400	-10.125	6.012	0.249	-2.336	184.020
C12	186.107	112674.500	302287.400	-10.125	12.762	0.528	-2.336	184.299
C13	186.107	112674.500	302287.400	-3.375	-7.488	-0.310	-0.779	185.019
C14	186.107	112674.500	302287.400	-3.375	-0.738	-0.031	-0.779	185.298
C15	186.107	112674.500	302287.400	-3.375	6.012	0.249	-0.779	185.577
C16	186.107	112674.500	302287.400	-3.375	12.762	0.528	-0.779	185.856
C17	186.107	112674.500	302287.400	3.375	-7.488	-0.310	0.779	186.576
C18	186.107	112674.500	302287.400	3.375	-0.738	-0.031	0.779	186.855
C19	186.107	112674.500	302287.400	3.375	6.012	0.249	0.779	187.134
C20	186.107	112674.500	302287.400	3.375	12.762	0.528	0.779	187.413
C21	186.107	112674.500	302287.400	10.125	-7.488	-0.310	2.336	188.133
C22	186.107	112674.500	302287.400	10.125	-0.738	-0.031	2.336	188.413
C23	186.107	112674.500	302287.400	10.125	6.012	0.249	2.336	188.692
C24	186.107	112674.500	302287.400	10.125	12.762	0.528	2.336	188.971
C25	186.107	112674.500	302287.400	16.875	-7.488	-0.310	3.893	189.691

C26	186.107	112674.500	302287.400	16.875	-0.738	-0.031	3.893	189.970
C27	186.107	112674.500	302287.400	16.875	6.012	0.249	3.893	190.249
C28	186.107	112674.500	302287.400	16.875	12.762	0.528	3.893	190.528
C29	186.107	112674.500	302287.400	23.625	-7.488	-0.310	5.451	191.248
C30	186.107	112674.500	302287.400	23.625	-0.738	-0.031	5.451	191.527
C31	186.107	112674.500	302287.400	23.625	6.012	0.249	5.451	191.806
C32	186.107	112674.500	302287.400	23.625	12.762	0.528	5.451	192.086
C33	186.107	112674.500	302287.400	-16.875	-14.238	-0.589	-3.893	181.625
C34	186.107	112674.500	302287.400	-10.125	-14.238	-0.589	-2.336	183.182
C35	186.107	112674.500	302287.400	-3.375	-14.238	-0.589	-0.779	184.739
C36	186.107	112674.500	302287.400	3.375	-14.238	-0.589	0.779	186.297
C37	186.107	112674.500	302287.400	10.125	-14.238	-0.589	2.336	187.854
C38	186.107	112674.500	302287.400	16.875	-14.238	-0.589	3.893	189.412
PW2	186.107	112674.500	302287.400	13.500	12.762	0.528	3.115	189.750
PW4	186.107	112674.500	302287.400	13.500	-14.238	-0.589	3.115	188.633
PW3	186.107	112674.500	302287.400	-20.250	12.762	0.528	-4.672	181.963
PW5	186.107	112674.500	302287.400	0.000	-14.238	-0.589	0.000	185.518
PW6	186.107	112674.500	302287.400	-23.625	-4.113	-0.170	-5.451	180.486
PW7	186.107	112674.500	302287.400	23.625	-4.113	-0.170	5.451	191.388
PW8	186.107	112674.500	302287.400	-3.375	-4.113	-0.170	-0.779	185.158
PW10	186.107	112674.500	302287.400	3.375	-5.113	-0.211	0.779	186.674
PW9	186.107	112674.500	302287.400	0.000	-0.738	-0.031	0.000	186.076
							Max	**192.086**

Hence maximum downward stress (192.0856 kN/m2) is less than safe bearing capacity (200 kN/m2) so **OK**.

In X-direction raft is divided in seven strips that is into seven equivalent beams, the beam with the respective soil pressure and moment are as follows.

Bending moment is obtained by coefficient (1/10) for midspan and coefficient (1/12) for support. We will use coefficient (1/10) throughout the span and provide uniform reinforcement along that direction. 'l' is centre to centre distance, from **IS 456 Cl. 22.5.1**

$+M = -M = wl^2/10$

In the Y-direction the raft is divided into eleven strips i.e., into eleven equivalent beams.

Beam	Width (m)	Length (m)	Coeff.	Equivalent Soil Pressure	Maximum moment per strip(kNm/m)
00-11	6.75	24.3	0.12	150.6312117	101.676
11-22	6.75	31.0	0.12	170.8872261	115.349
22-33	6.75	31.0	0.12	167.2380975	112.886
33-44	6.75	31.0	0.12	188.7537606	127.409
44-55	6.75	31.0	0.12	179.6827349	121.286
55-66	6.75	31.0	0.12	130.4503233	88.054
66-77	6.75	31.0	0.12	123.1454257	83.123
77-88	4	24.3	0.12	166.6528391	189.828
				Max	**189.828**

In the X-direction the raft is divided into six strips i.e., into six equivalent beams.

Beam	Width (m)	Length (m)	Coeff.	Equivalent Soil Pressure	Maximum moment per strip(kNm/m)
AA-BB	6.75	37.75	0.12	103.752	70.033
BB-CC	6.75	51.25	0.12	104.671	70.653
CC-DD	6.75	51.25	0.12	102.741	69.350
DD-EE	6.75	51.25	0.12	101.113	68.251
EE-FF	4	51.25	0.12	98.723	112.451
				Max	**112.451171**

Therefore, maximum moment is 189.828 kNm/m per strip.

Calculation of Depth of Foundation:

i. Calculation of Depth from Moment Criterion **(IS 456 : 2000, ANNEX G 1.1) :**

$$d = \sqrt{\frac{Mu}{Q \times b}}$$

Where, $Q = 0.36 \times fck \times \frac{Xu,lim}{d} \times (1 - 0.416\frac{Xu,lim}{d})$

Here,

f_{ck} = 30 MPa

For Fe500 grade of steel, $\frac{Xu,lim}{d} = 0.48$ **(IS 456: 2000, Cl. 38.1)**

$\therefore Q = 4.15\ and\ Mu = 189.828\ kNm$

$$\therefore d = 676.326\ mm$$

ii. Calculation of Depth from Two Way Shear:

Depth of raft will govern by two-way shear at one of the exterior columns. In case, location of critical shear is not obvious it may be necessary to check all locations. When shear reinforcement is not provided, the calculated shear stress at critical section shall not exceed $K_s \times \tau_c$. i.e. $\tau_v \leq K_s \times \tau_c$. **(IS 456 : 2000, Cl. 31.6.3.1)**

Where,

$K_s = (0.5 + \beta_c)$ but not greater than 1, βc being the ration of short side to long side of the column/capital; and

$\tau_c = 0.25\sqrt{fck}$ in limit state method of design and $0.16\sqrt{fck}$ in working stress method of design.

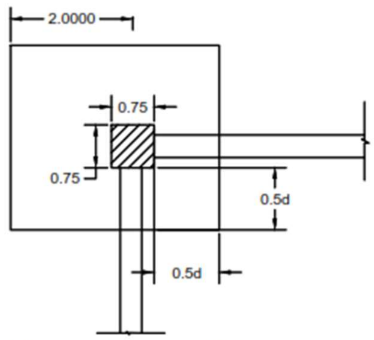
Here, $\beta_c = 1$

$K_s = 1+0.5 = 1.5 > 1$

Hence, $K_s = 1$

Shear strength of concrete (τ_c) = $0.25 \times \sqrt{30} = 1.369$ N/mm^2

For Corner Column C4



Corner Column C4

Column Load = 3152.868 kN

Perimeter (p_o) = 4 (0.5d + 750 + 2000)

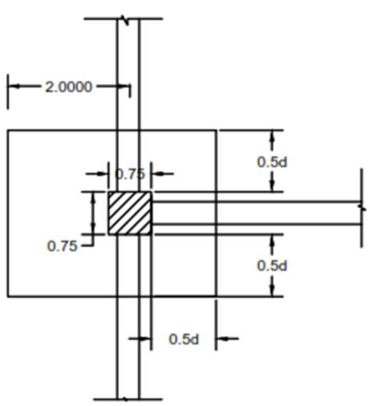
The nominal shear stress in flat slabs shall be taken as V/(p_o×d) where 'V' is the shear force due to design, 'p_0' is the periphery of the critical section and 'd' is the effective depth of the slab. **(IS 456: 2000, Cl. 31.6.2.1)**

$$\tau v = \frac{Vu}{(\,Po \times d)} = \frac{3152.868 \times 10^3}{4(0.5d + 750 + 2000) \times d}$$

$$or, 1.369 = \frac{3152.868 \times 10^3}{4(0.5d + 2750) \times d}$$

$\therefore d$ = 205.429 mm

For Edge Column C3



Edge Column C3

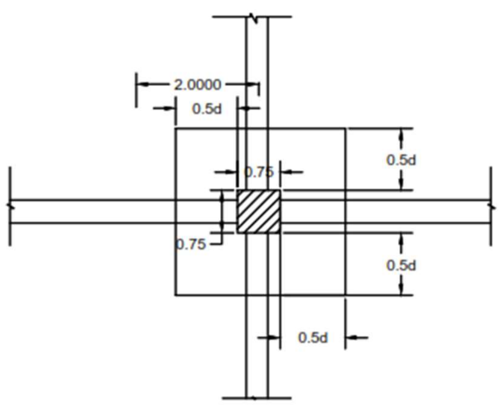
Column Load = 1600.377 kN

Perimeter (p_o) = d+750+0.5d+750+2000+0.5d+750+2000 = 6150 + 2d

$$\tau v = \frac{Vu}{(\,Po \times d)} = \frac{1600.377 \times 10^3}{(6150 + 2d) \times d}$$

$$or, 1.369 = \frac{1600.377 \times 10^3}{(6150 + 2d) \times d}$$

$$\therefore d = 198.657\ mm$$



Interior column C9

For Interior Column C17

Column Load = 11182.57 kN

Perimeter (p_o) = 4 (d + 750)

$$\tau v = \frac{Vu}{(\,Po \times d)} = \frac{11182.57 \times 10^3}{4(d + 750) \times d}$$

$$or, 1.369 = \frac{11182.57 \times 10^3}{4(d + 750) \times d}$$

$$\therefore d = 1102.41\ mm$$

Hence, **Depth is governed by two-way shear.**

But, depth of mat foundation cannot be less than 1m.

Adopt Overall Depth (D) = 1200 mm

Diameter of steel used (Ø) = 16 mm

Adopt Clear cover of 50 mm (IS456: 2000, Cl. 26.4.2.2)

Effective depth adopted = 1200 - (16/2) - 50 = 1142 mm

In Y-direction

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

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

$$or, 189.828 \times 10^6 = 0.87 \times 500 \times \mathrm{A_{st}} \times (1142 - \frac{500 \times \mathrm{A_{st}}}{30 \times 1000})$$

Solving we get,

$$A_{st} = 386.33\ mm^2$$

Minimum reinforcement in slab = 0.12%×1000×1000 = 1200 mm^2 **(IS 456: 2000, Cl. 26.5.2.1)**

Adopted A_{st} = 1200 mm^2

Using 16 mm Ø bars,

$$Spacing\ (S) = \frac{A_{bar}}{A_{st}} \times 1000 = \frac{201.061}{1200} \times 1000 = 167.55\ mm$$

Hence, **Provide 16 mm Ø Bars @ 150 mm c/c in Y direction.**

Therefore, $A_{st_{provided}} = \left(\frac{201.061}{150}\right) \times 1000 = 1340.406\ mm^2$

In X-direction:

Adopt effective depth = 1200 – 50 – (16/2) – 16 = 1126 mm

Reinforcement in longer direction is given by,

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

$$or, 112.451 \times 10^6 = 0.87 \times 500 \times \mathrm{A_{st}} \times (1126 - \frac{500 \times \mathrm{A_{st}}}{30 \times 1000})$$

Solving we get,

$A_{st} = 282.4\ mm^2$

Minimum reinforcement in slab = 0.12%×1000×1000 = 1200 mm^2 **(IS 456 : 2000, Cl. 26.5.2.1)**

Adopted A_{st} = 1200 mm^2

Using 16 mm Ø bars,

$$Spacing\ (S) = \frac{A_{bar}}{A_{st}} \times 1000 = \frac{201.061}{1200} \times 1000 = 167.55\ mm$$

Hence, **Provide 16 mm Ø Bars @ 150 mm c/c in X direction.**

Therefore, $A_{st_{provided}} = \left(\frac{201.061}{150}\right) \times 1000 = 1340.406\ mm^2$

Check for Development Length:

Bond Stress (τ_{bd}) = 1.5 N/mm^2, For M30 Concrete. This value can be increased by 60% for High Strength Steel. **(IS 456: 2000, Cl. 26.2.1.1)**

The development length (L_d) is given by **(IS 456: 2000, Cl. 26.2.1)**

$$L_d = \frac{\emptyset \times \sigma_s}{4\tau_{bd}} = \frac{16 \times 0.87 \times 500}{4 \times 1.5 \times 1.6} = 725\ mm$$

$L_d \leq 1.3\frac{M_l}{V} + l_o$ **(IS 456: 2000, Cl. 26.2.1)**

l_o= Effective depth or 12$\emptyset$, whichever is greater.

$L_d \leq 1.3\frac{483.67\times10^6}{5928.919\times10^3} + 936 = 1042.05\ mm$ **(OK)**

Load Transfer from Column to footing:

Nominal bearing stress in column concrete $(\sigma_{br}) = \frac{P_u}{A_c} = \frac{11182.57\times10^3}{750\times750} = 19.88\ N/mm^2$

Allowable bearing stress = 0.45×f_{ck} = 0.45×30 = 13.5 N/mm^2 < 19.88

(IS 456: 2000, Cl. 34.4)

When the permissible bearing stress on the concrete in the supporting or supported member would be exceeded, reinforcement shall be provided for developing the excess force by dowels. **(IS 456: 2000, Cl. 34.4.1)**

Dowel of at least 0.5% of the cross-sectional area of the supported column and a minimum of four bars shall be provided. Diameter of the Dowels shall not exceed the diameter of column bar by more than 3 mm. **(IS 456 : 2000, Cl. 34.4.1)**

Area of Dowels Bar = 0.5% × 750 × 750 = 2812.5 mm^2

Provide 25 mm Ø as Dowel bar.

Development length for dowel bar = $\frac{\emptyset \times \sigma_s}{4 \times \tau_{bd}} = \frac{25 \times 0.87 \times 500}{4 \times 1.5 \times 1.6} = 1132.812\ mm$

Length of Dowel bar into column = L_d of column bar = 906 mm

Length of Dowel bar into footing = L_d of dowel bar = 1132.812 mm

Hence,

Provide length of Dowel bar in column = 950mm

Provide length of Dowel bar in footing = 1200 mm

Use **6 – 25 mm Ø Bars as Dowel Bar**, then $(Ast)_{provided} = 4 \times \pi \times 12.5^2 = 2943.75\ mm^2 > 2812.5\ mm^2$

Chair Bars:

Height of chair = Height of footing – (2 x clear cover) – (Dia of bottom main bar) – (Dia of top main bar + Dia of top distribution bar)

= 1200 – (2x50) – 16 – (16 + 16) = 1052 mm

Length of head = (2 x spacing of distribution bar) + (2x25)

= 350 mm

Length of leg = (2xspacing of bottom main bar) + 50

= 350 mm

Hence, provide chair of height 1052mm, head 350mm and leg 350mm.

6.3 Ductility and Ductile Detailing

A ductile material is the one that can undergo larder strains while resisting loads. When applied to reinforced concrete members and structures, **the term ductility implies the ability to sustain significant in-elastic deformations prior to collapse**. It is the ratio of absolute maximum deformation or curvature or rotation to the corresponding yield deformation. **Under reinforced section shows ductile deformation whereas over reinforced section shows brittle deformation, so, ORS should be avoided while designing structural elements.**

6.3.1 Significance of Ductility

While a ductile structure is subjected to overloading it will tend to deform in-elastically and in doing so, will re-distribute the excess load to elastic parts of the structure. This concept can be utilized in several ways:

a. If a structure is ductile, it can be expected to adapt to unexpected overloads, load reversals, impact and structural movements due to foundation settlement and volume changes. These items are generally ignored in analysis and design but assumed to have been taken care of by the presence of some ductility in the structure.

b. If a structure is ductile, its occupant will have sufficient warning of the impending failure thus reducing the probability of loss of life in the event of collapse.

c. The limit state design procedure assumes that all the critical section in the structure will reach their maximum capacities at design load for the structure. For this to occur, all joints and splices must be able to withstand forces and deformations corresponding to yielding of the reinforcement.

6.3.2 Variables Affecting Ductility

a. Tension steel ratio p
b. Compression steel ratio p
c. Shape of cross-section
d. Lateral reinforcement.

6.3.3 Design for Ductility

Selection of cross-section having adequate strength is rather easy but it's more difficult to achieve desired strength as well as ductility. For this the designer should pay attention to detailing of reinforcement, bar cut-offs, splicing and joint details. Sufficient ductility can be ensured by following certain simple design details:

a. The structural layout should be simple and regular avoiding offsets of beam to columns, or offsets of column from floor to floor. Changes in stiffness should be gradual from floor to floor.

b. The amount of tensile reinforcement in beams should be restricted and more compression reinforcement should be provided. The later should be enclosed by stirrups to prevent it from buckling.

c. Beams and columns in a reinforced concrete frame should be designed in such a manner that inelasticity is confined to beams only and column remains elastic. To ensure this,

$$\sum M_{column} > 1.2 \sum M_{beam}$$

d. The shear reinforcement should be adequate to ensure that the strength in shear exceeds the strength in flexure and thus prevent from non-ductile failure.

e. Splices and anchorages must be sufficient to prevent bond failures.

f. Beam-column connections should be made **monolithic.**

g. The reversal of stresses in beam and column due to reversal of earthquake force must be taken into account in the design by appropriate reinforcement.

6.3.4 Detailing for Ductility (Based on IS 13920: 2016)

- At least two bars should be provided continuously both at top and bottom.
- The positive moment resistance at the face of a joint should not be less than one-half of the negative moment resistance provided at that face of the joint.
- Neither the negative nor the positive moment resistance at any section along the member length should be less than one-fourth of the maximum moment resistance provided at the face of either joint.

6.4 Monolithic Beam to Column Joints

A beam-column joint is a very critical element in reinforced concrete construction where the elements intersect in all the three dimensions. Joints are most critical because they ensure continuity of structure and transfer forces that are present at the ends of the members into and through the joint. Frequently joints are points of weakness due to lack of adequate anchorage for bars entering the joint from the columns and beams.

The code is silent regarding the design of beam-column joints. A joint should maintain its integrity in the core for smooth transfer of stress and should be designed so that it is stronger than the members framing into it. Failure should not occur within the joint. **In fact, failure due to over loading should occur in beams through large flexural cracking and plastic hinging but not in columns.**

The joint shear causes diagonal tension and compression in the joint. With each reversal of seismic loading, the joint shear changes sign causing cracks due to diagonal tension in both directions. Moreover, the nature of bond stress also changes in the joint around the beam and column reinforcement. It causes splitting stresses in the concrete around the bar.

Quite often, the beam-column joint is under a severe congestion of reinforcement due to too many bars converging within the limited space of the joint. By selecting little larger concrete area and lower reinforcement percentage, it is possible to avoid congestion of steel.

6.5 Torsion in Buildings

A three-dimensional building has series of frames in orthogonal direction X & Y to resist gravity loads and lateral loads. A floor is generally quite rigid in its own plane and each frame may have different stiffness distribution and mass distribution.

The earthquake force acts through center of mass and is resisted by the building through its center of rigidity. This leads to horizontal twisting of building and is called torsion. The floor generally rotates as a rigid body. The magnitude of the torsional moment depends on the distance between center of mass and center of rigidity which is referred as eccentricity.

A three-dimensional analysis of building using general purpose matrix analysis computer programs is able to take care of eccentricity but without displaying its magnitude. However, there is no general purpose computer which is able to account for the design eccentricity because there is no direct method to compute center of rigidity or shear center of each floor/story. This is the main reason why most of designers adopt approximate methods for the torsional analysis of building. Several studies made of structural damages during past wind and earthquakes reveal that the torsion is the most critical factor causing partial structural damages or complete collapse of buildings.

6.6 Nominal Cover

A reinforcing bar must be surrounded by concrete for the following principle reasons:

- To develop the desired strength of a bar by ensuring proper bond between concrete and steel through-out its perimeter.
- To provide protection against **corrosion** and **fire**.

Table 6.2Nominal cover

Exposure	Cover (mm)
Mild	25
Moderate	30
Severe	45
Very severe	50
Extreme	75

6.7 Curtailment of Tension Reinforcement in Flexural Member

A reinforcement bar is curtailed for one or more of the following reasons:

- <u>For economy:</u> Bending moment varies along the span of a member. It is a general practice to vary the number of bars i.e., curtail bars, at suitable sections where the bending is less.
- <u>Standard length:</u> If a member is longer and the available bars are shorter or vice versa, a joint or curtailment becomes necessary.

6.8 Fire Resistance of Concrete Elements

The principles employed in the calculations of the fire resistance of structural elements are based on the international research data on the insulating properties of the concrete, strength of concrete and steel reinforcement/pre-stressing tendons at high temperatures (≈700°C) and considerations of such effects as spalling, disposition of reinforcement and the nature of load distribution.

The factors that influence the fire resistance of concrete elements are as follows:

- **a.** size and shape of structural element
- **b.** loads distribution
- **c.** disposition and properties of the reinforcing bars and pre-stressing tendons
- **d.** type of aggregate and concrete
- **e.** end conditions
- **f.** cover to reinforcement

7. Discussion:

Various discussions and conclusions are given below in sub-heads:

7.1 Structural designs with or without seismic considerations

There are all-together 25 combinations among which two combos don't include seismic considerations. During our analysis, we found out seismic considerations considerably increase the values of bending moment and shear forces and this is encountered in the structural elements. Bending moment and shear, being important in designing of such member (esp. beam and column) should be justified by addressing such increases esp. in terms of providing steel areas.

7.2 ETABS and its limitations

ETABS is widely used structural analysis software. Main benefits of using such software are that they make analysis easy and fast. But great care should be given during input of data, because degree to which we obtain correct output depends upon degree to which we input values in computer (GIGO).

Limitations of Software esp. ETABS is discussed below:

- ETABS does-not consider a single worst combination during design of single structural element. It takes into account all worst values of several combinations (individual worst value of M_x, M_y and P_u from all combinations) and designs each components of single element with that.
- Another limitation of ETABS is that it does not follow Ductile detailing code completely, which is must during detailing of structural elements.

7.3 Check criteria for input data

After input of data/ loads, analysis is performed in ETABS. To be sure that all loads are input correctly in ETABS or any other analysis tool, a simple check can be done. For this, total summation of un-factored base reactions must be close to building weight calculated during preliminary design. If it's within tolerance limit, we can be sure input data are correct and can perform further design.

A careful study of frequencies and mode shape may give an idea of correctness of input data. There is a need to very carefully interpret the computer results of complete frame analysis tagging into account the sequence of construction.

7.4 Static analysis

In static analysis, only one mode is considered and its modal time period is longest one. It considers building shows only one modal shape in action of lateral forces. In this static analysis, though having longest time period, 90% of total seismic mass has participated during earthquake.

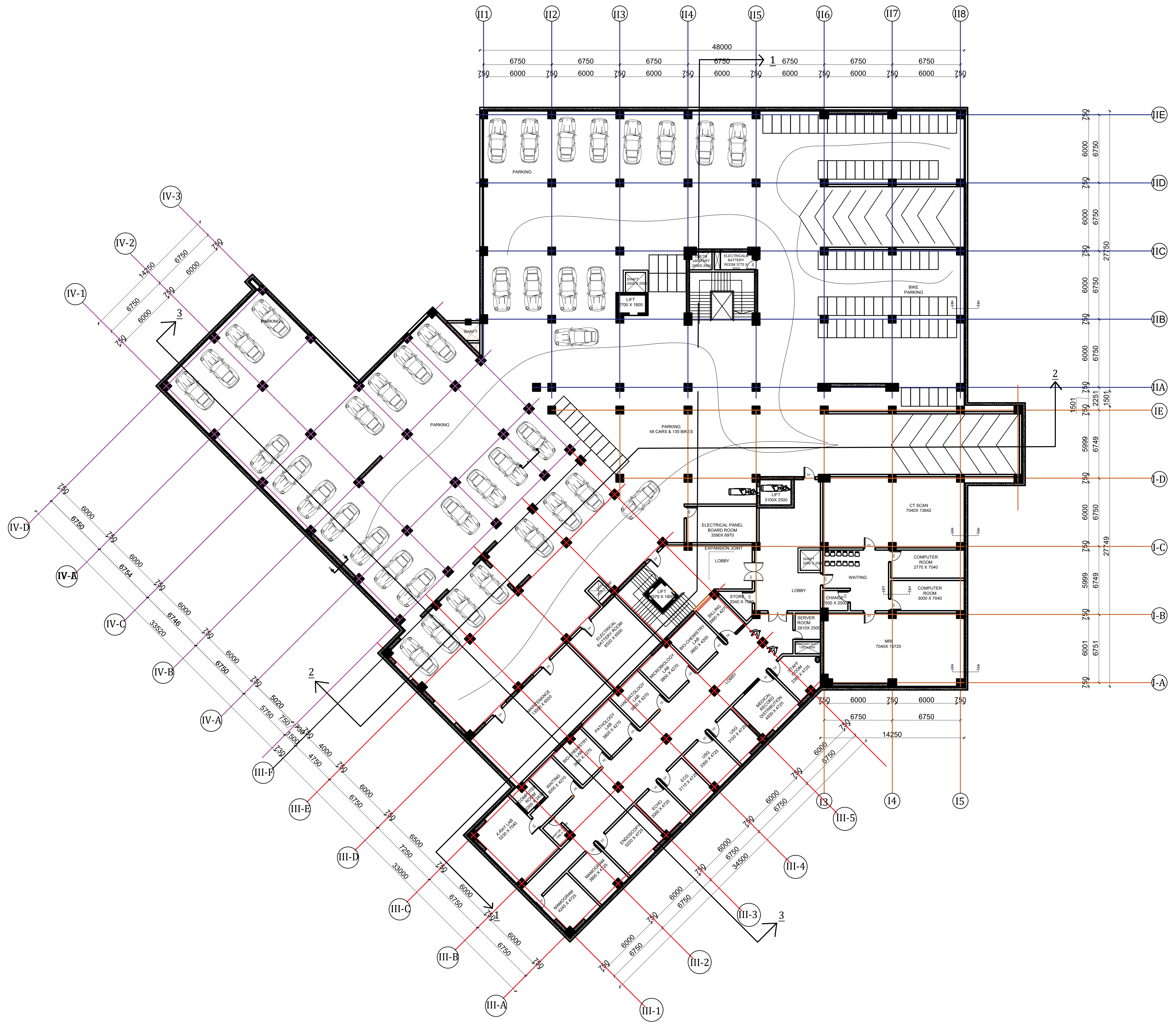
7.5 Percentage of Reinforcement

The percentage of reinforcement required for columns were found to be less than 4% which is allowed by IS: 456-2000 which fulfills the condition for design of column without increasing the grade of concrete.

8. Conclusion

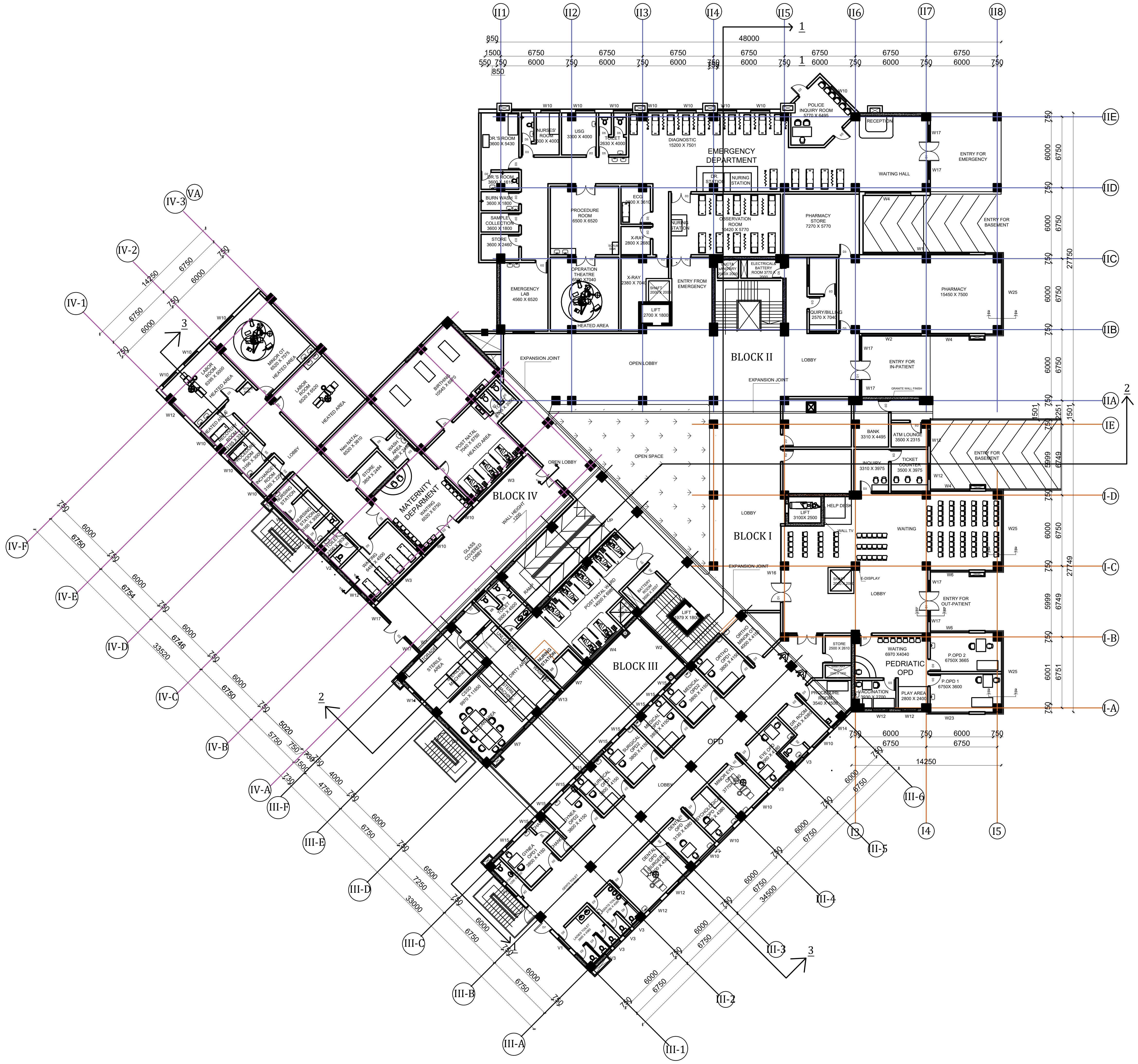
During the course of this project different problems were encountered and solutions to these problems were effectively found under the guidance of Supervisor Asst. Prof. Arun Paudel. The project gave us general idea of how the designs of different structural elements are carried out and how the detailing for earthquake resistant structure is done. We hope this project report will help others to understand basic behavior of structure under the action of earthquake and also the procedure required for the safe design of such structure.

The project **"Seismic Analysis and Design of Multistoried Hospital Building"** helped us to understand the effect of earthquake load on structural elements and in structure as-a-whole.



BASEMENT PLAN
AREA= 4308.411 SQ.M.

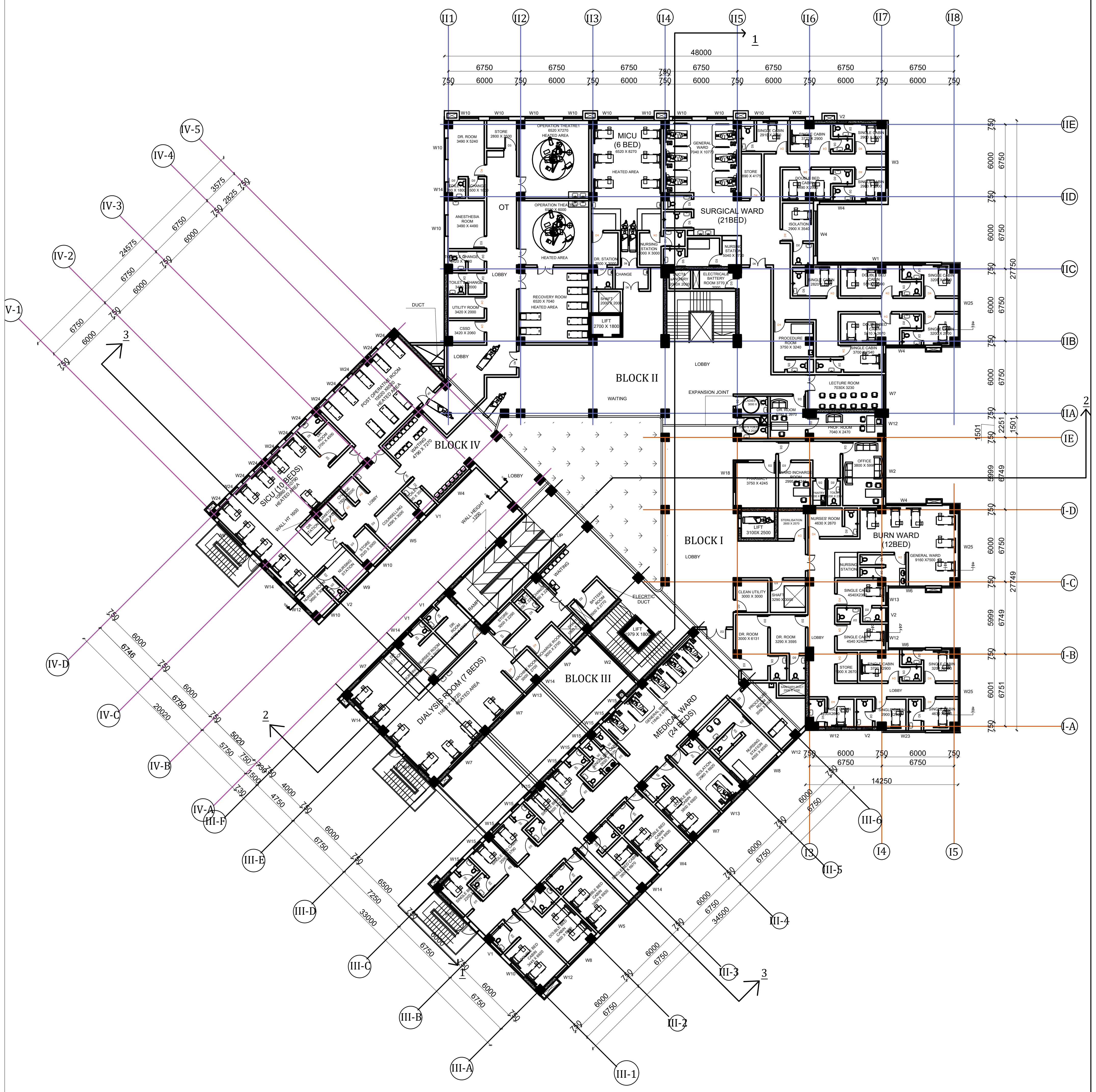
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : ARCHITECTURAL PLAN	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. 1 Sheet No. 1



GROUND FLOOR PLAN
AREA= 3922.742 SQ.M.

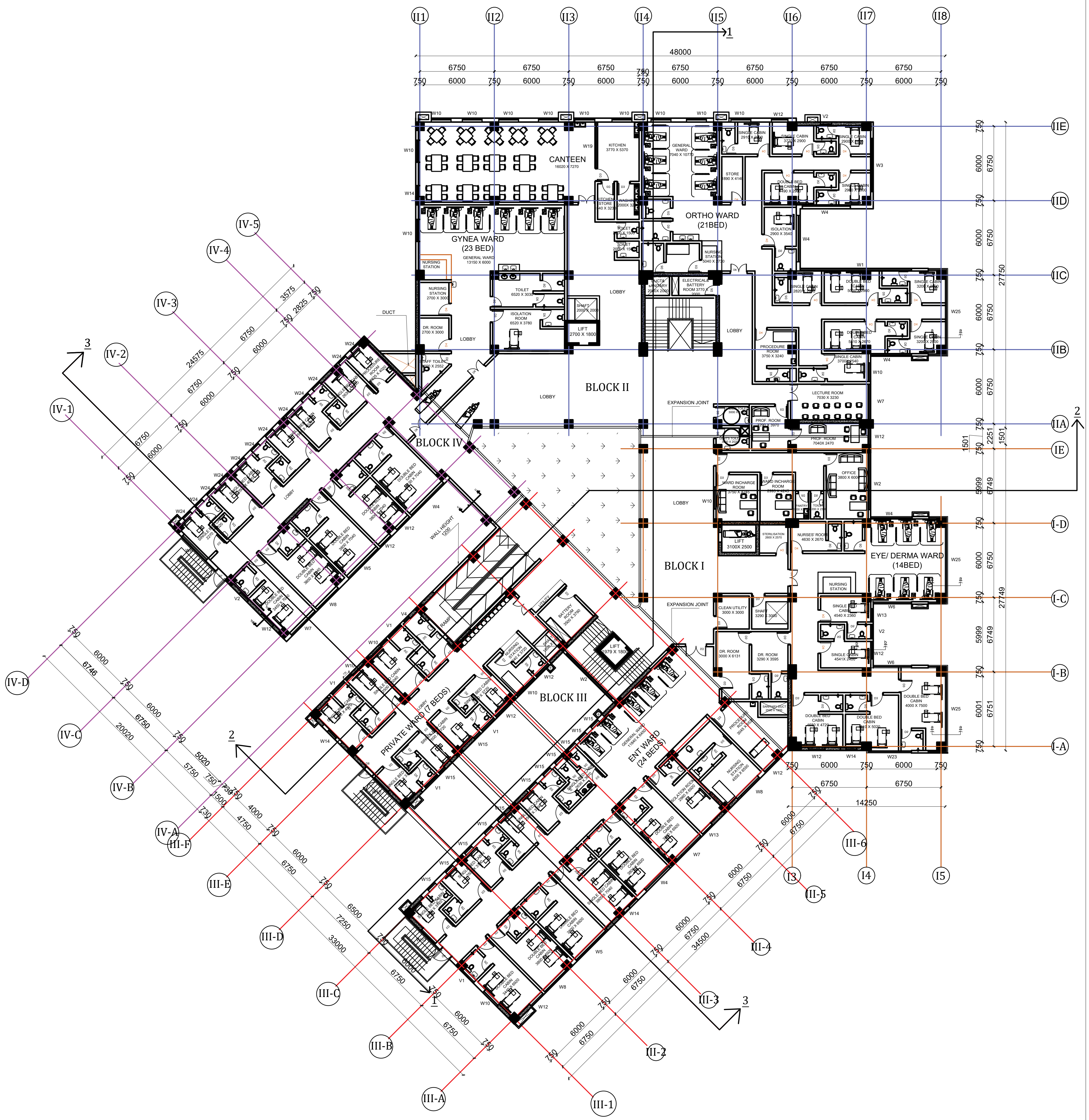
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : ARCHITECTURAL PLAN	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. 2 Sheet No. 2

FIRST FLOOR PLAN
AREA= 3249.139 SQ.M.

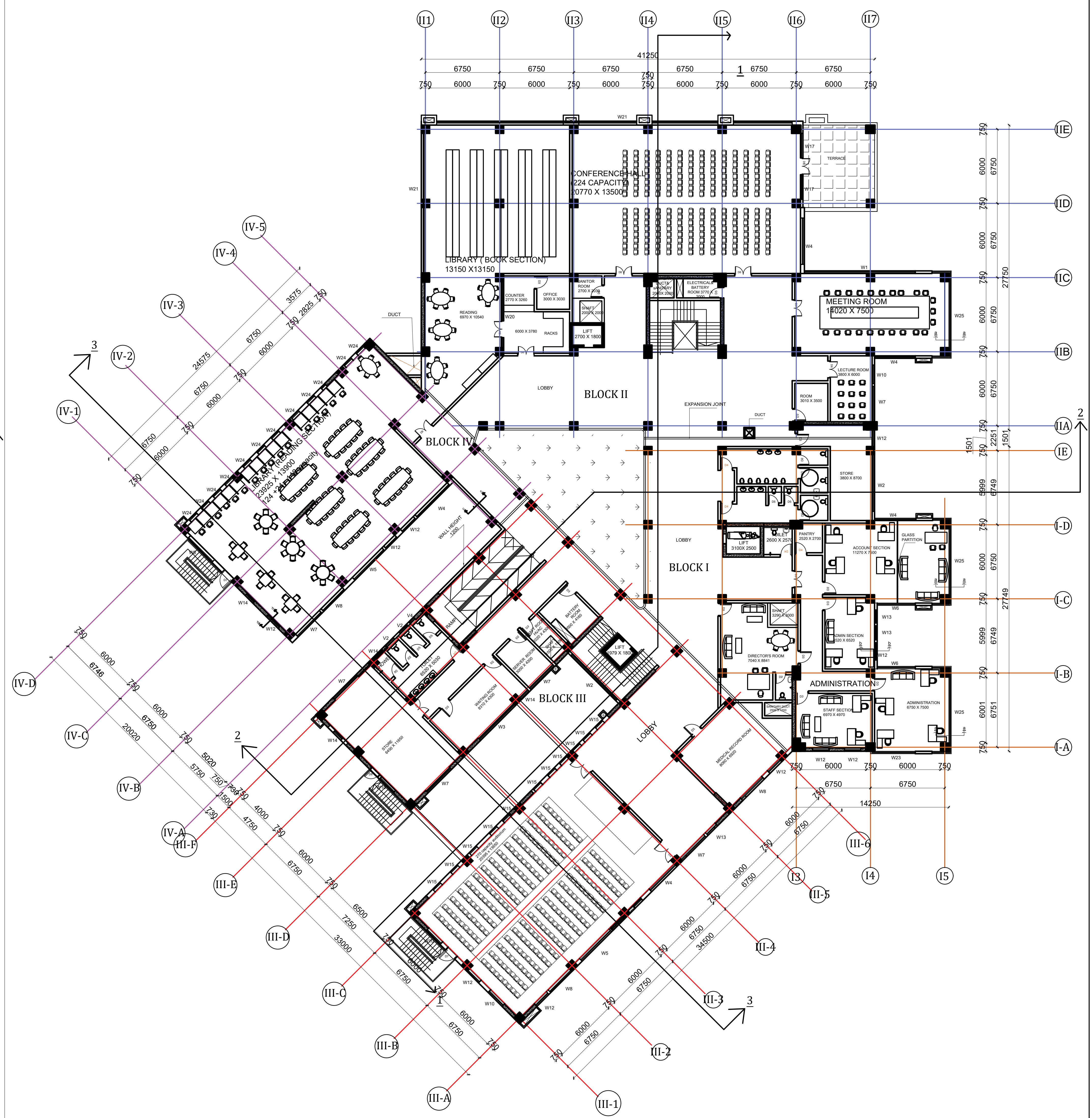


SECOND FLOOR PLAN
AREA=3249.139 SQ.M.

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : ARCHITECTURAL PLAN	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. 3 Sheet No. 3

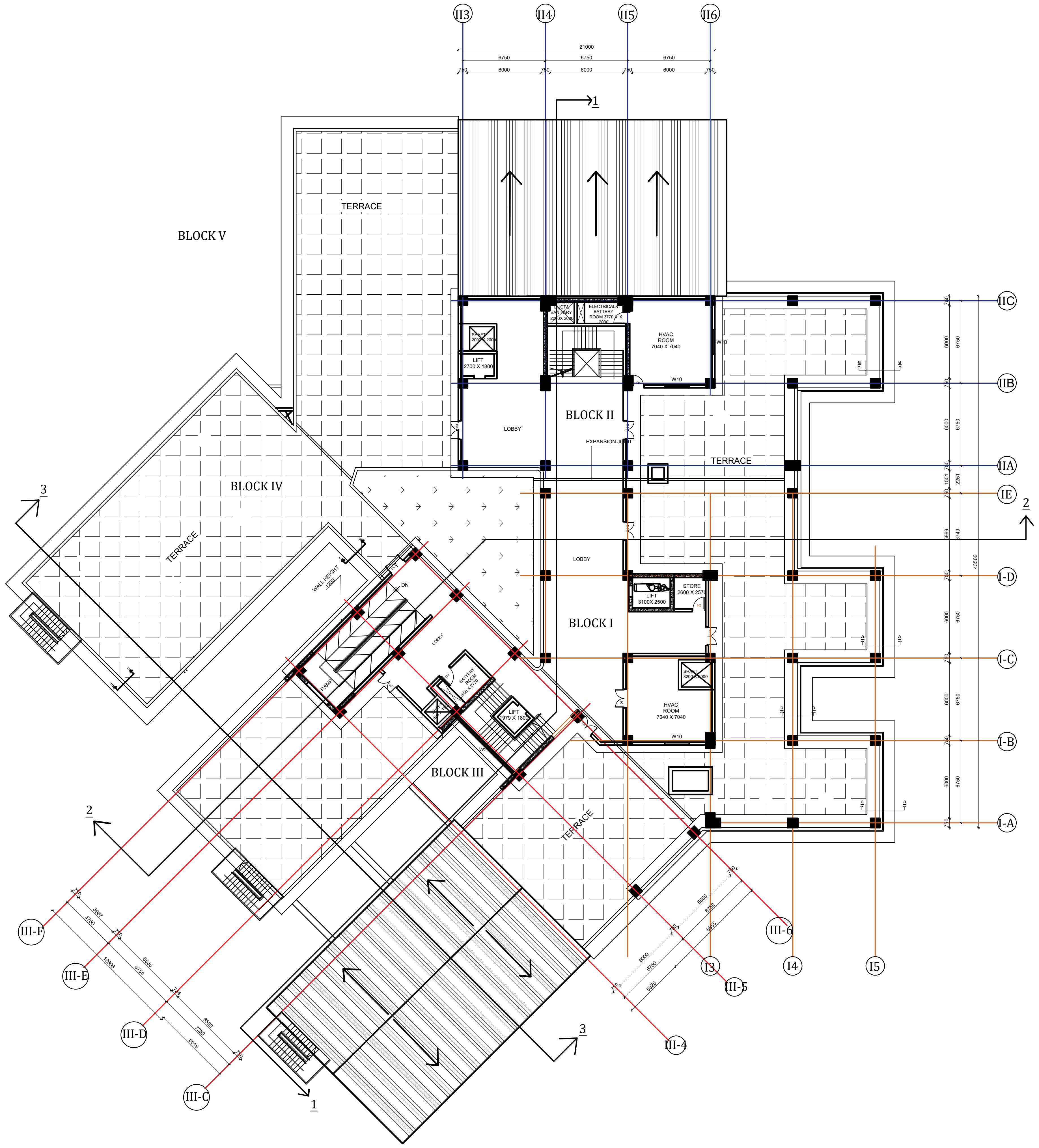


THIRD FLOOR PLAN
AREA=3249.139 SQ.M.

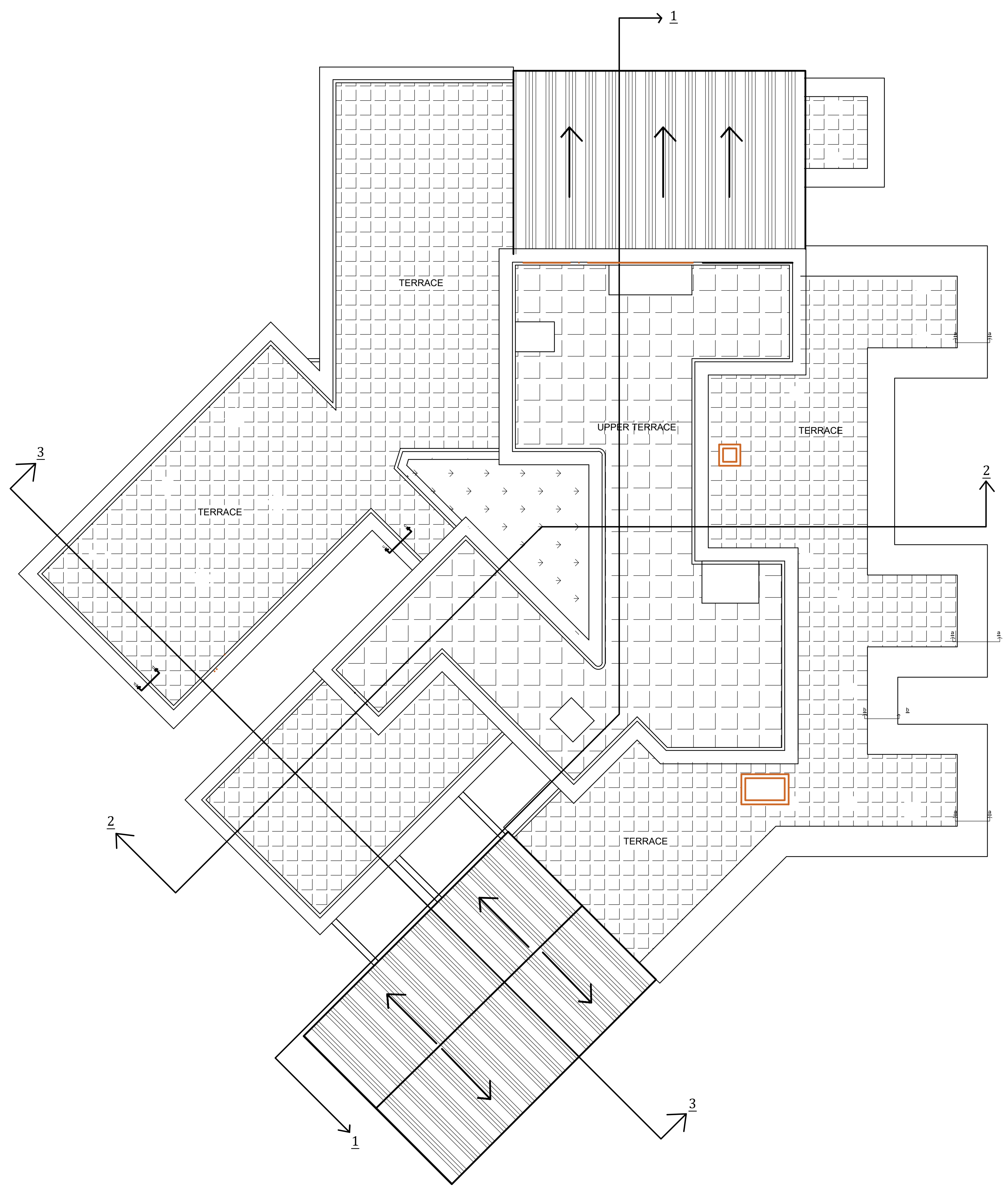


FOURTH FLOOR PLAN
AREA= 3194.75 SQ.M.

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : ARCHITECTURAL PLAN	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. 4 Sheet No. 4



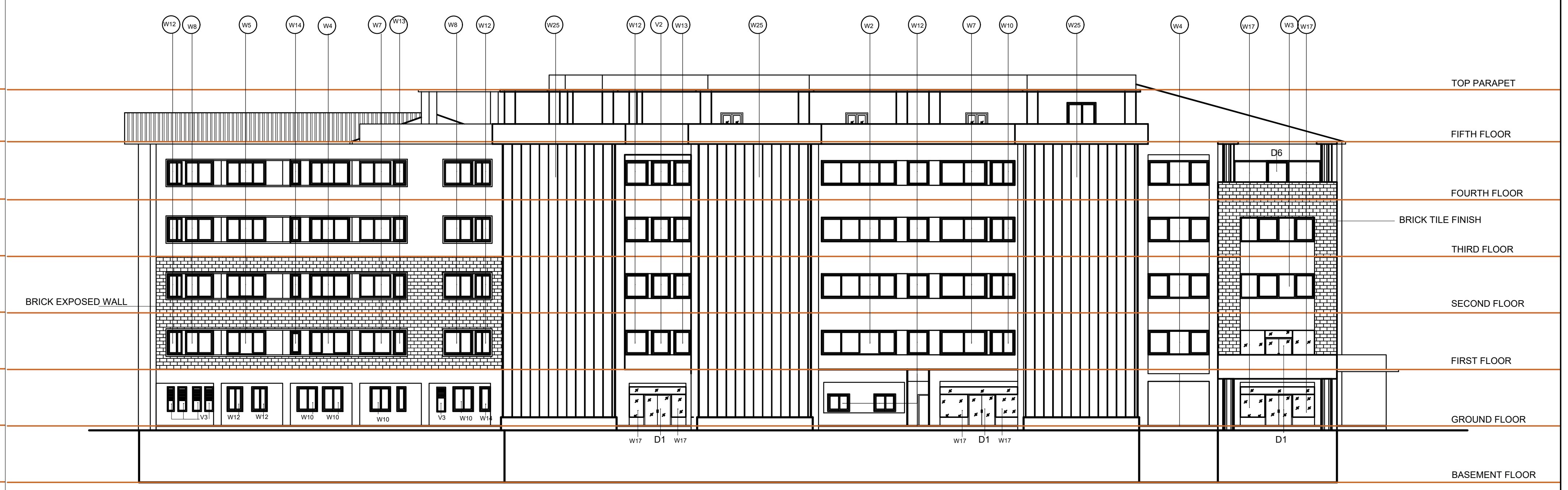
TOP FLOOR PLAN
AREA= 564.76 SQ.M.



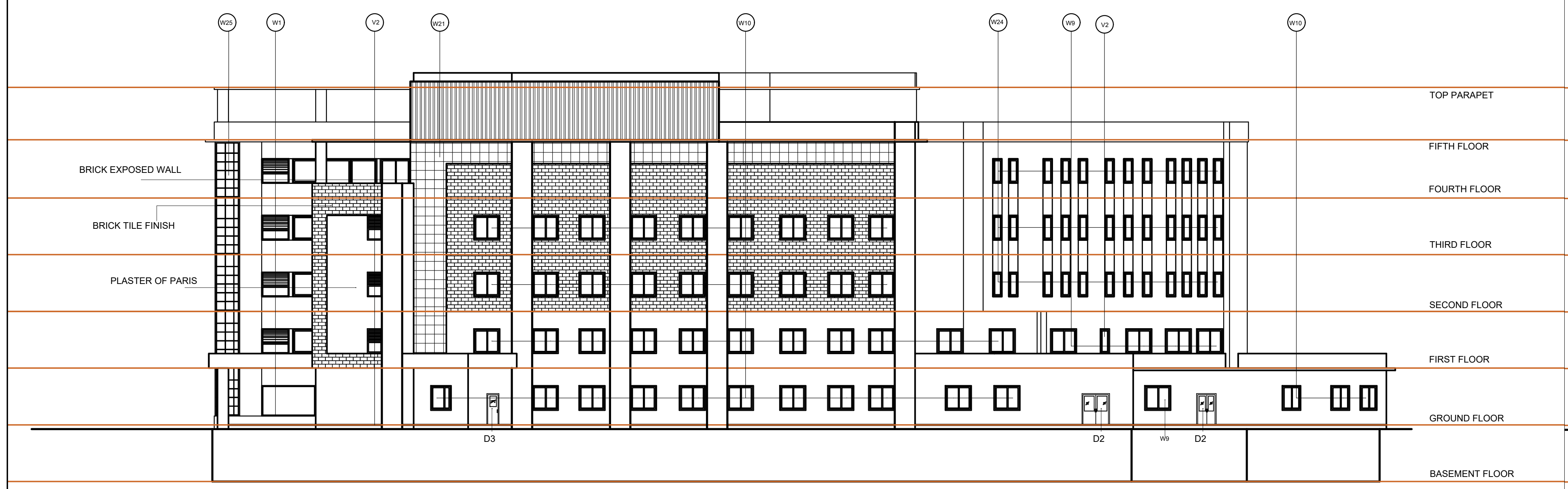
ROOF PLAN
AREA= 564.76 SQ.M.

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : ARCHITECTURAL PLAN	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. 5 Sheet No. 5

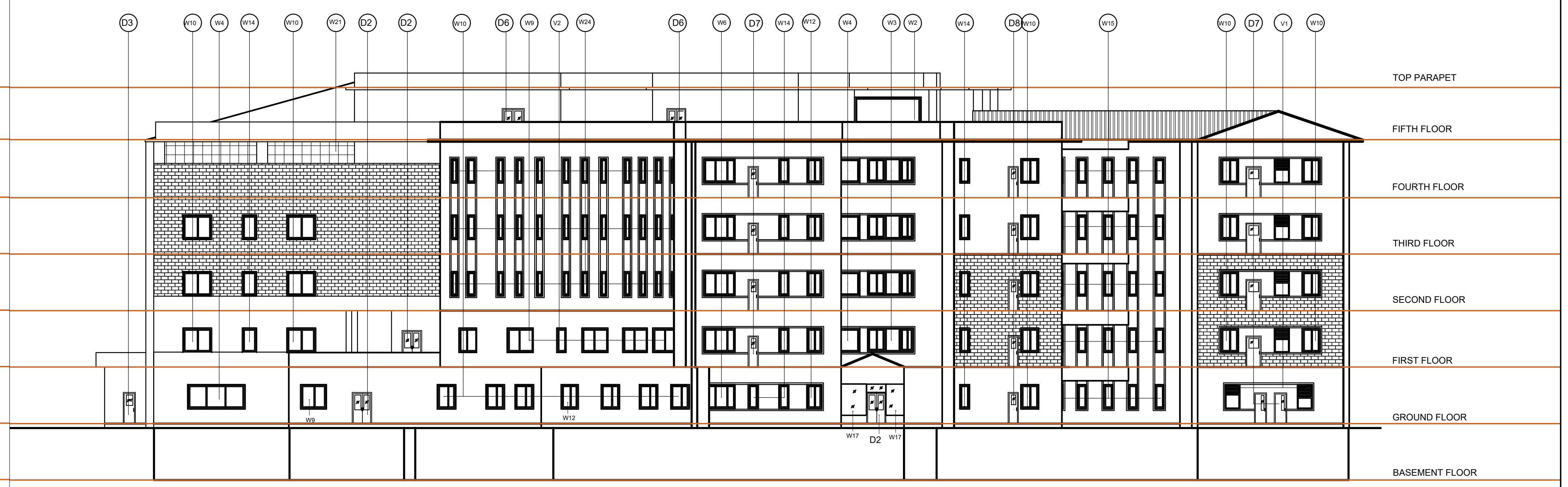
SOUTH ELEVATION



EAST ELEVATION



NORTH ELEVATION



WEST ELEVATION

TRIBHUVAN UNIVERSITY
INSTITUTE OF
ENGINEERING
PULCHOWK CAMPUS

Project Title :

EARTHQUAKE RESISTANT
DESIGN OF MULTI-STORIED
HOSPITAL BUILDING

Sheet Title :

ELEVATIONS

Group Members :

Dipak Dhakal	(075 BCE 051)
Kiran Kumar Maharjan	(075 BCE 075)
Kushal Sharma	(075 BCE 076)
Nimee Tiwari	(075 BCE 087)
Nishant Awasthi	(075 BCE 090)
Nishchal Nath Sigdel	(075 BCE 091)

Project Supervisor :

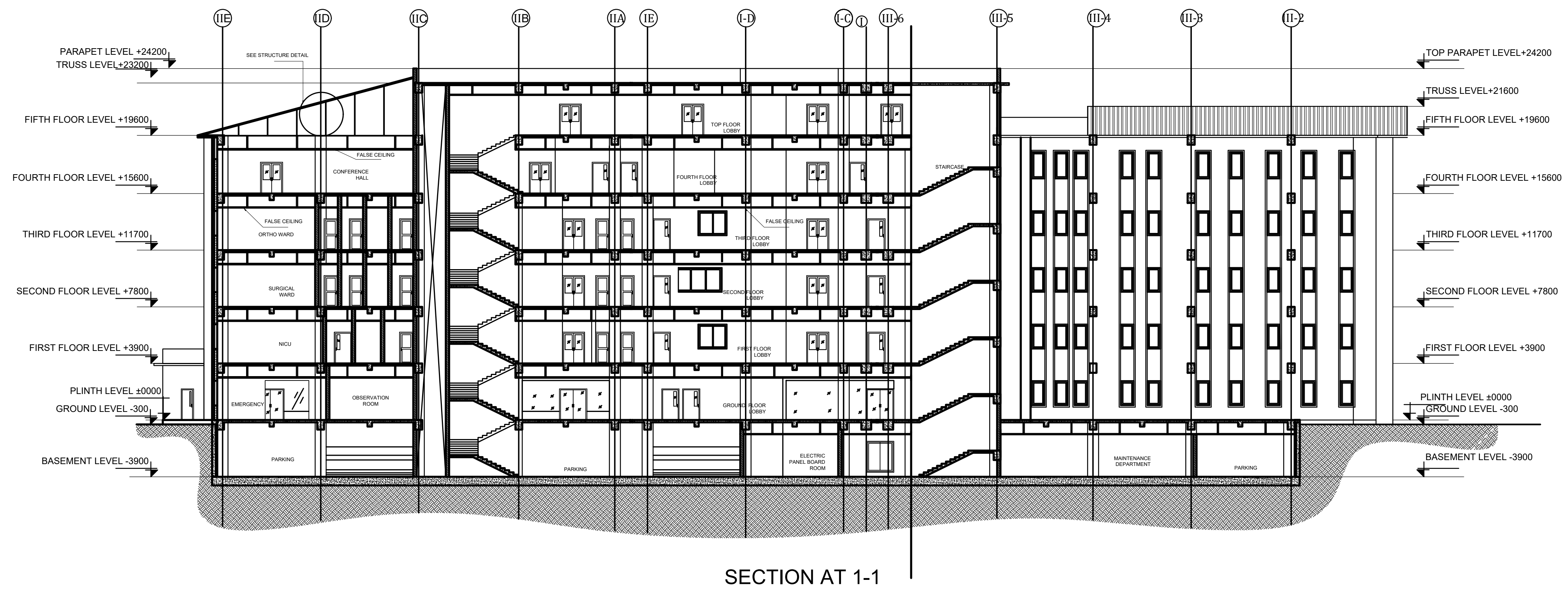
Asst. Prof. Arun Paudel

Date : 2080 / 01 / 14

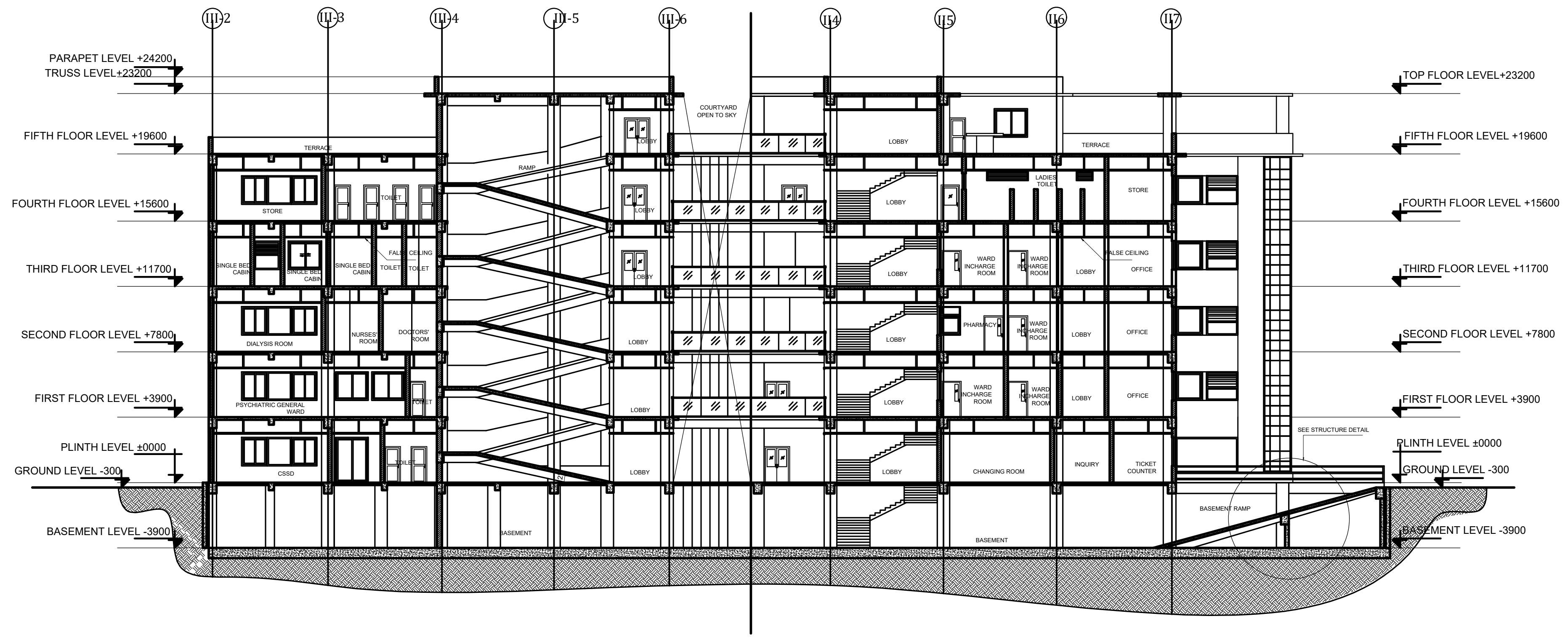
Fit to scale

DWG No. 6

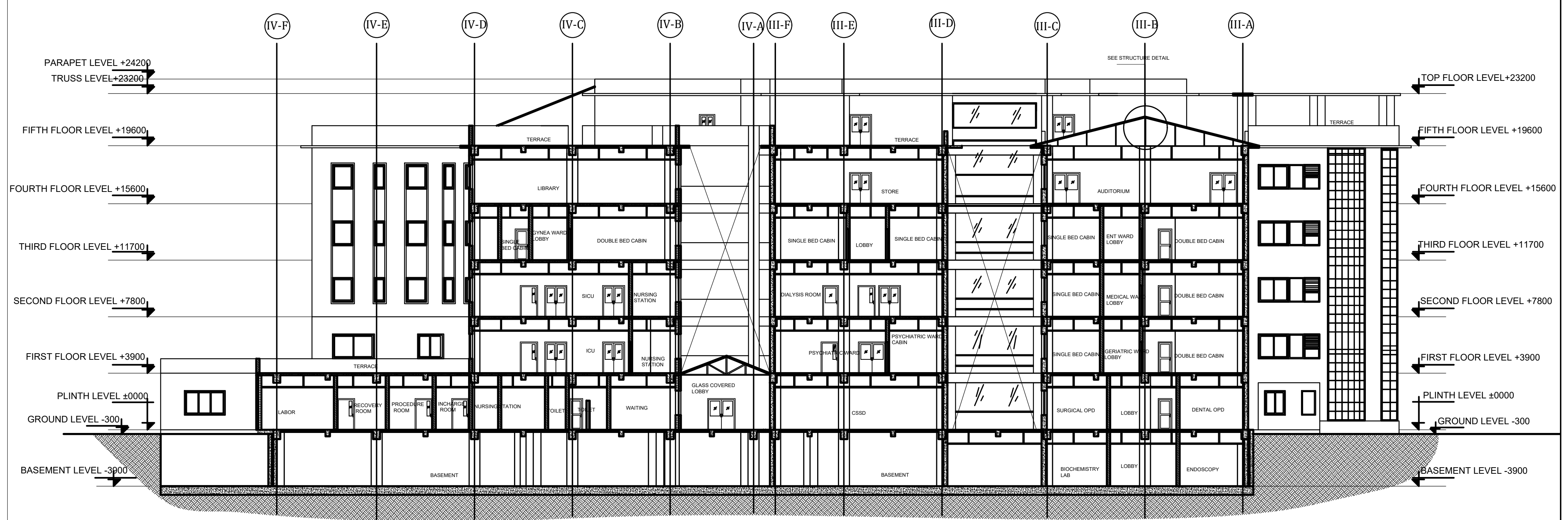
Sheet No. 6



SECTION AT 1-1

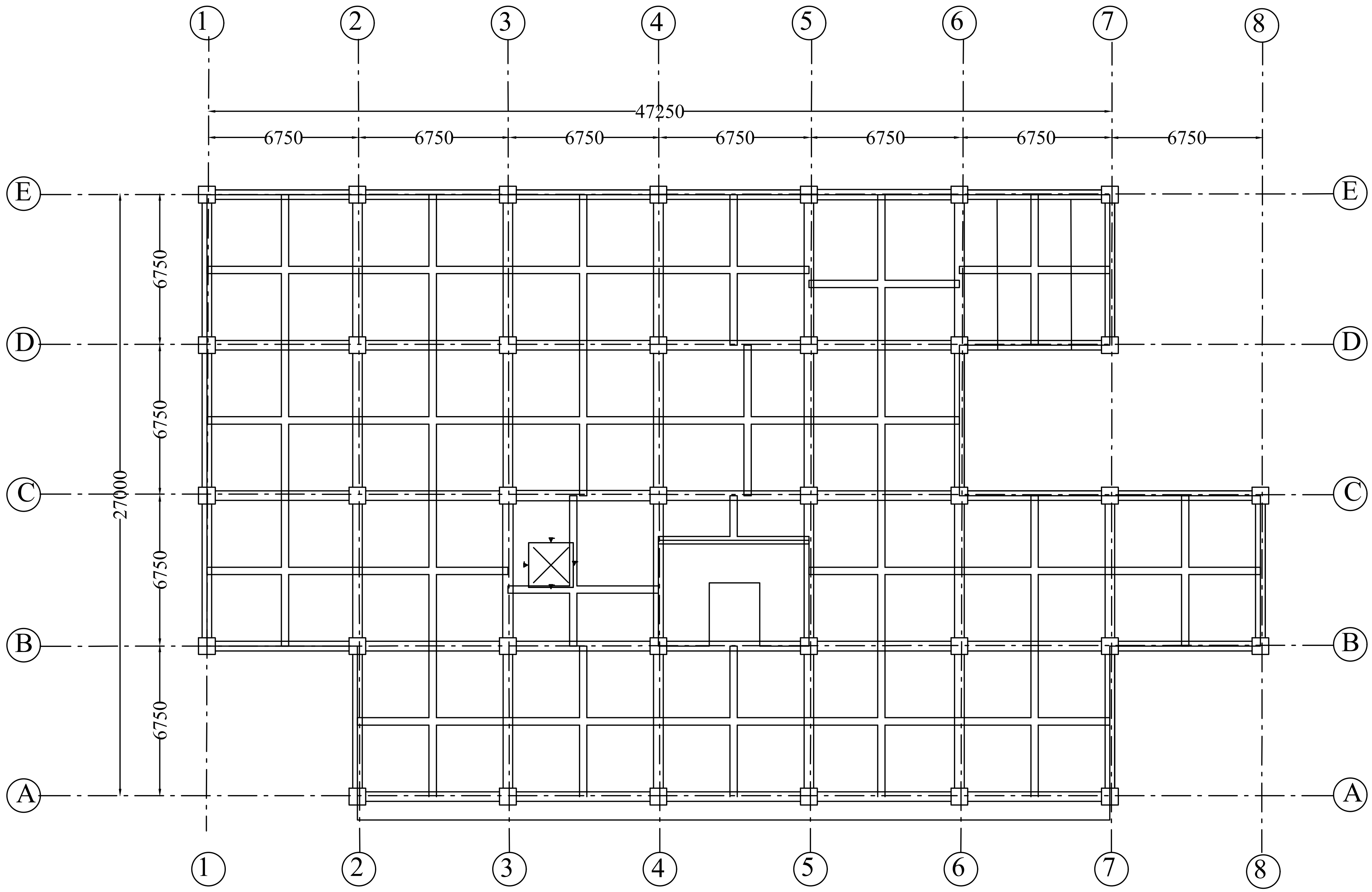


SECTION AT 2-2



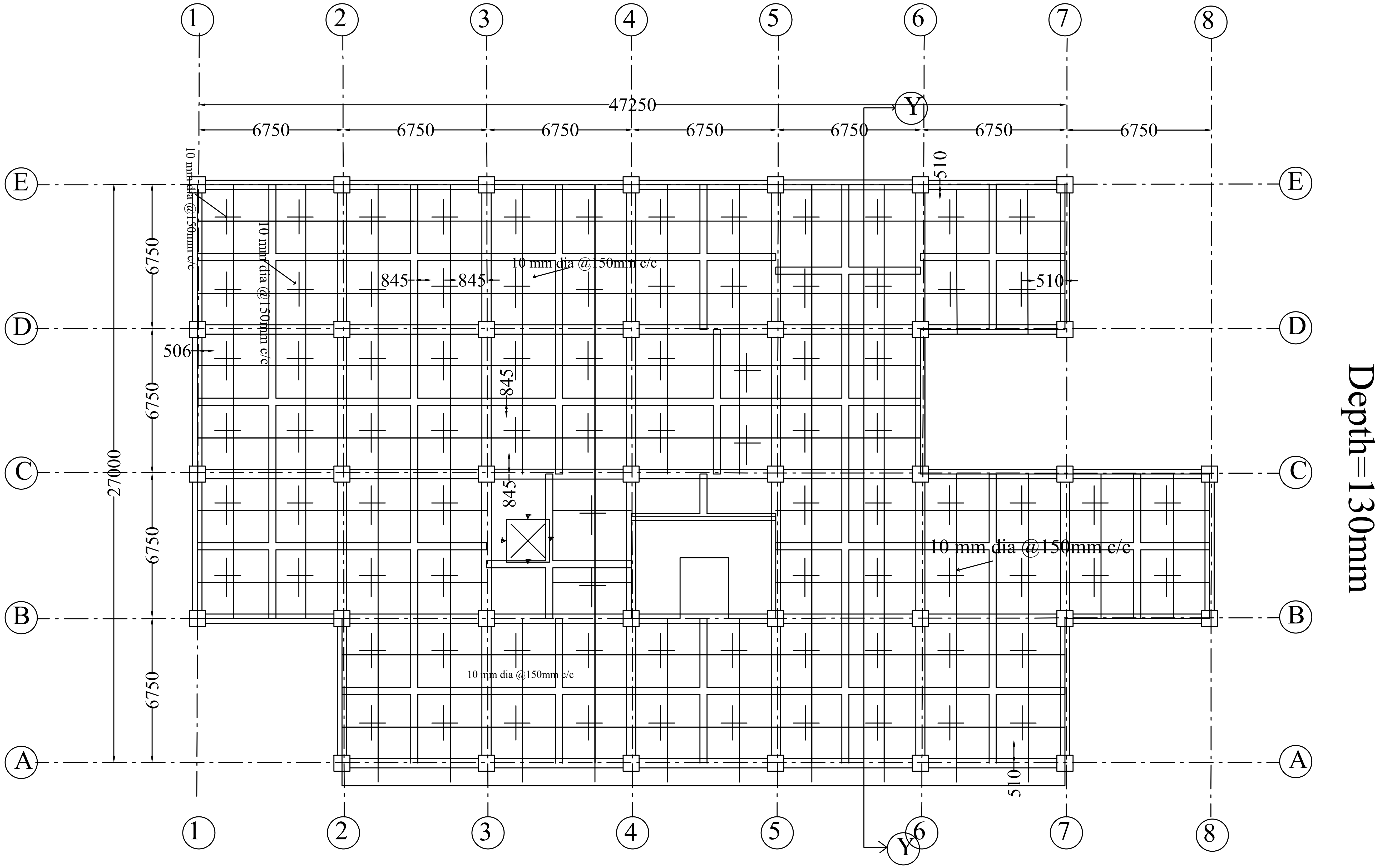
SECTION AT 3-3

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : SECTIONS	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. 7 Sheet No. 7



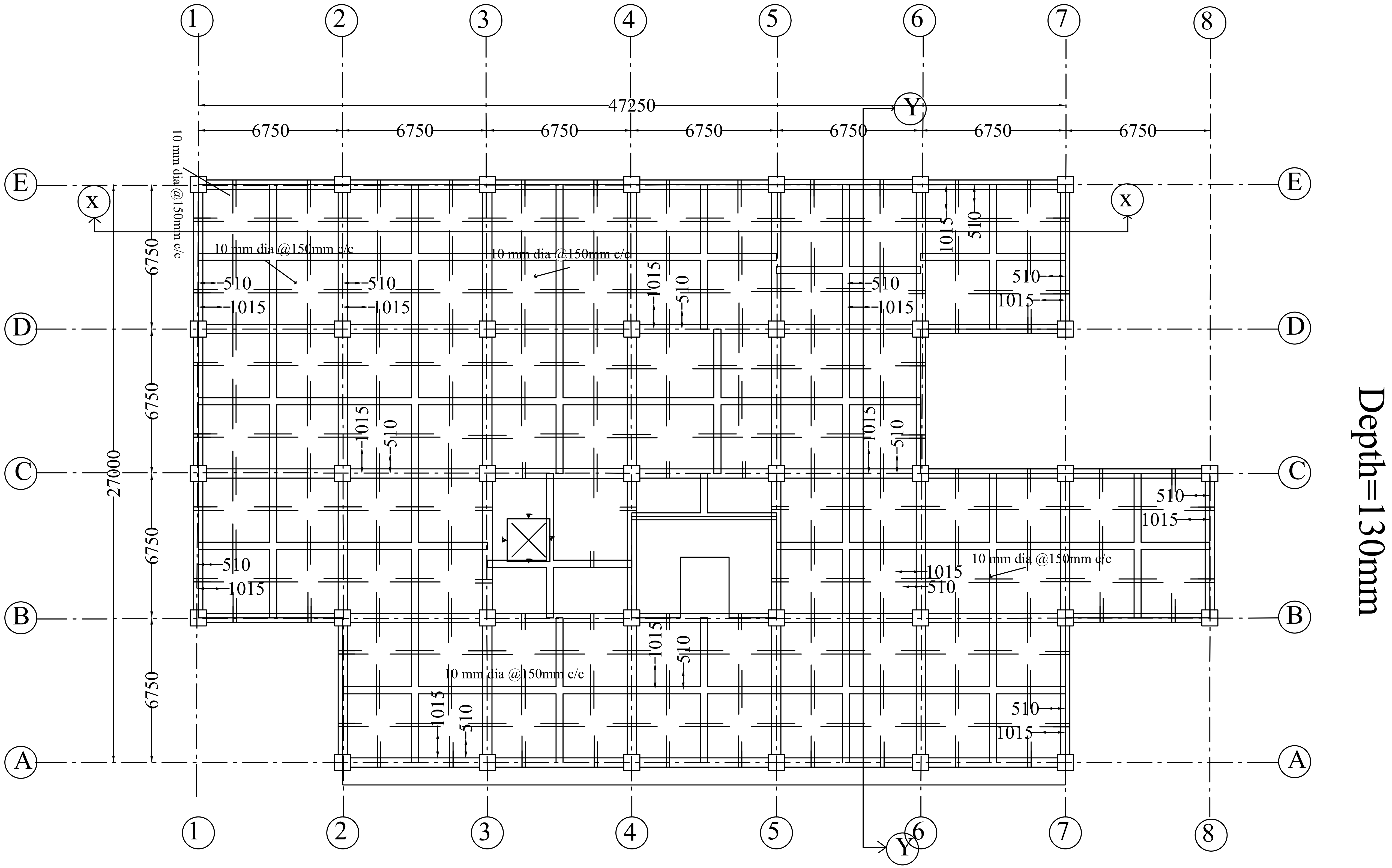
SLAB PANEL LAYOUT

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : SLAB PANEL LAYOUT	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. :8 Sheet No. :8



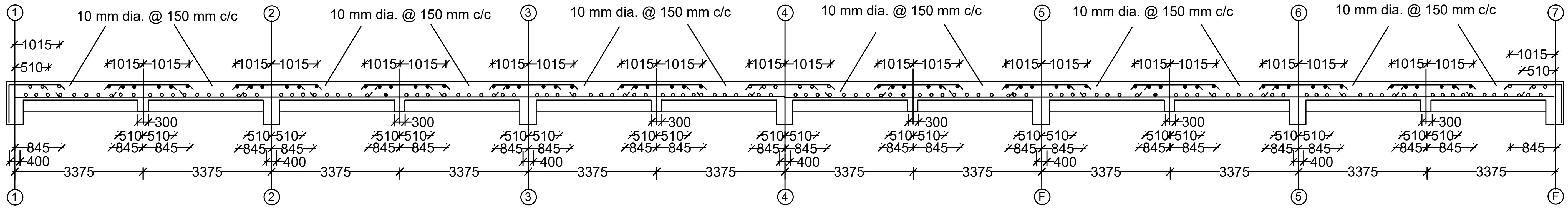
BOTTOM REINFORCEMENT OF SLAB OF BLOCK II

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : BOTTOM REINFORCEMENT OF SLAB OF BLOCK II	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to scale DWG No. :9 Sheet No. :9

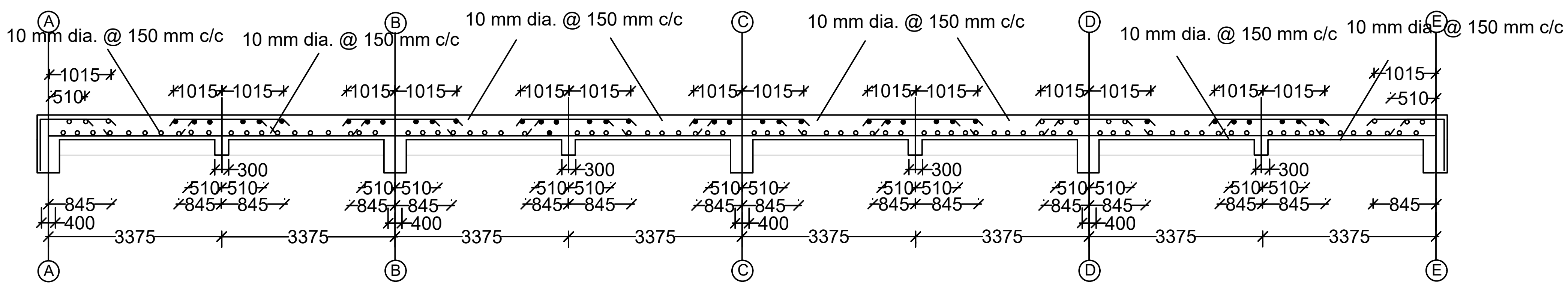


TOP REINFORCEMENT OF SLAB OF BLOCK II

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : TOP REINFORCEMENT PLAN OF SLAB OF BLOCK II	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. :10 Sheet No. :10



Section Along X-X



Section Along Y-Y

TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS

Project Title :

EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING

Sheet Title :

SLAB SECTION DETAILS

Group Members :

Dipak Dhakal	(075 BCE 051)
Kiran Kumar Maharjan	(075 BCE 075)
Kushal Sharma	(075 BCE 076)
Nimee Tiwari	(075 BCE 087)
Nishant Awasthi	(075 BCE 090)
Nishchal Nath Sigdel	(075 BCE 091)

Project Supervisor :

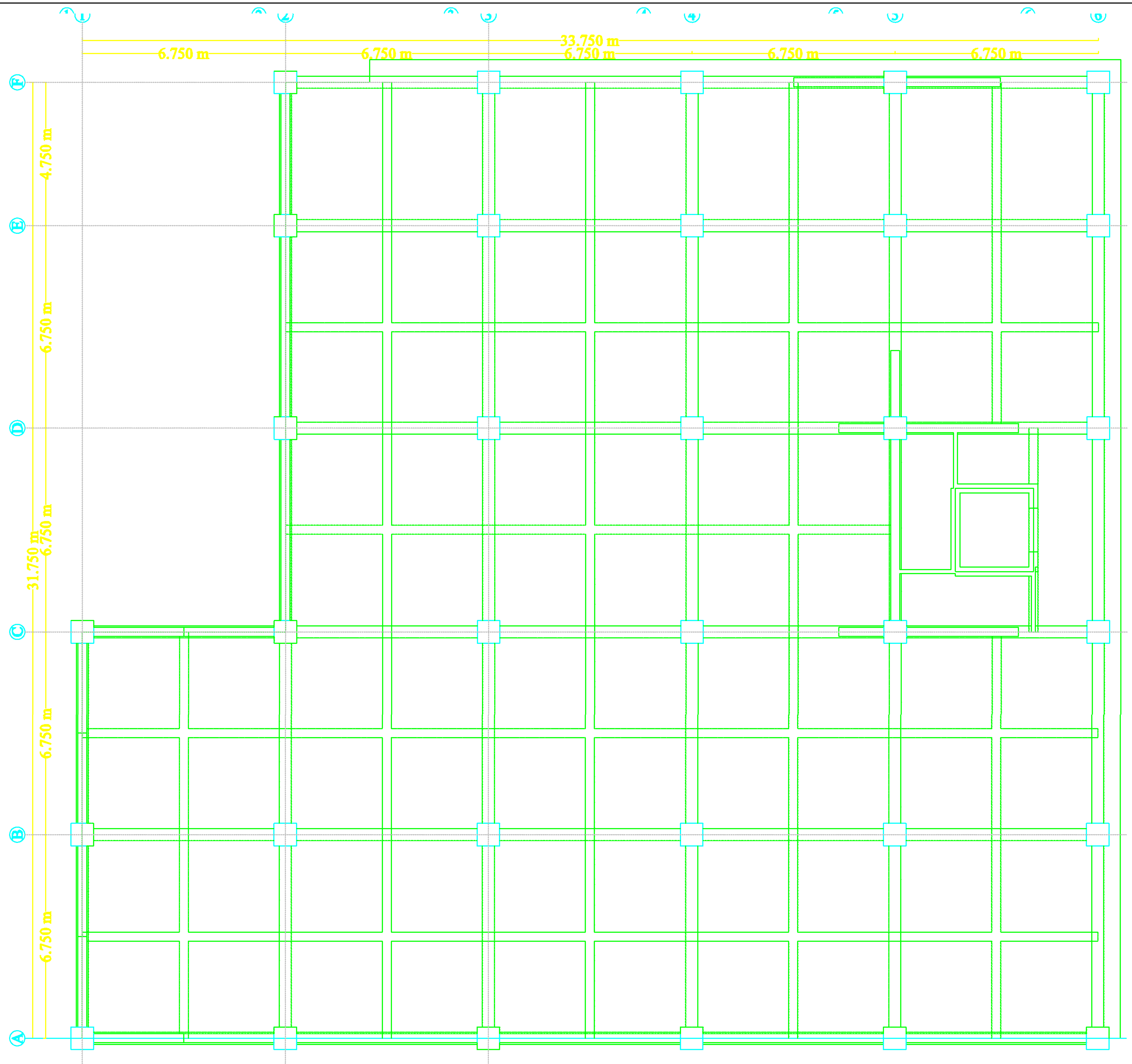
Asst. Prof. Arun Paudel

Date : 2080 / 01 / 14

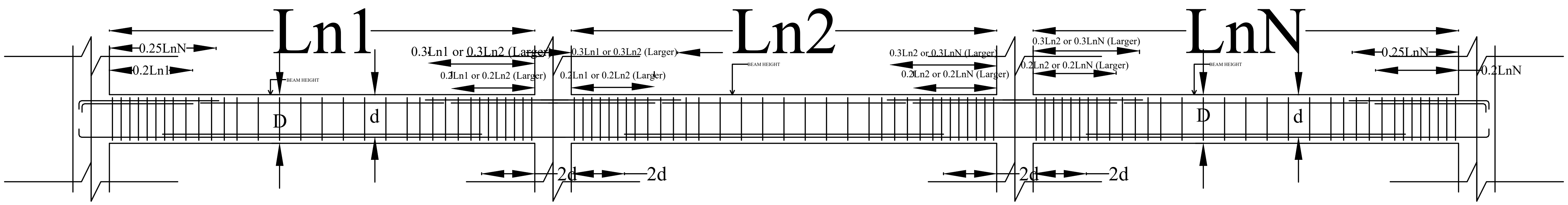
Fit to scale

DWG No. :11

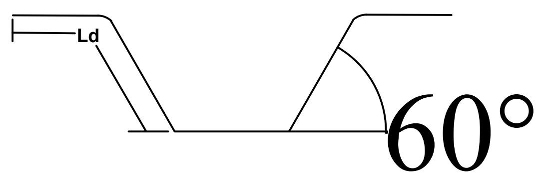
Sheet No. :11



TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : Concrete Beam Layout	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to Scale DWG No. :12 Sheet No. :12

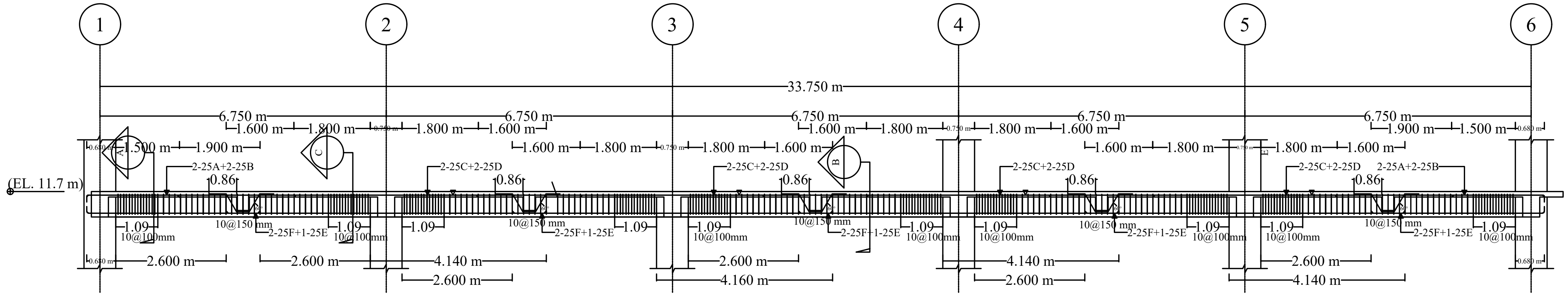


TYPICAL BEAM REINFORCEMENT DETAILING

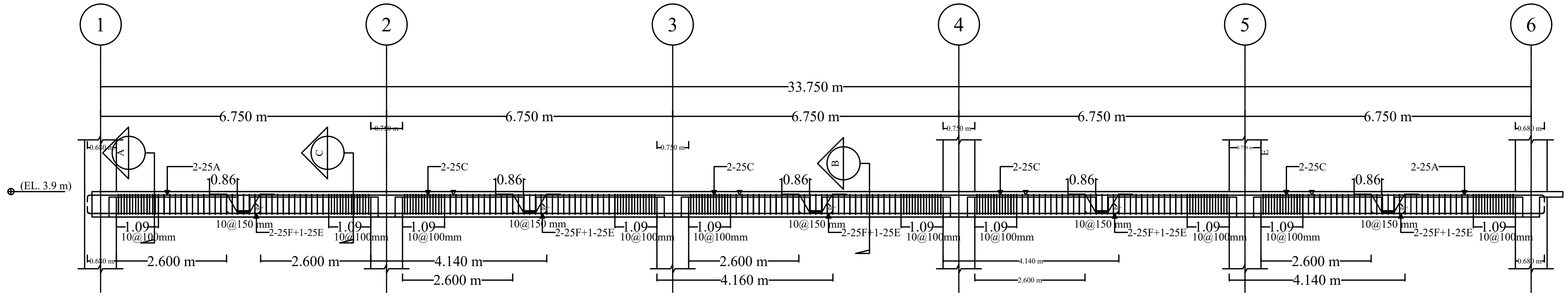


BENT UP HANGER TYPE BARS

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : TYPICAL BEAM REINFORCEMENT DETAILING	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to Scale DWG No. :13 Sheet No. :13



Longitudinal Section of Ground Floor (Grid B)



Longitudinal Section of First Floor (Grid B)

TRIBHUVAN UNIVERSITY
INSTITUTE OF
ENGINEERING
PULCHOWK CAMPUS

Project Title :

EARTHQUAKE RESISTANT
DESIGN OF MULTI-STORIED
HOSPITAL BUILDING

Sheet Title :

LONGITUDINAL
SECTION OF BEAM

Group Members :

Dipak Dhakal	(075 BCE 051)
Kiran Kumar Maharjan	(075 BCE 075)
Kushal Sharma	(075 BCE 076)
Nimee Tiwari	(075 BCE 087)
Nishant Awasthi	(075 BCE 090)
Nishchal Nath Sigdel	(075 BCE 091)

Project Supervisor :

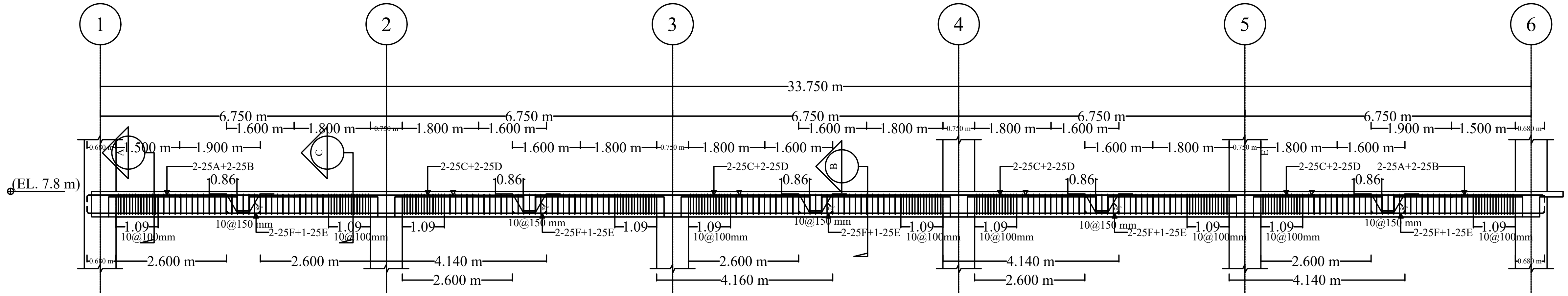
Asst. Prof. Arun Paudel

Date : 2080 / 01 / 14

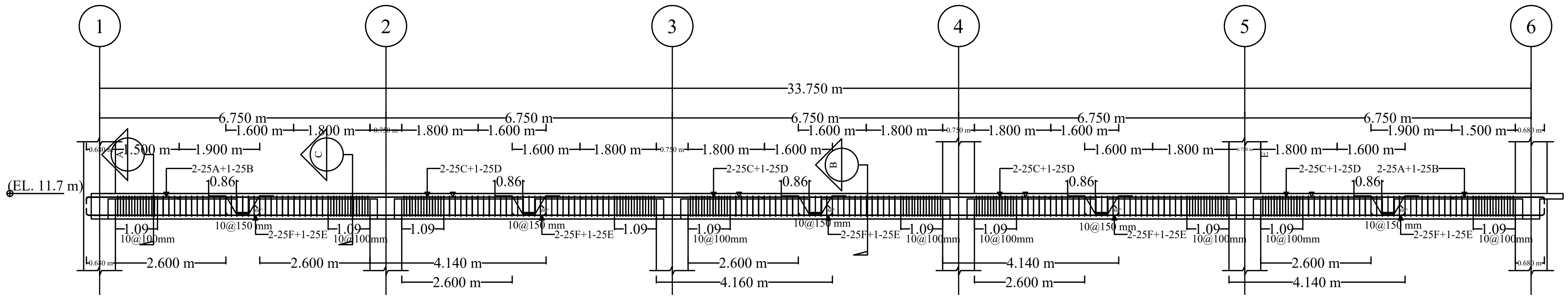
Scale : Fit to Scale

DWG No. :14

Sheet No. :14

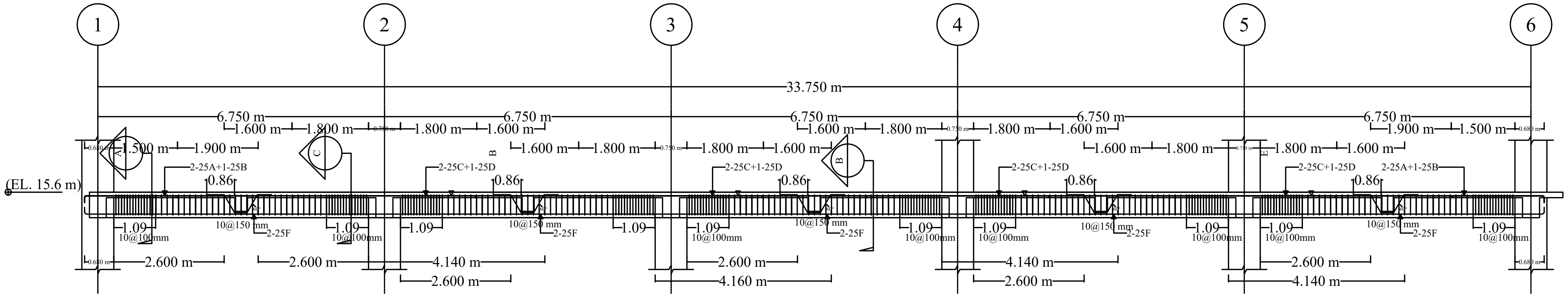


Longitudinal Section of Second Floor (Grid B)

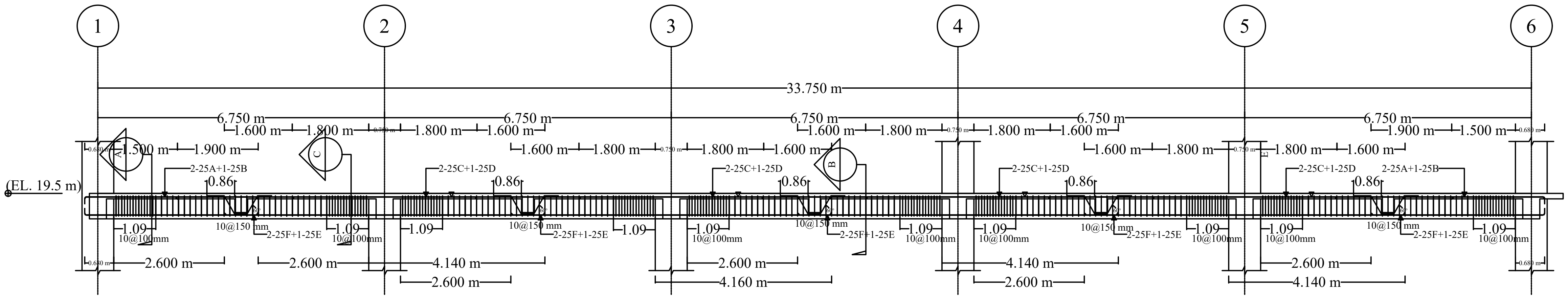


Longitudinal Section of Third Floor (Grid B)

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : LONGITUDINAL SECTTION OF BEAM	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to Scale DWG No. :15 Sheet No. :15

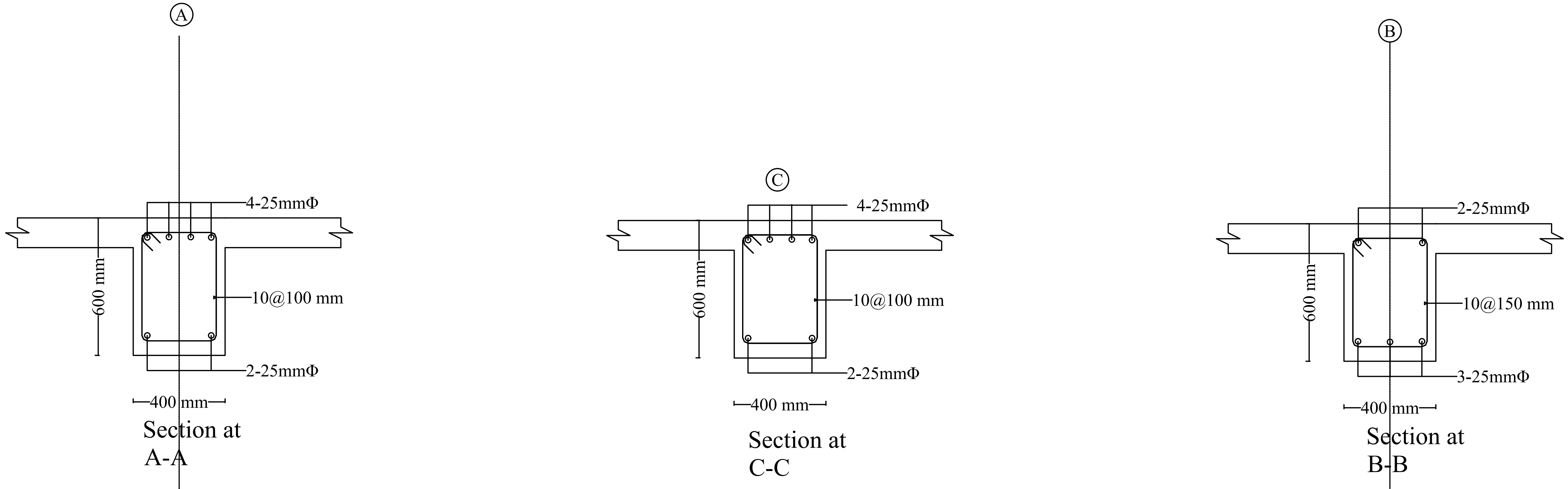


Longitudinal Section of Fourth Floor (Grid B)

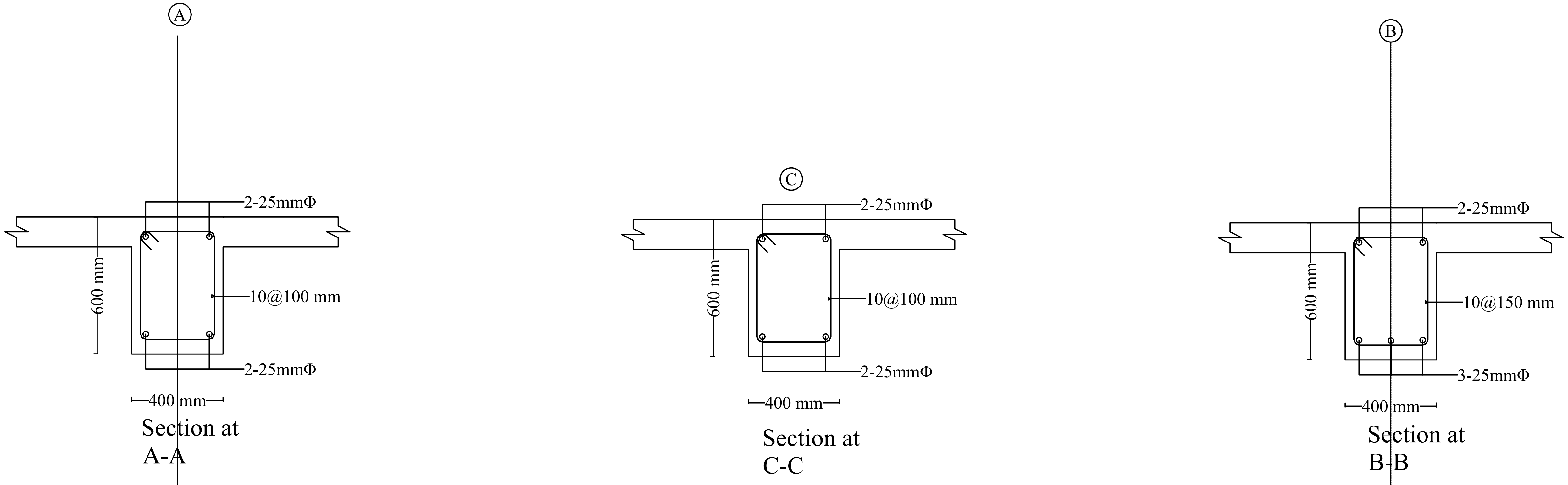


Longitudinal Section of Fifth Floor (Grid B)

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : LONGITUDINAL SECTION OF BEAM	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to Scale DWG No. :16 Sheet No. :16

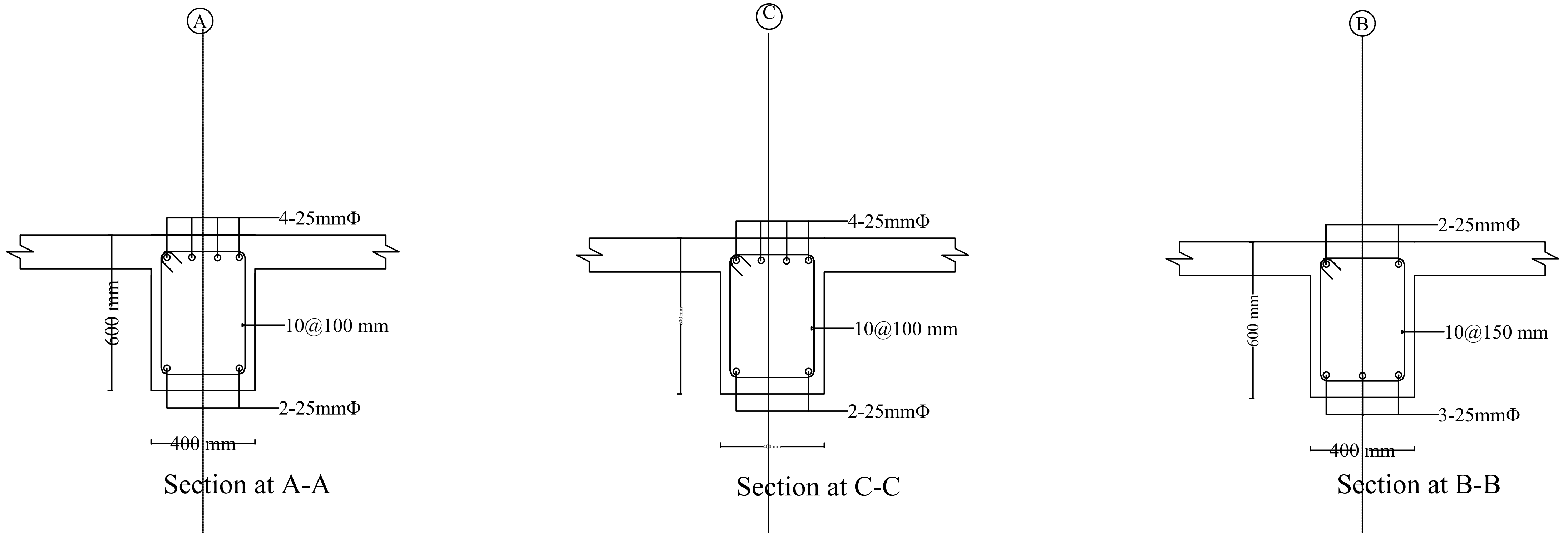


Cross Section of Ground Floor (Grid B)

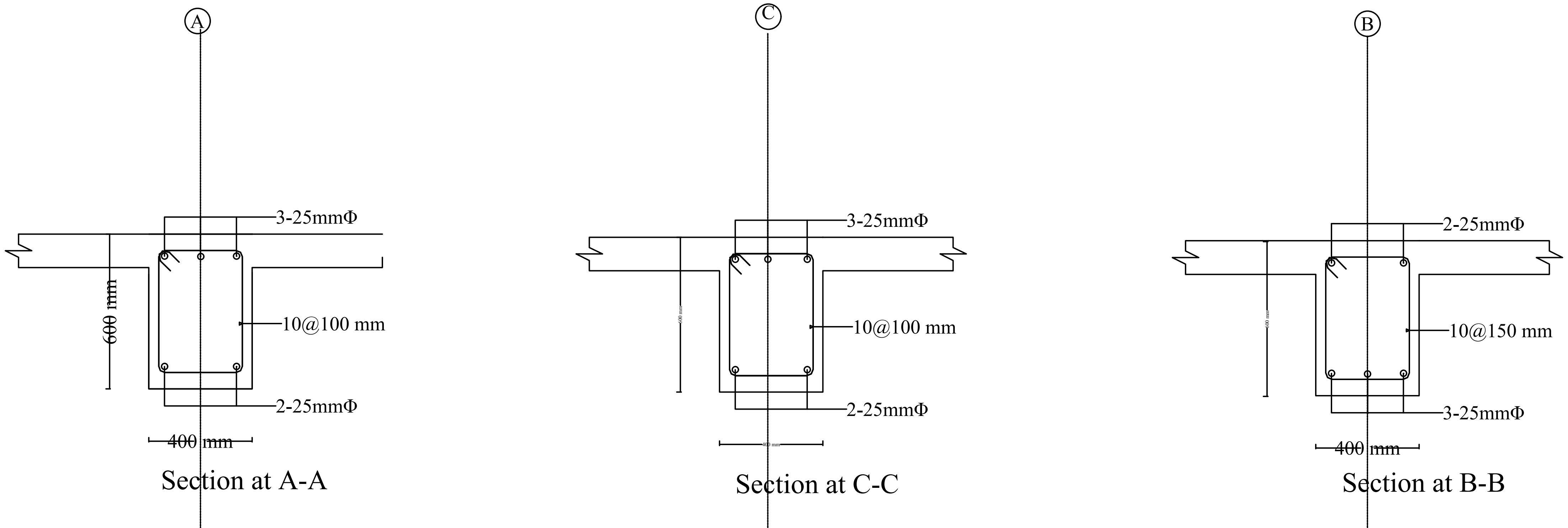


Cross Section of First Floor (Grid B)

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : CROSS SECTION OF BEAM	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to Scale DWG No. :17 Sheet No. :17

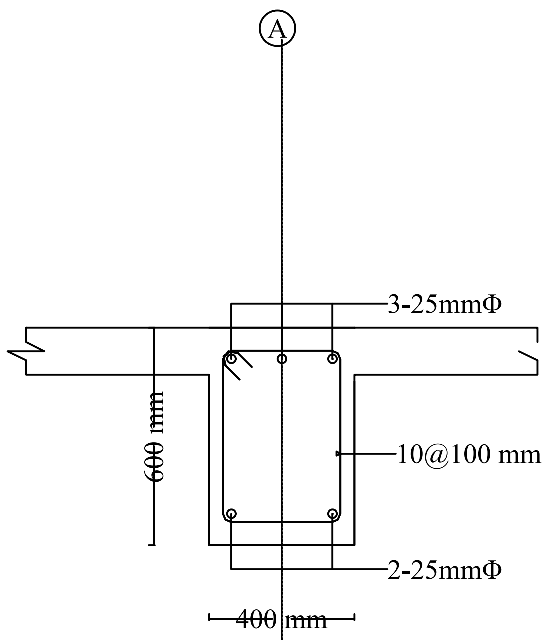


Cross Section of Second Floor (Grid B)

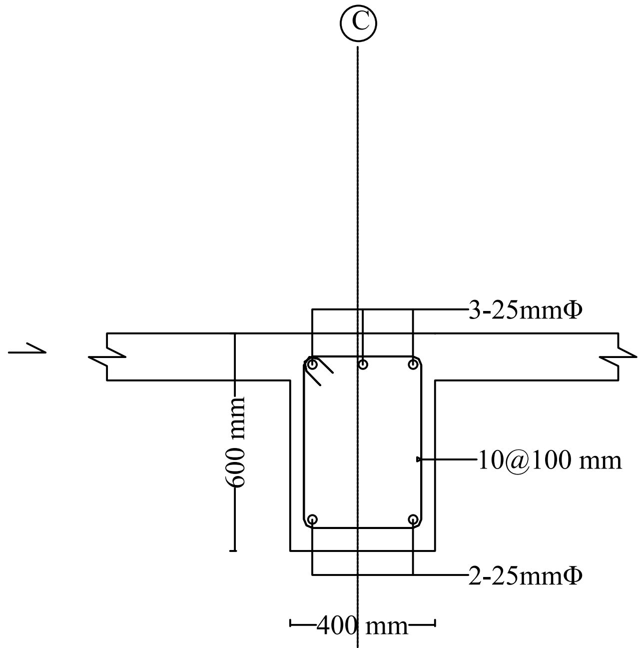


Cross Section of Third Floor (Grid B)

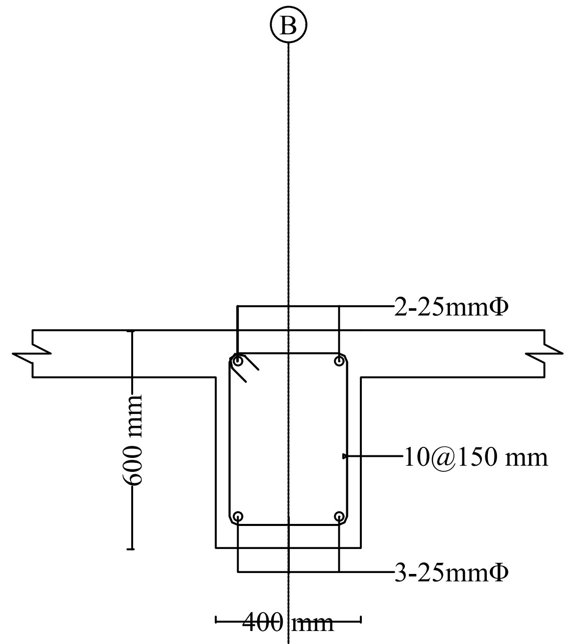
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : CROSS SECTION OF BEAM	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to Scale DWG No. :18 Sheet No. :18



Section at A-A

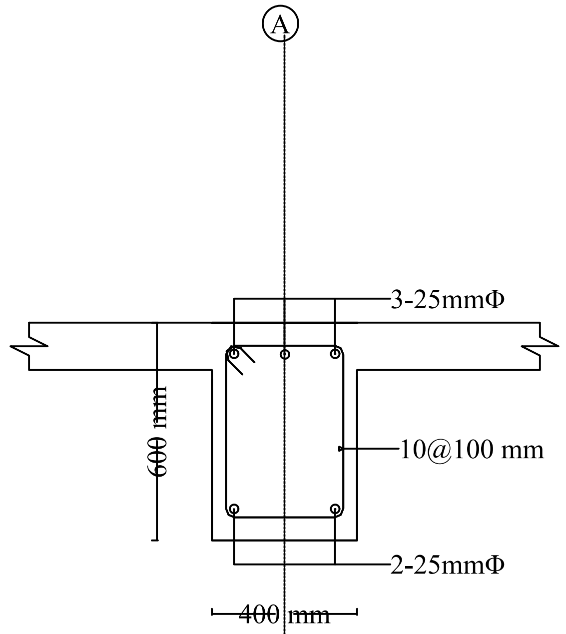


Section at C-C

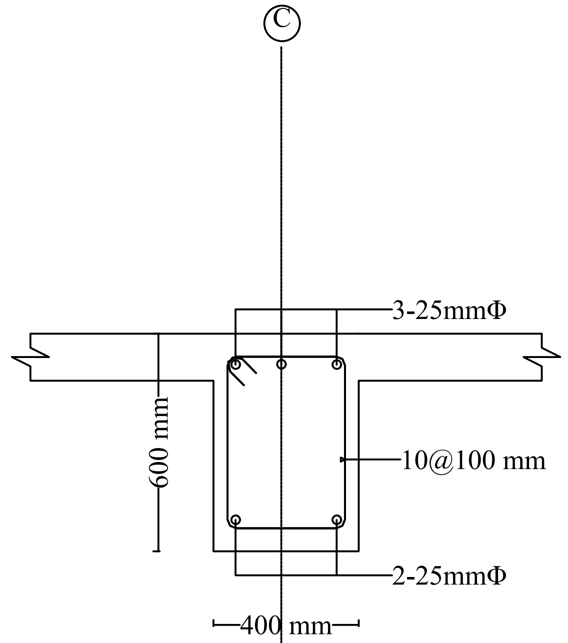


Section at B-B

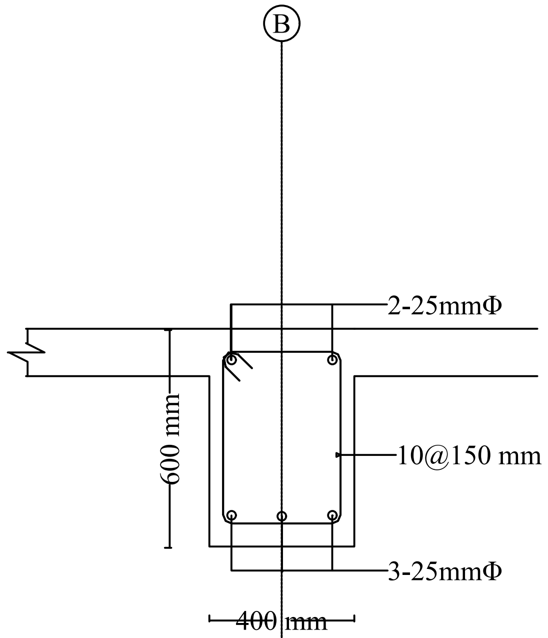
Cross Section of Fourth Floor (Grid B)



Section at A-A



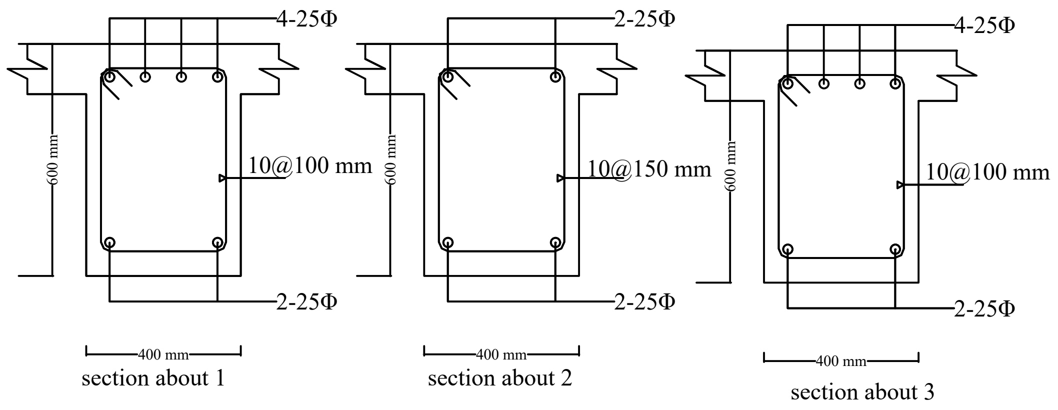
Section at C-C



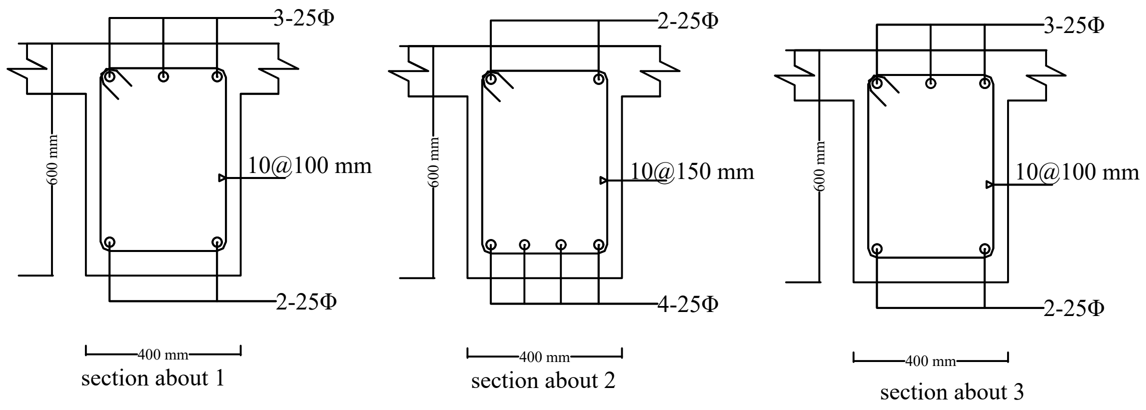
Section at B-B

Cross Section of Fifth Floor (Grid B)

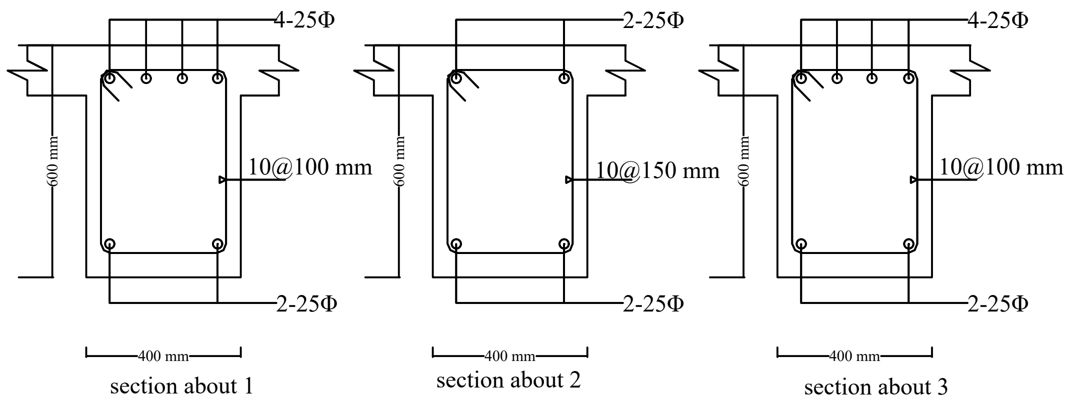
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : CROSS SECTION OF BEAM	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to Scale DWG No. :19 Sheet No. :19



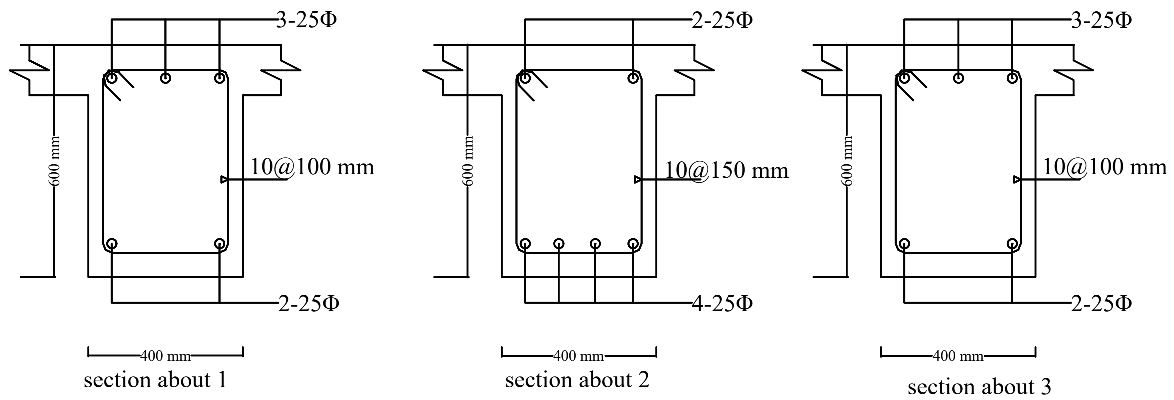
Transverse section of basement beam (grid 3)



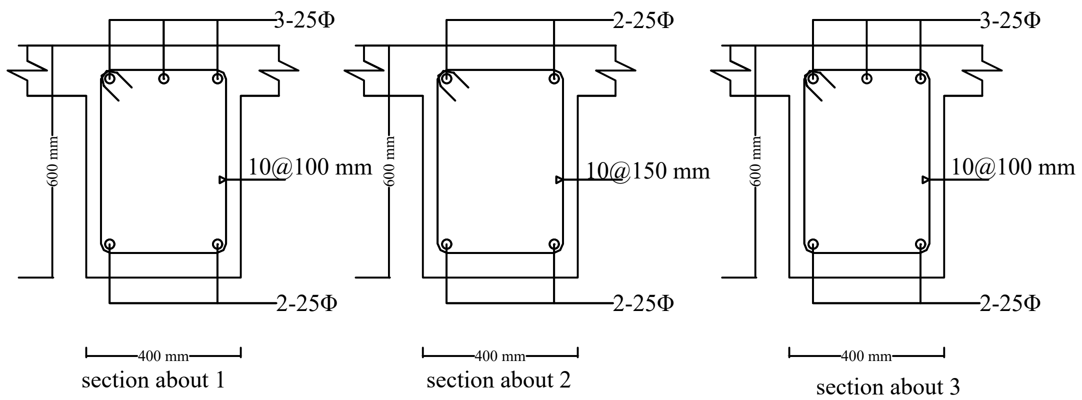
Transverse section of first floor beam (grid 3)



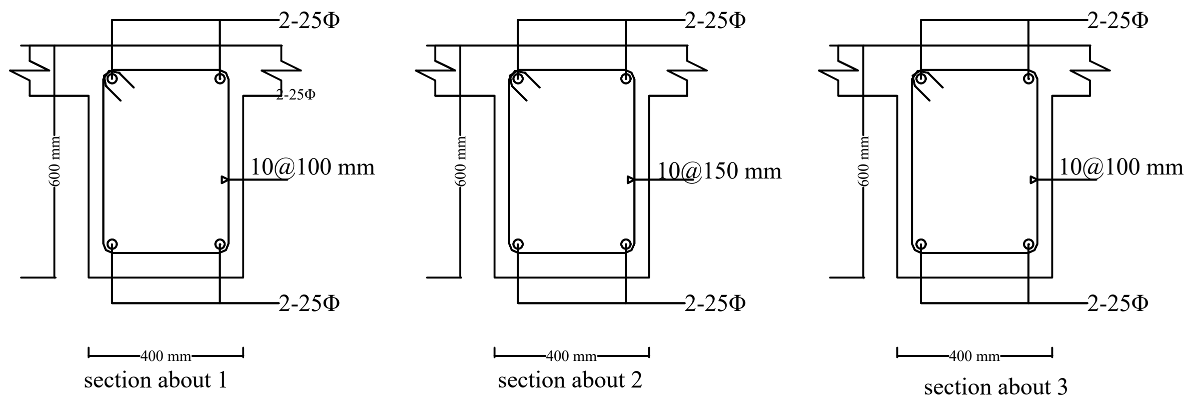
Transverse section of second floor beam (grid 3)



Transverse section of third beam (grid 3)

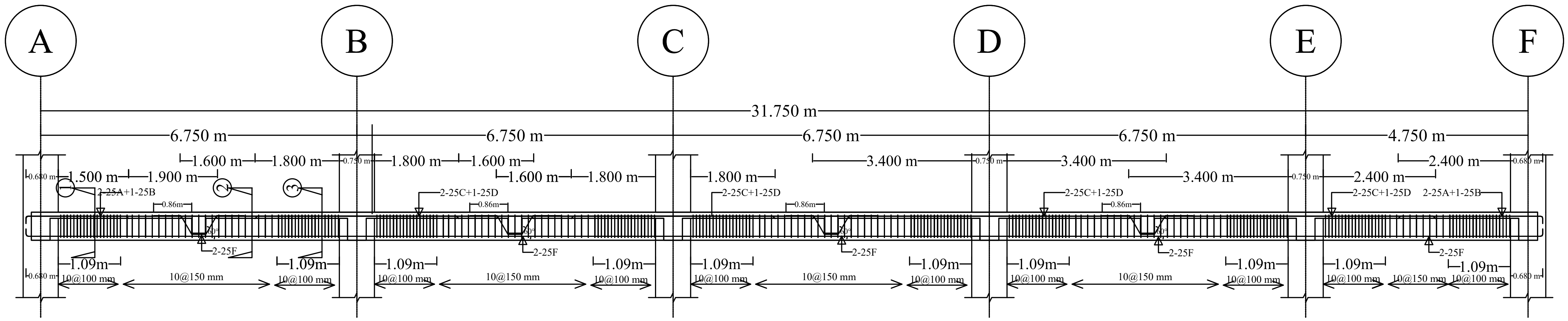


Transverse section of fourth floor beam (grid 3)

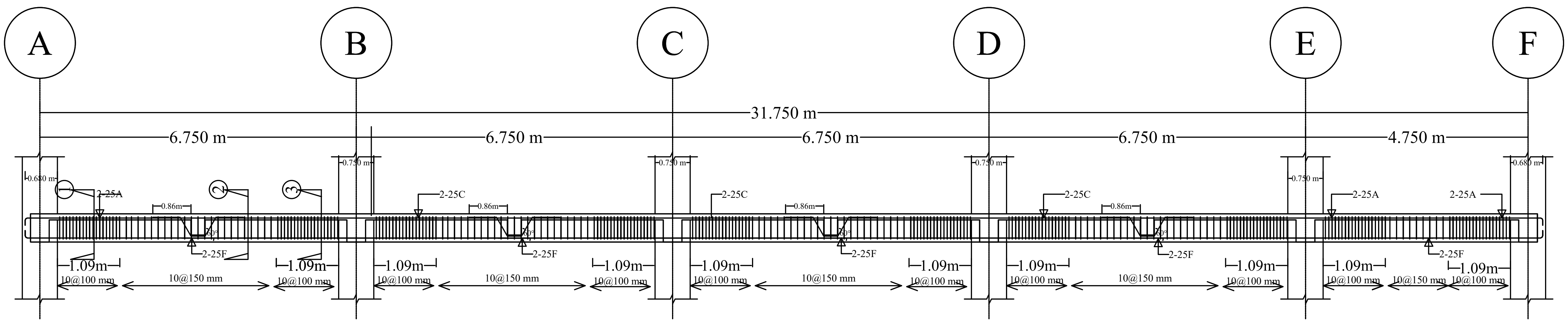


Transverse section of top beam (grid 3)

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : Detailing of Beam	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. :20 Sheet No. :20

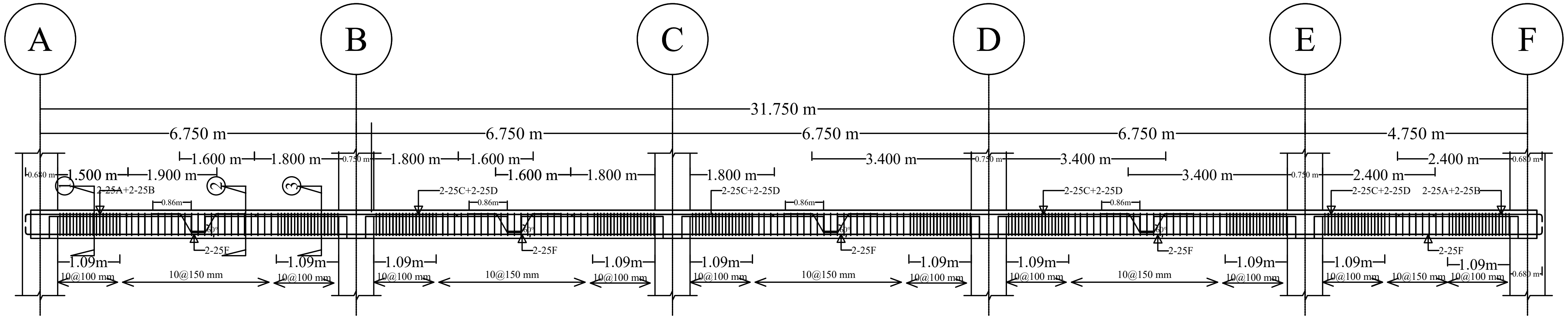


Longitudinal section of fourth floor grid 3

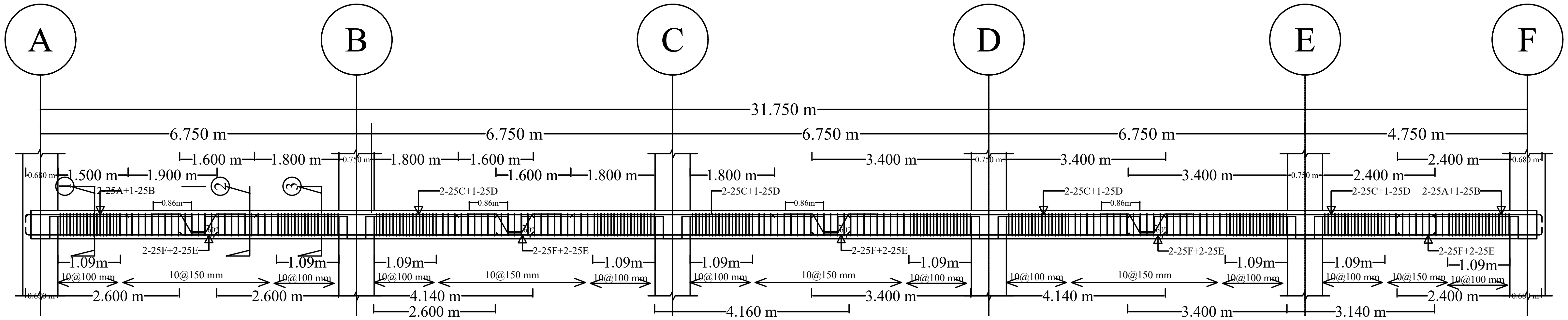


Longitudinal section of top floor grid 3

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : Detailing of Beam	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. :21 Sheet No. :21

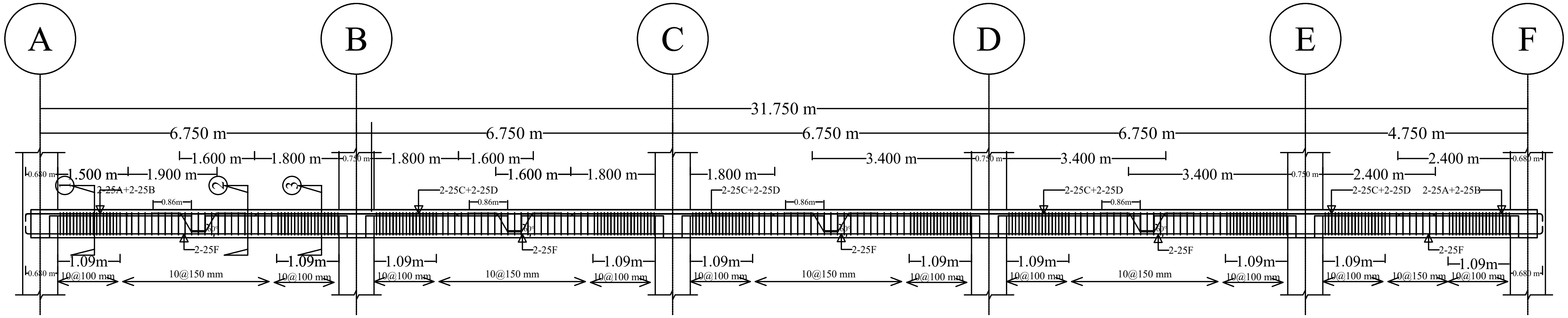


Longitudinal section of second floor grid 3

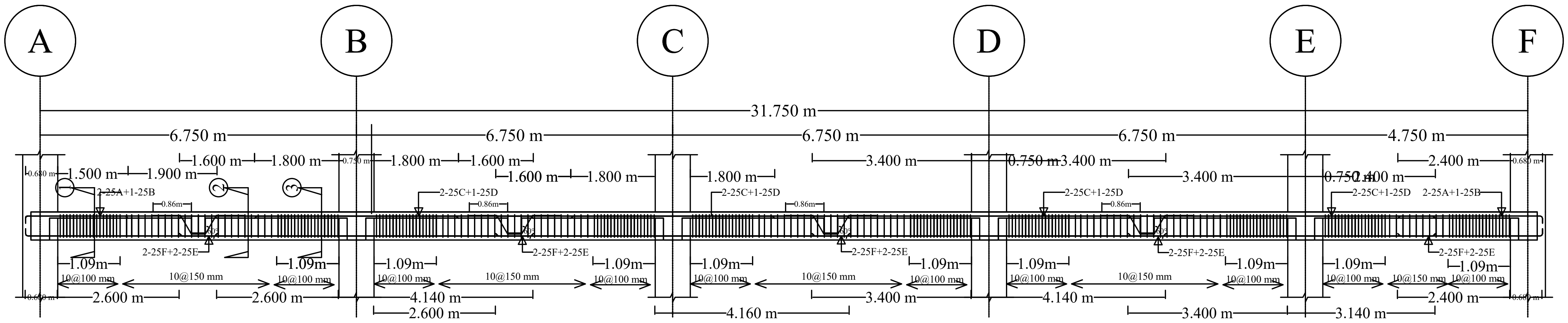


Longitudinal section of third floor grid 3

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : Detailing of Beam	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. :22 Sheet No. :22

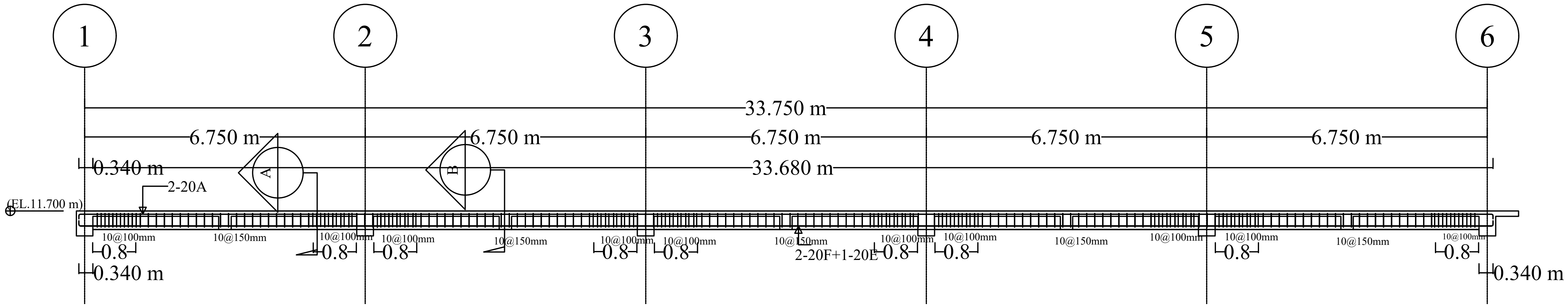


Longitudinal section of ground grid 3

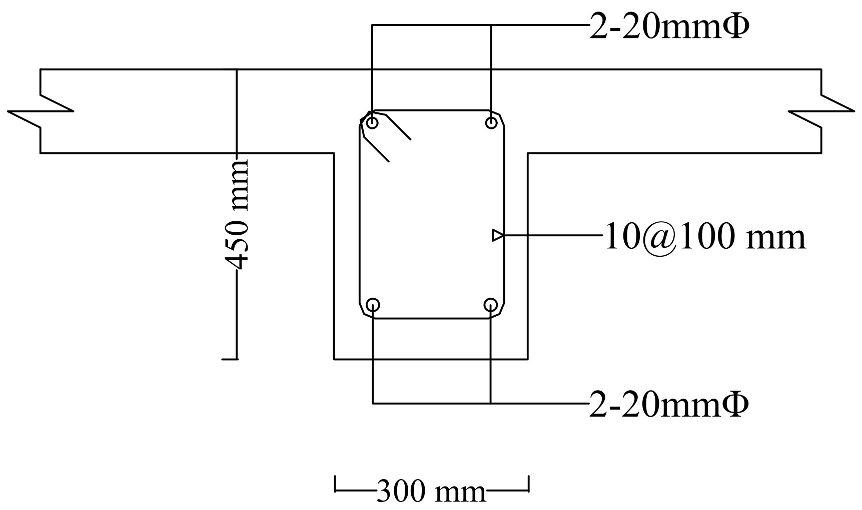


Longitudinal section of first floor grid 3

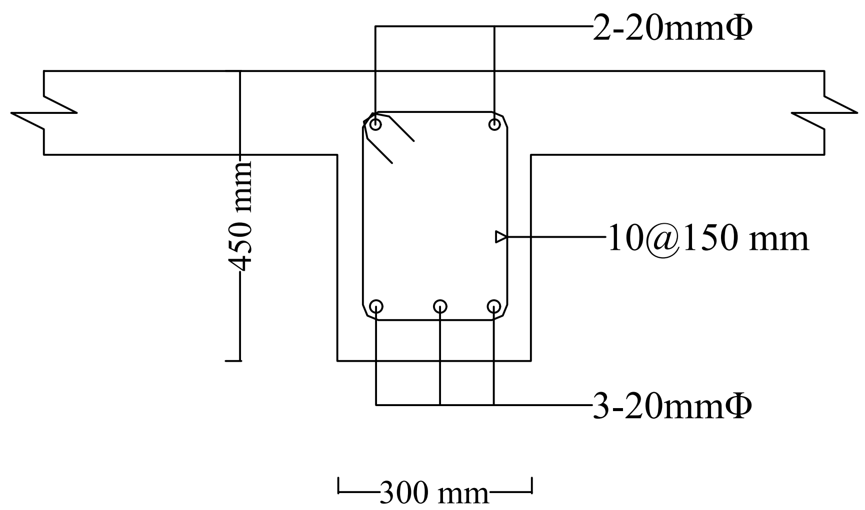
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : Detailing of Beam	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. :23 Sheet No. :23



Longitudinal Reinforcement of Secondary Beam Top Floor



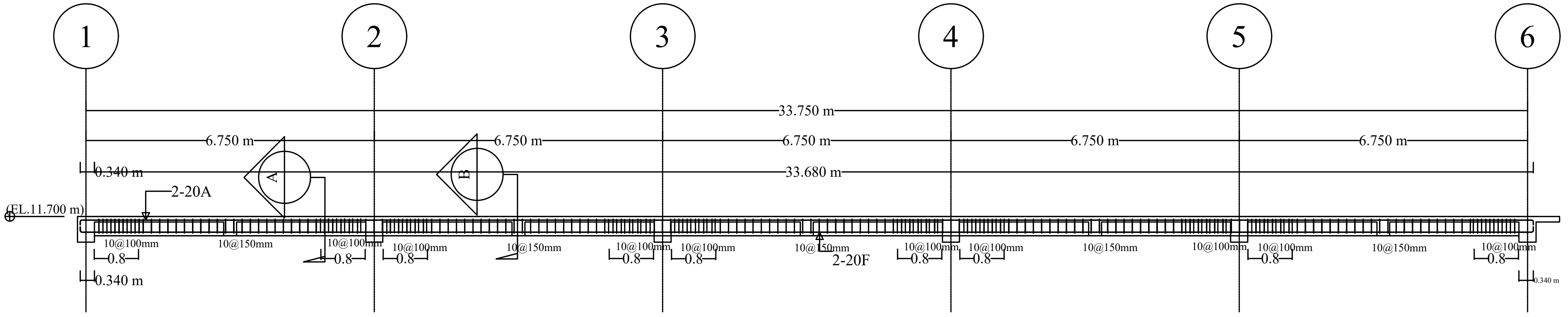
Section A-A



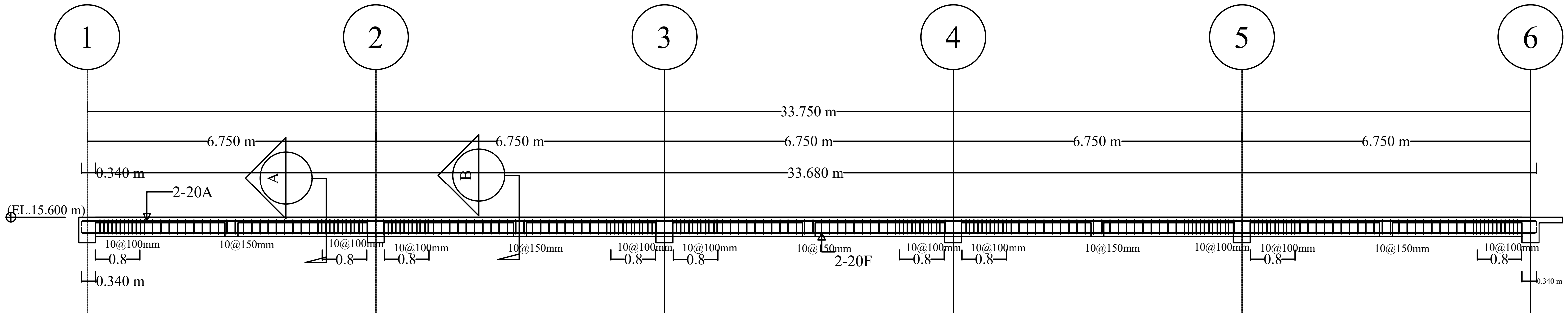
Section B-B

Cross Section of Secondary Beam Top Floor

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : LONGITUDINAL AND CROSS SECTION OF SECONDARY BEAM	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to Scale DWG No. :24 Sheet No. :24

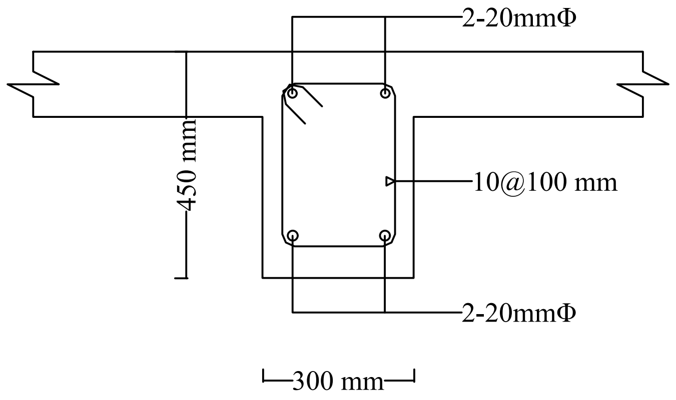


Longitudinal Reinforcement of Secondary Beam 3rd Floor

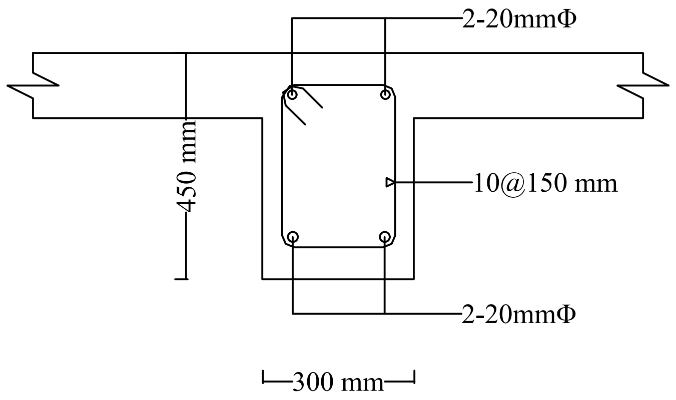


Longitudinal Reinforcement of Secondary Beam 4th Floor

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : LONGITUDINAL SECTION OF SECONDARY BEAM	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to Scale DWG No. :25 Sheet No. :25

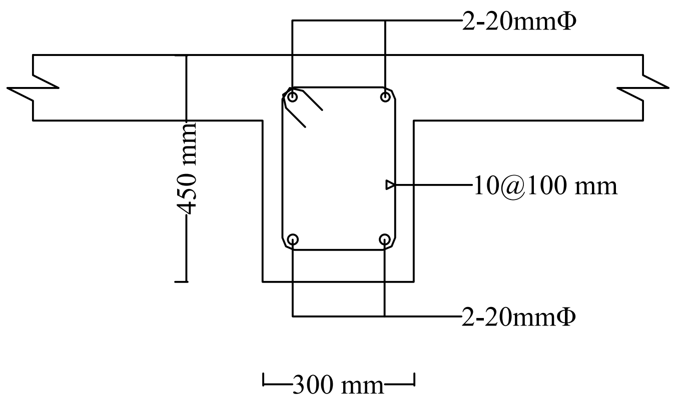


Section A-A

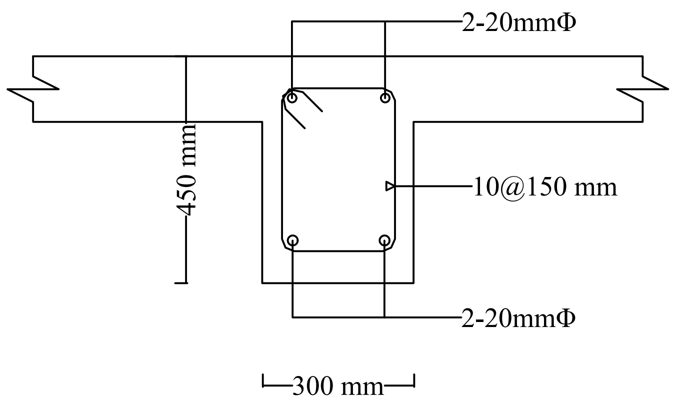


Section B-B

Cross Section of Secondary Beam (3rd Floor)



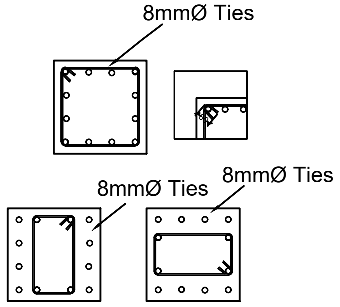
Section A-A



Section B-B

Cross Section of Secondary Beam (4th Floor)

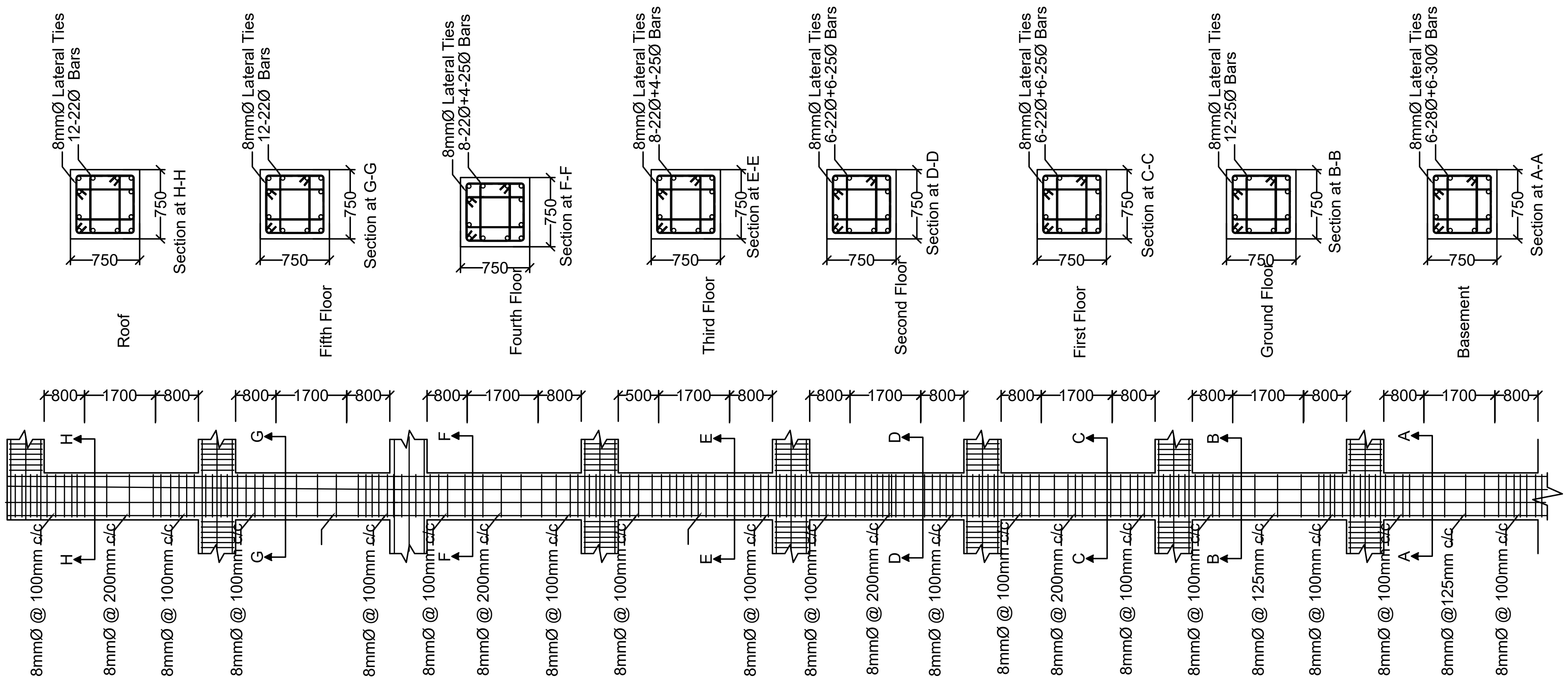
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : CROSS SECTION OF SECONDARY BEAM	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to Scale DWG No. :26 Sheet No. :26



Detailing of Lateral Ties

Column Rebar Schedule - A

Column Type	Storey	Reinforcement	Lateral Ties
A	Basement	6-28Ø+6-30Ø	8 mm
A	GF	12-25Ø	8 mm
A	1	6-22Ø+6-25Ø	8 mm
A	2	6-22Ø+6-25Ø	8 mm
A	3	8-22Ø+4-25Ø	8 mm
A	4	8-22Ø+4-25Ø	8 mm
A	Top	12-22Ø	8 mm
A	Roof	12-22Ø	8 mm

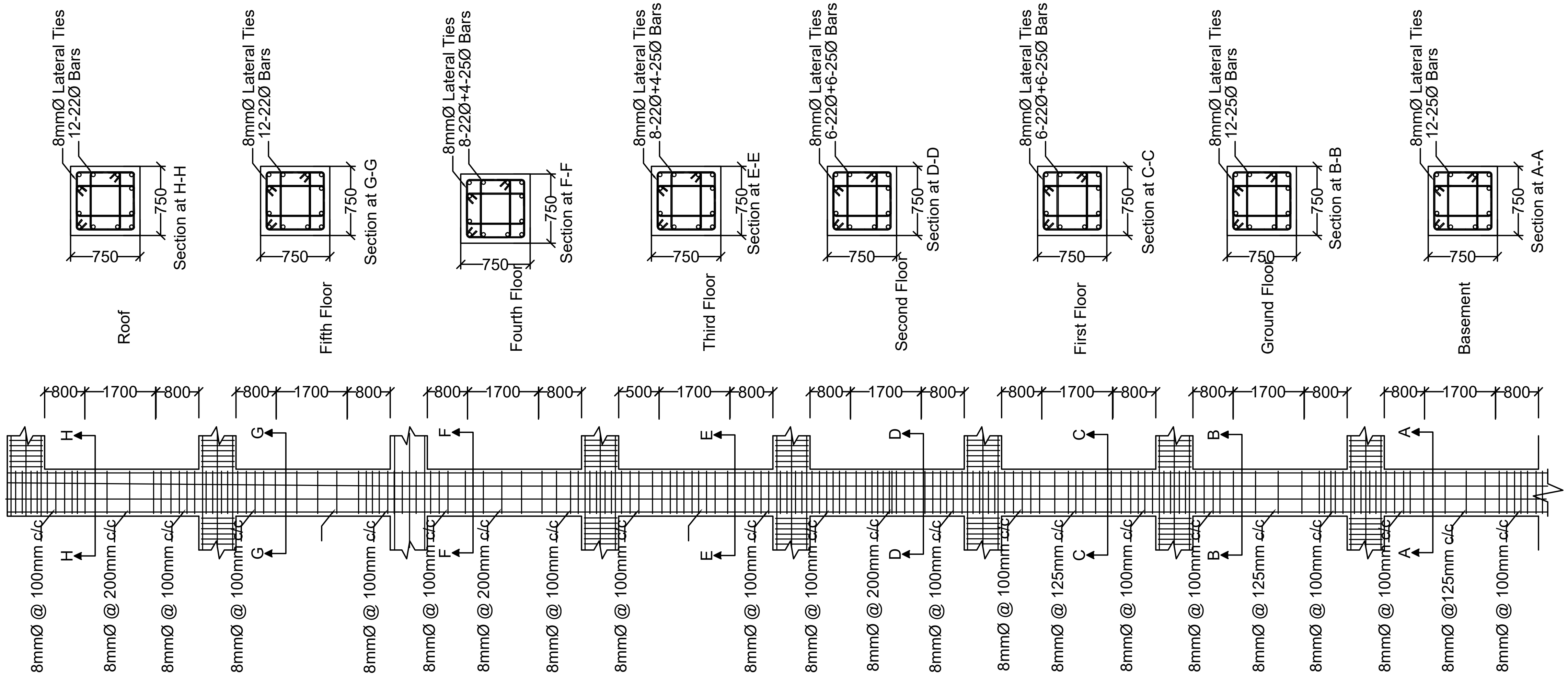


REINFORCEMENT DETAILING OF INTERIOR COLUMN OF BLOCK III (TYPE - A)

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : Detailing of Lift Shear Wall and Basement Wall	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to scale DWG No. : 27 Sheet No. : 27

Column Rebar
Schedule - A

Column Type	Storey	Reinforcement	Lateral Ties
A	Basement	12-25Ø	8 mm
A	GF	12-25Ø	8 mm
A	1	6-22Ø+6-25Ø	8 mm
A	2	6-22Ø+6-25Ø	8 mm
A	3	8-22Ø+4-25Ø	8 mm
A	4	8-22Ø+4-25Ø	8 mm
A	Top	12-22Ø	8 mm
A	Roof	12-22Ø	8 mm

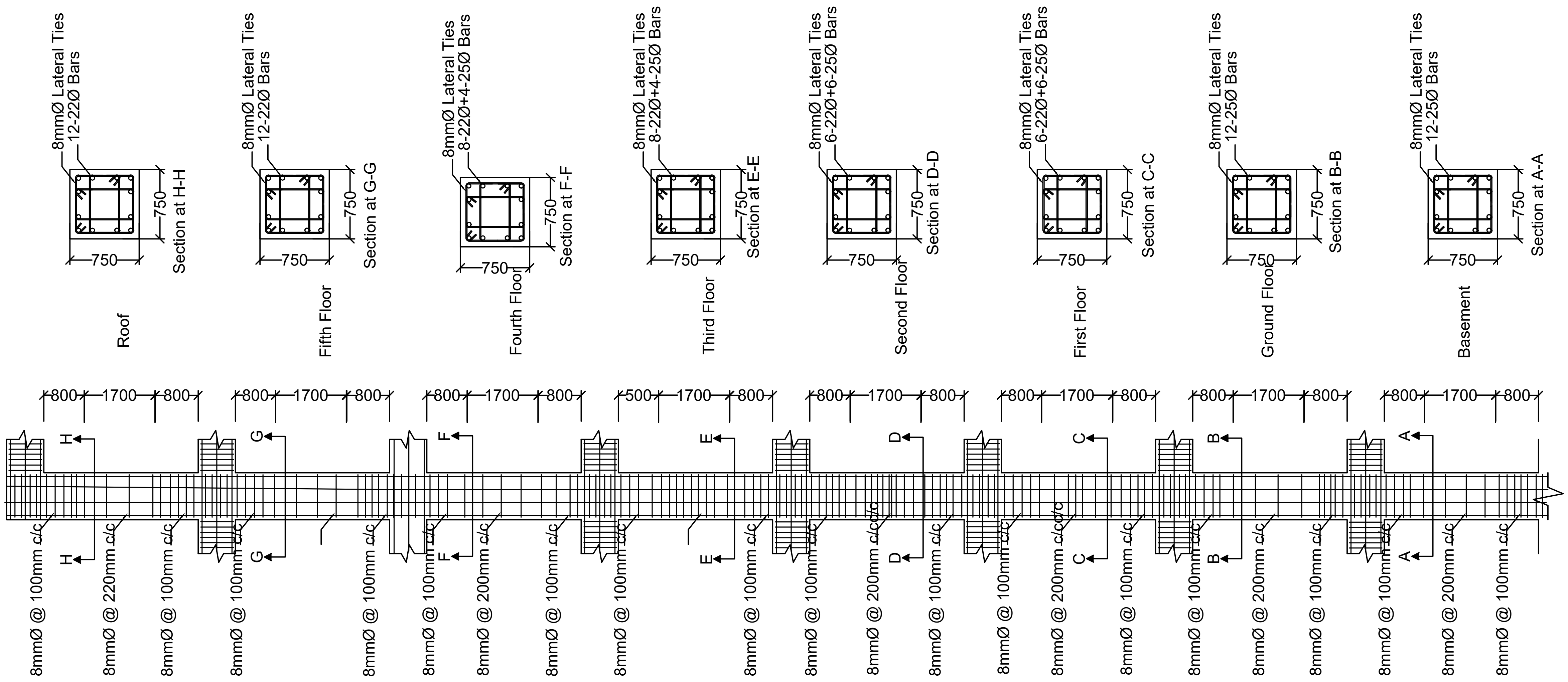


REINFORCEMENT DETAILING OF CORNER COLUMN OF BLOCK I (TYPE - B)

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : Detailing of Lift Shear Wall and Basement Wall	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to scale DWG No. : 28 Sheet No. : 28

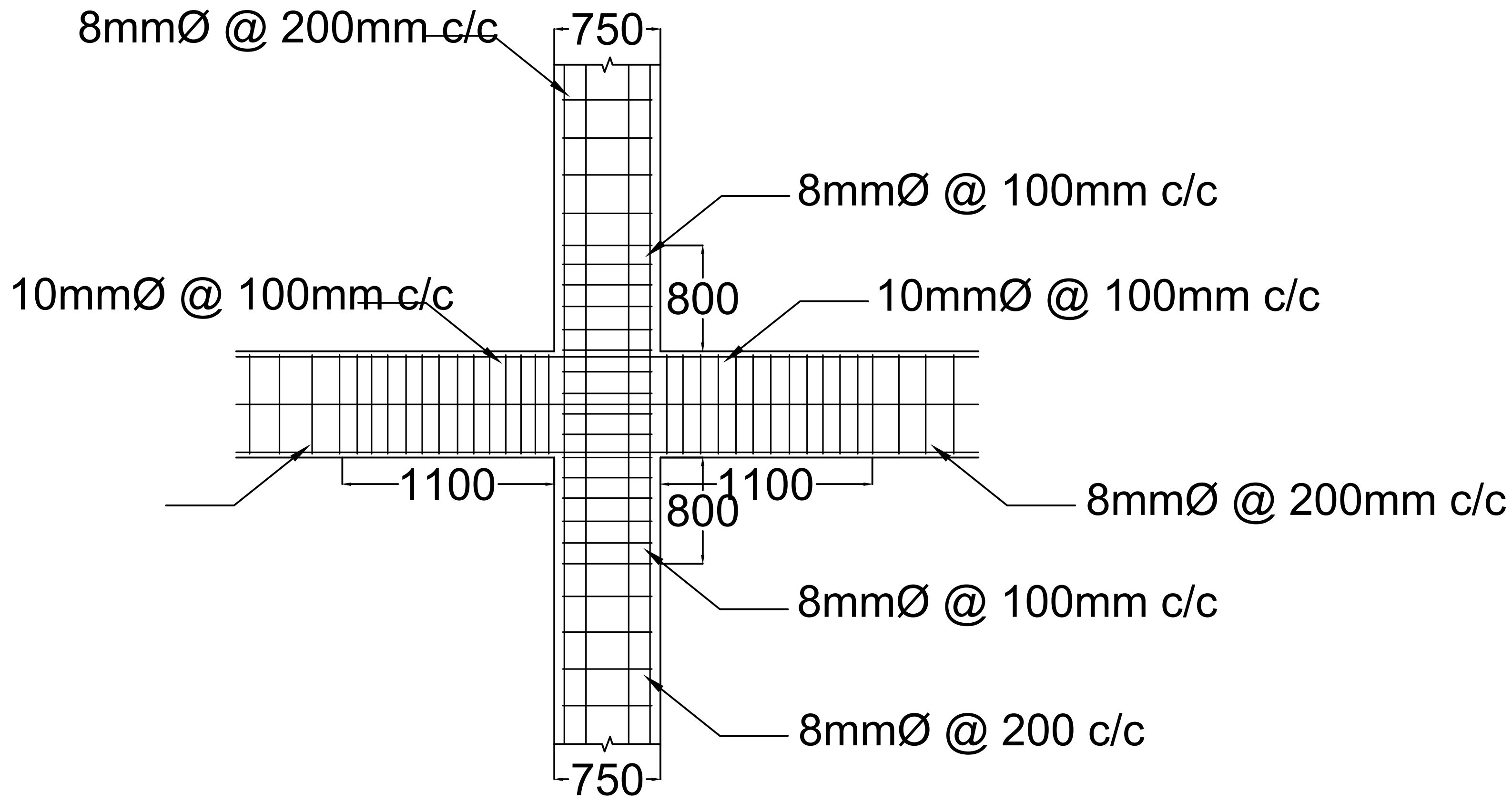
Column Rebar
Schedule - A

Column Type	Storey	Reinforcement	Lateral Ties
A	Basement	12-25Ø	8 mm
A	GF	12-25Ø	8 mm
A	1	6-22Ø+6-25Ø	8 mm
A	2	6-22Ø+6-25Ø	8 mm
A	3	8-22Ø+4-25Ø	8 mm
A	4	8-22Ø+4-25Ø	8 mm
A	Top	12-22Ø	8 mm
A	Roof	12-22Ø	8 mm

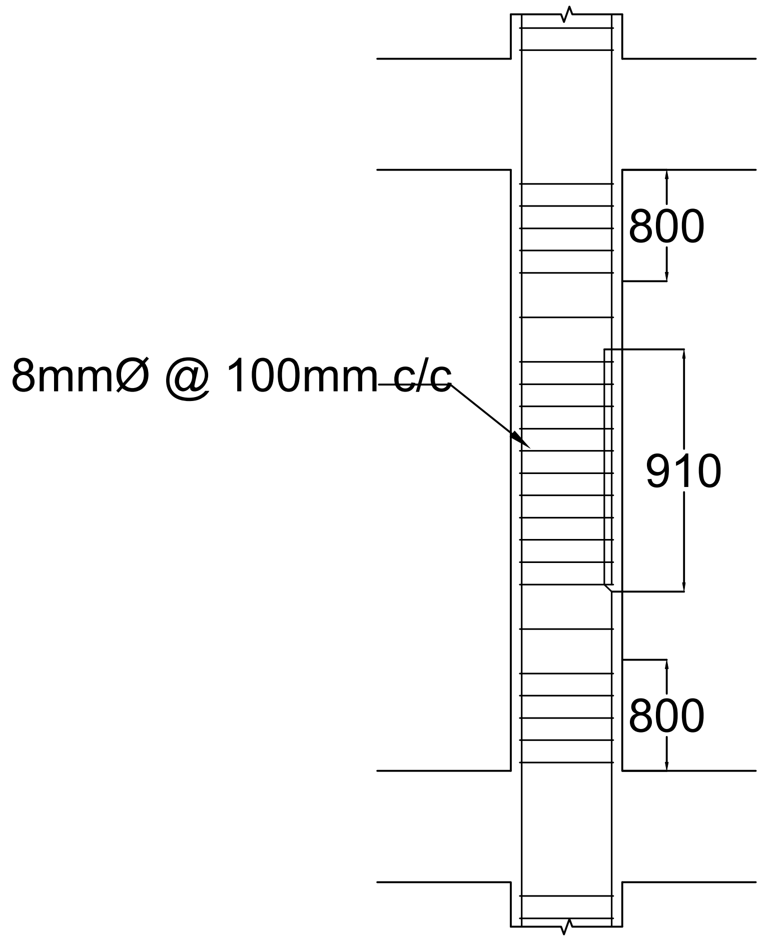


REINFORCEMENT DETAILING OF FACE COLUMN OF BLOCK IV (TYPE - C)

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : Detailing of Lift Shear Wall and Basement Wall	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to scale DWG No. : 29 Sheet No. : 29



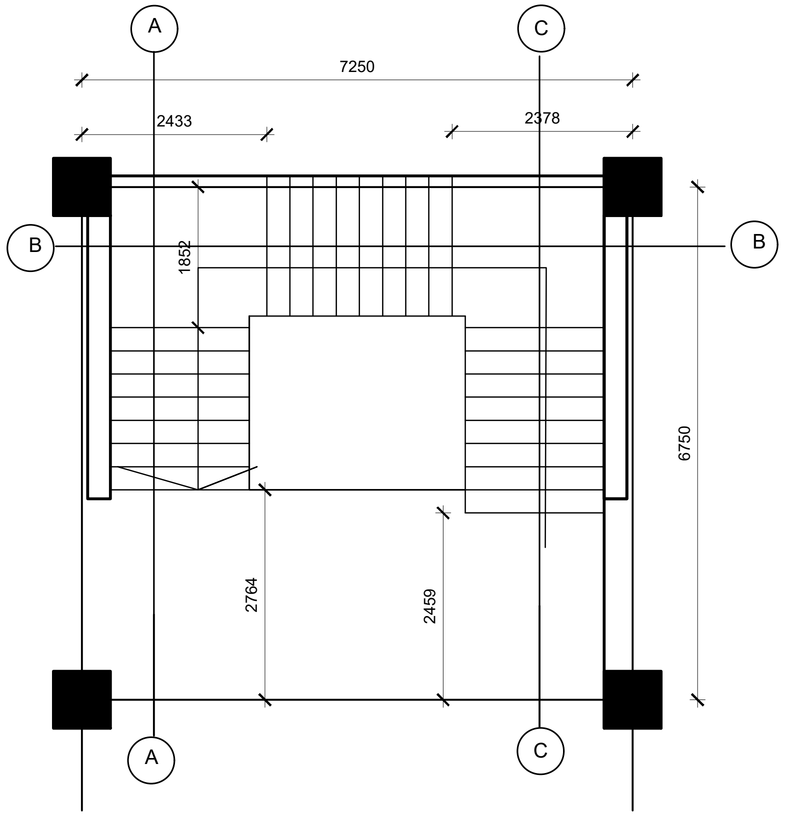
BEAM-COLUMN JOINT DETAILS



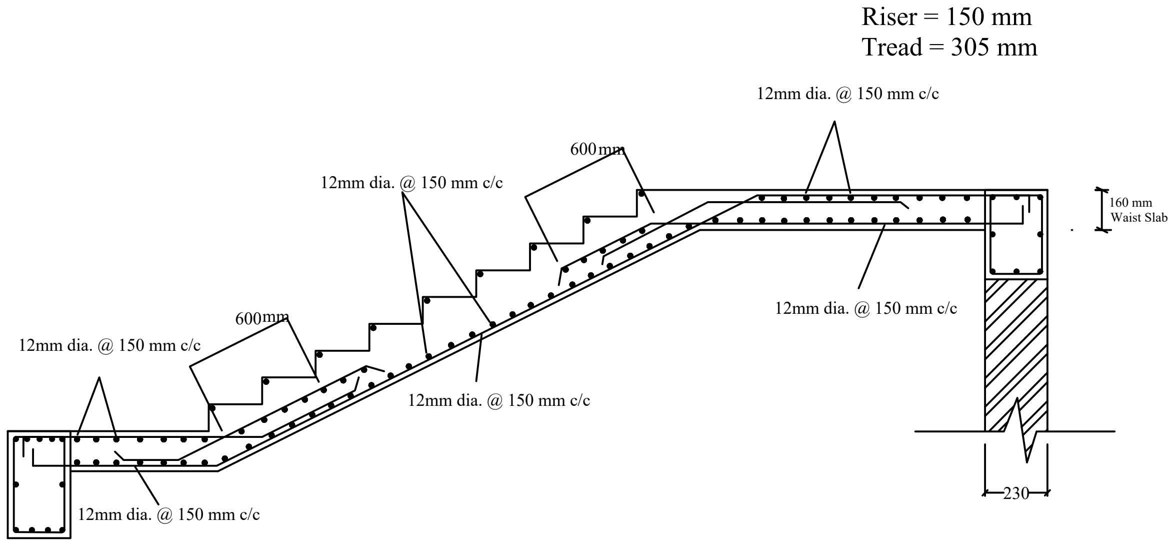
COLUMN (TYPE - A) - (25mm Dia.)

COLUMN SPLICING DETAILS

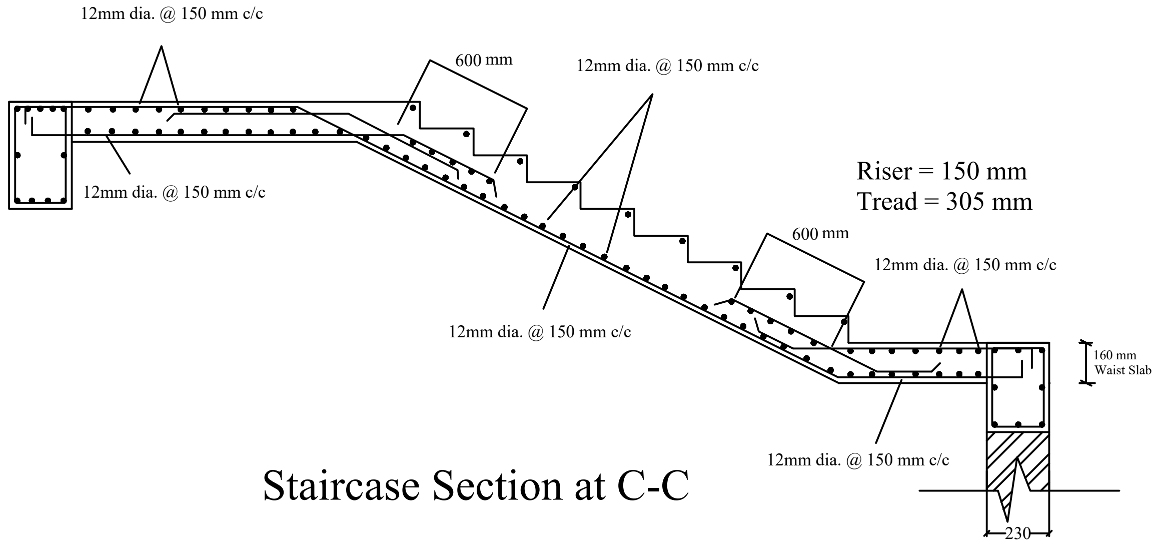
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : Detailing of Lift Shear Wall and Basement Wall	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. : 30 Sheet No. : 30



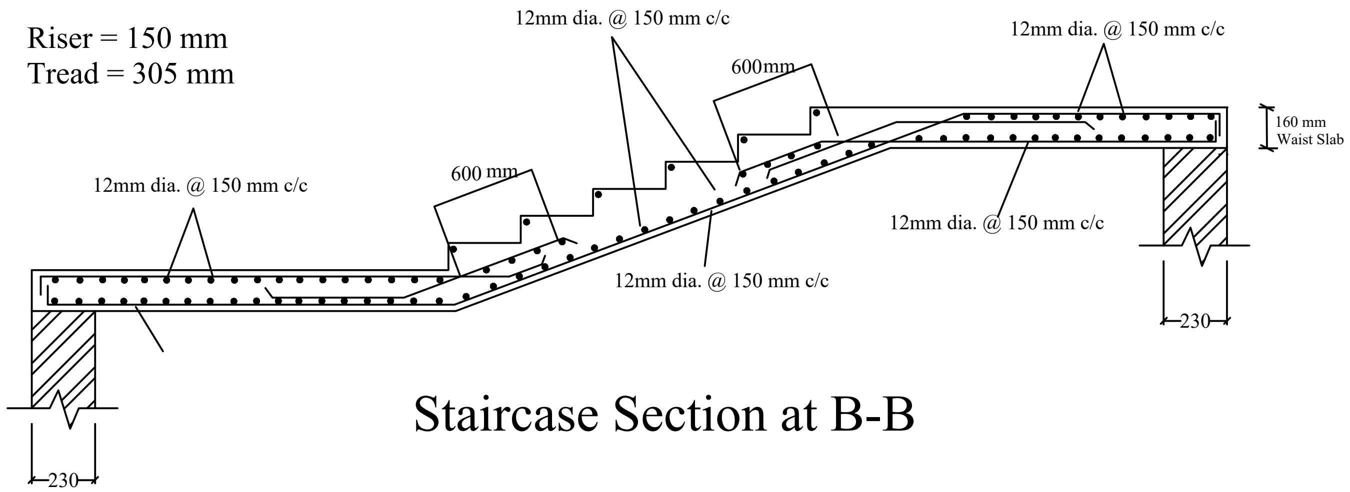
STAIRCASE PLAN



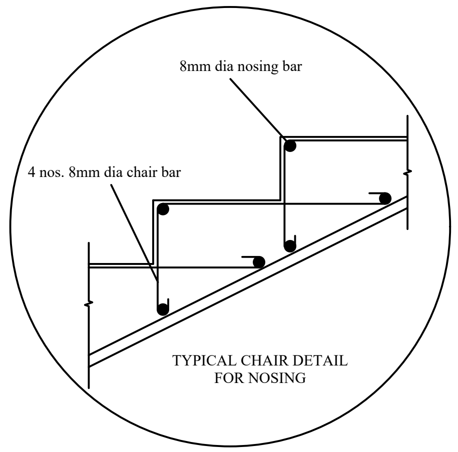
Staircase Section at A-A



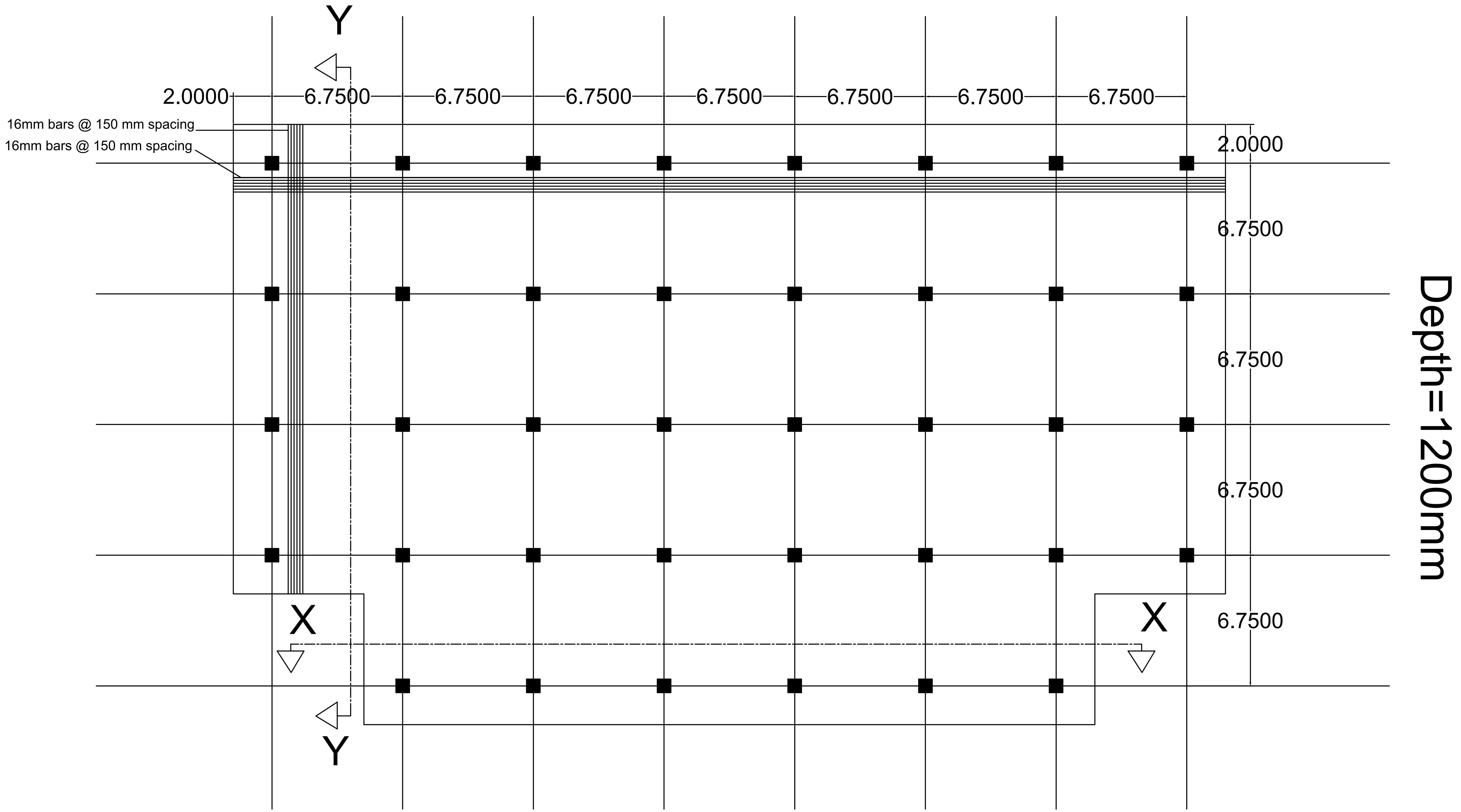
Staircase Section at C-C



Staircase Section at B-B

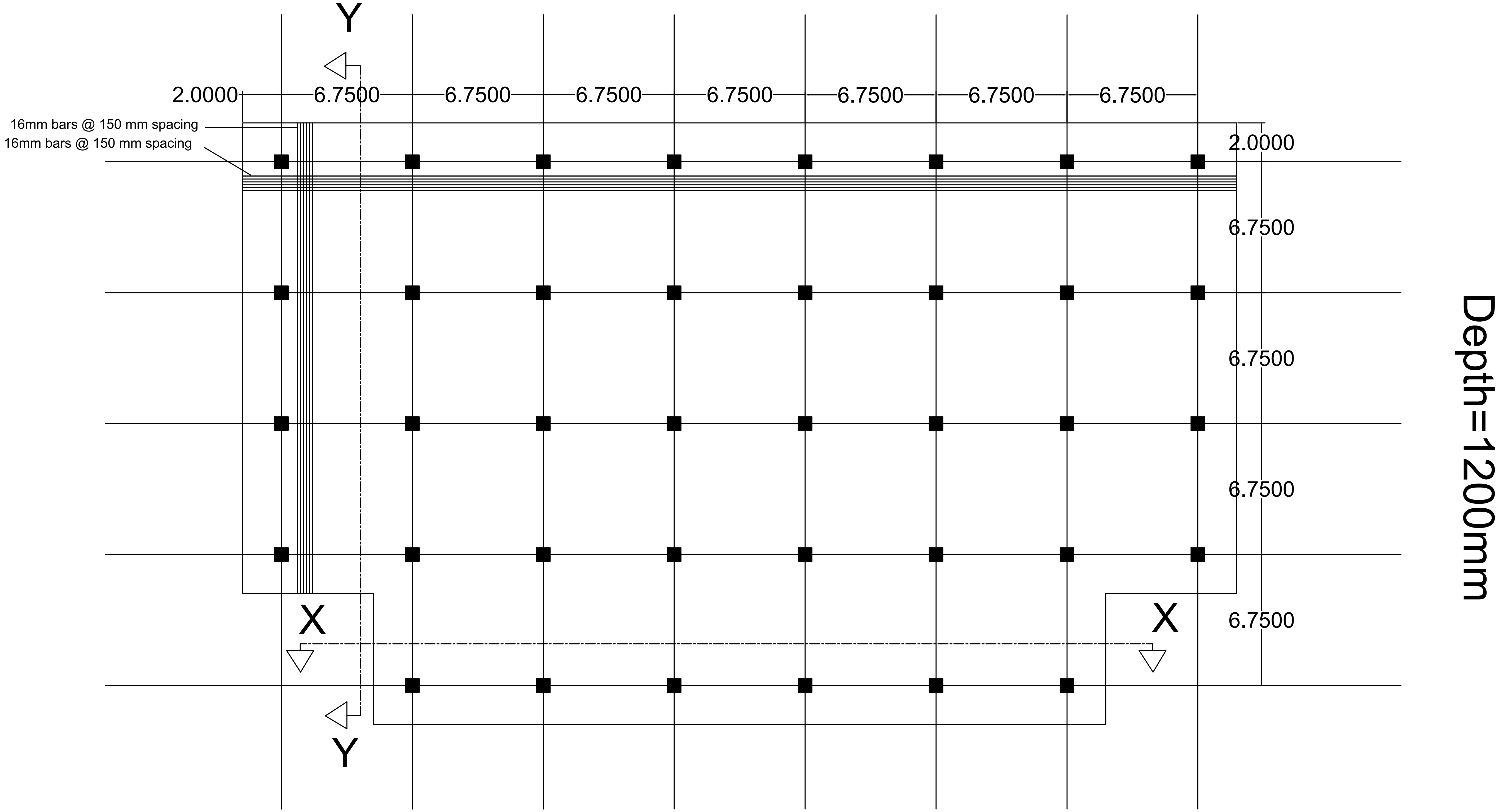


TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : STAIRCASE PLAN, SECTION, AND DETAILING	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Scale : Fit to scale DWG No. :31 Sheet No. :31



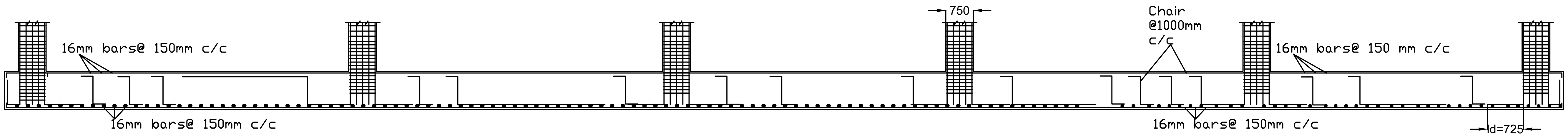
Top Bar Reinforcement of Footing

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : TOP BAR REINFORCEMENT IN MAT FOUNDATION	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. :32 Sheet No. :32

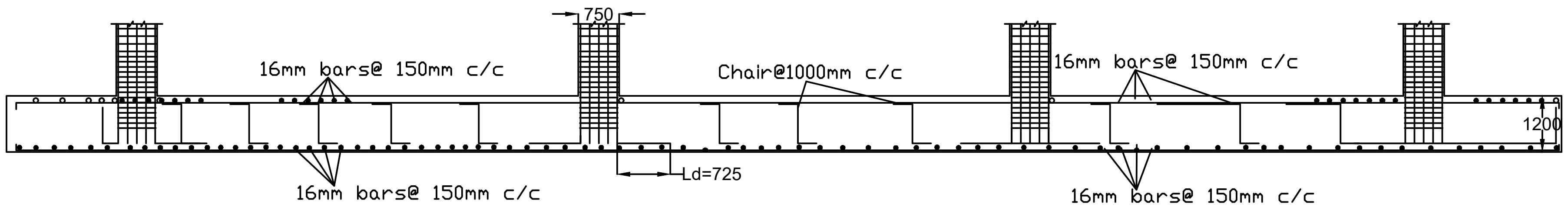


Bottom Bar Reinforcement of Footing

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : BOTTOM BAR REINFORCEMENT IN MAT FOUNDATION	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. :33 Sheet No. :33

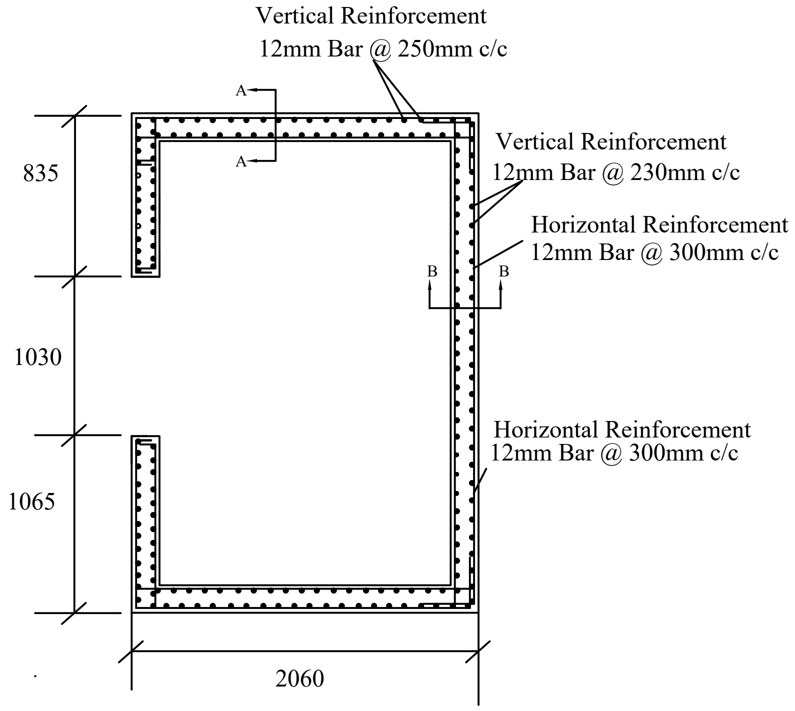


SECTION ALONG X-X AXIS

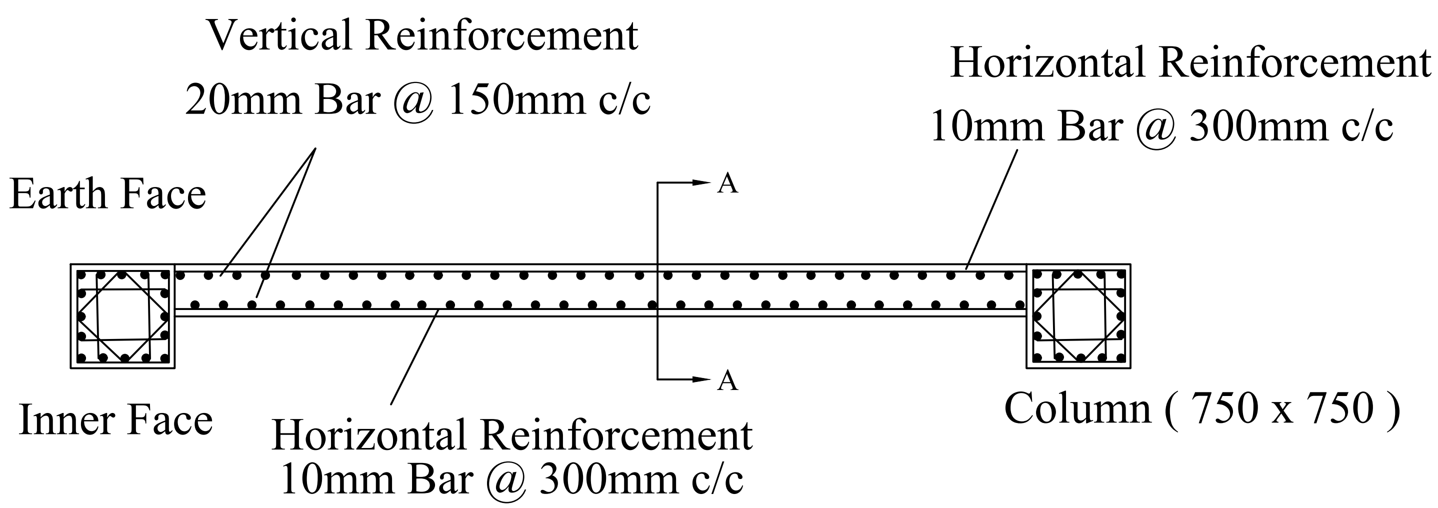


SECTION ALONG Y-Y AXIS

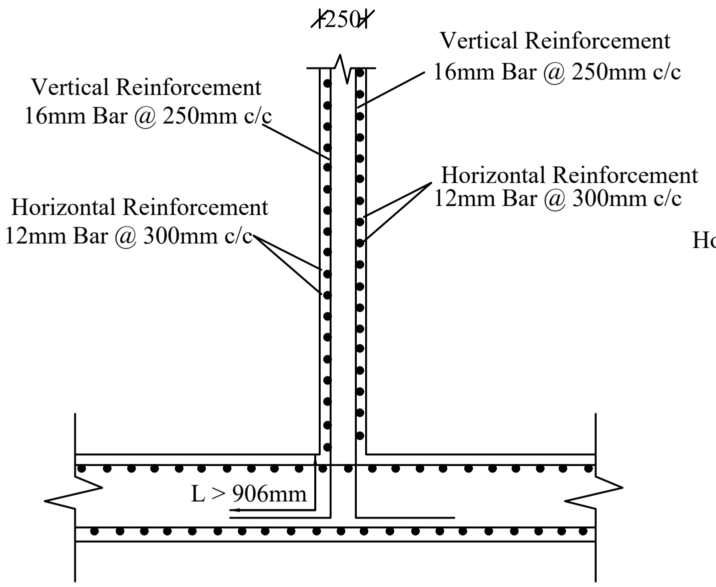
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : FOOTING SECTIONS	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. :34 Sheet No. :34



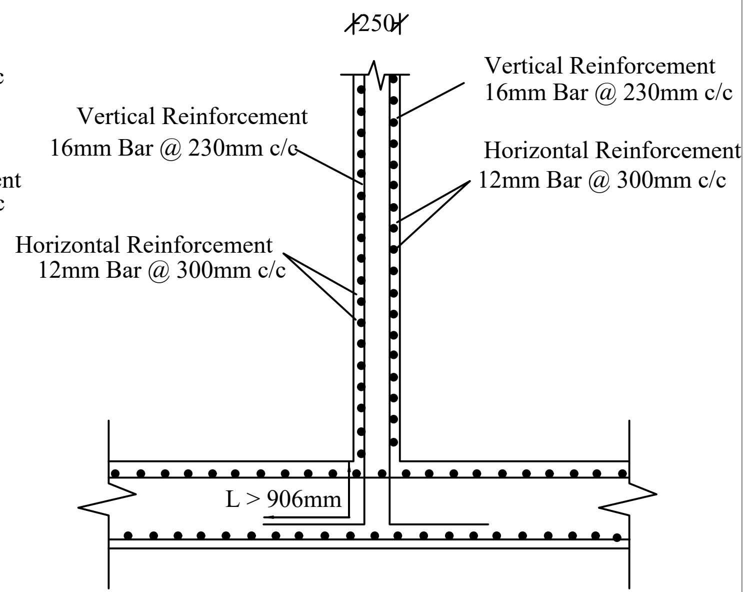
Lift Shear Wall Detailing in Plan



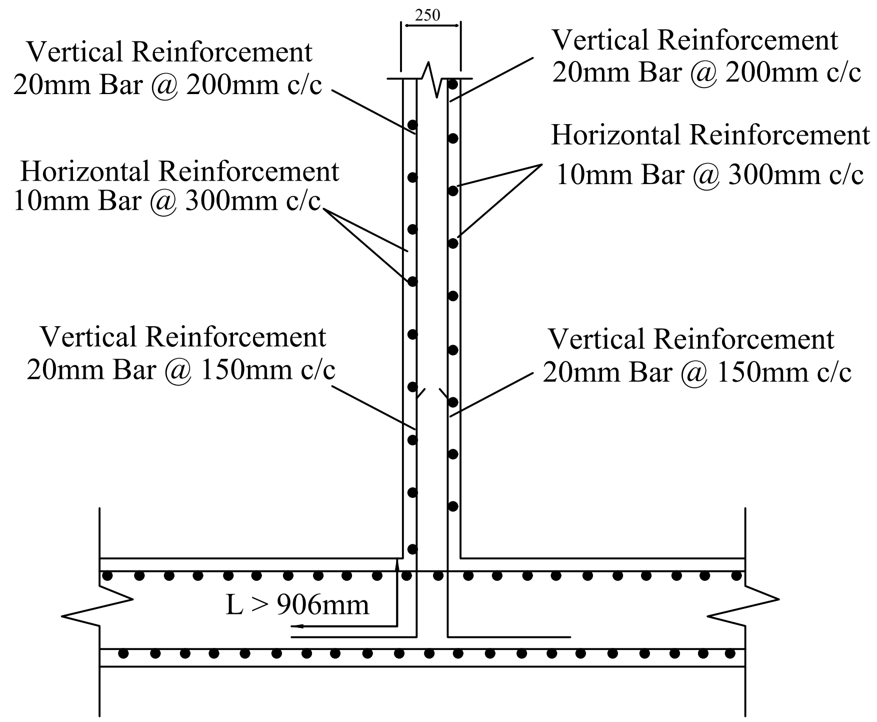
Basement Retaining Wall Detailing in Plan



Vertical Section at A-A



Vertical Section at B-B



Vertical Section at A-A

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	Project Title : EARTHQUAKE RESISTANT DESIGN OF MULTI-STORIED HOSPITAL BUILDING	Sheet Title : Detailing of Lift Shear Wall and Basement Wall	Group Members : Dipak Dhakal (075 BCE 051) Kiran Kumar Maharjan (075 BCE 075) Kushal Sharma (075 BCE 076) Nimee Tiwari (075 BCE 087) Nishant Awasthi (075 BCE 090) Nishchal Nath Sigdel (075 BCE 091)	Project Supervisor : Asst. Prof. Arun Paudel Date : 2080 / 01 / 14	Fit to scale DWG No. :35 Sheet No. :35

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