TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
DEPARTMENT OF CIVIL ENGINEERING

FINAL YEAR PROJECT REPORT ON "EARTHQUAKE RESISTANT DESIGN OF EIGHT STOREY APARTMENT BUILDING"

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Supervisor:

Asst. Prof. Subash Bastola

APRIL, 2023

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

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CERTIFICATE

This is to certify that this project work entitled "**EARTHQUAKE RESISTANT DESIGN OF EIGHT STOREY APARTMENT 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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PREFACE

This project work is not only focused on the curriculum of B.E. Civil final semester, but also available to the student community, a report that deals with various aspects under the heading "Earthquake Resistant Analysis and Design of Multistoried Commercial Building.

The use of reinforced concrete as a constructional material, for building shelters has increased significantly in last few decades, especially in the urban areas of Nepal. There may be various reasons which lead to the design of building by proper analysis, design and detailing with respect to safety, economy, stability, strength. The verification of quality of design of the various structural components of a building before construction and quality control of work during construction is important. Moreover, considerations of different types of loads that are to act during service life of structure is must. This report presents all considerations made, procedure adopted and results obtained for the structural design of commercial building to be constructed in Kathmandu.

The purpose of this report is to present the necessary background in using the method set out in IS 456-2000, IS 875 part (I, II) (1987), IS 13920-2016 for ductility, Seismic code IS 1893-2016, Design aids SP34 and SP16. The report comprises all the subject matter regarding the topic of concern, maintained in sections and annexes. 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.

Every care has been taken to make this report free of errors, yet slip may occur. We warmly welcome constructive criticism and shall be obliged, if errors are brought to our notice.

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Abbreviations

IS	Indian Standard
RS	Response Spectrum
NBC	National Building Code of Nepal
ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ULS	Ultimate Limit State
DL	Dead Load
LL	Live Load
EQ	Earthquake
BF	Basement Floor
GF	Ground Floor
1F	First Floor
2F	Second Floor
3F	Third Floor
RF	Roof Floor
HF	Helipad Floor
EC	Elevator Cover
FOS	Factor Of Safety
CM	Center of Mass
CS	Center of Stiffness
Cl.	Clause
RCC	Reinforced Cement Concrete
IBC	International Building Code
OPD	Outpatient Department
CCSD	Central Sterile Supply Department
CT	Computed Tomography
SLS	Serviceability Limit State
SB	Secondary Beam
FEM	Finite Element Method
MOI	Moment of Inertia
SPT	Standard Penetration Test
SN	Serial Number
SP	Special Publication
LC	Load Combination
FEMA	Federal Emergency Management Agency
NEHRP	National Earthquake Hazards Reduction Program

1. OVERVIEW

1.1 Introduction

The proposed project is "Earthquake Resistant Analysis and Design of Multistoried Residential Building" to be built in Kathmandu. 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 Storey Building".

The bearing capacity of foundation soil at site condition is taken as 150 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.

A designer has to deal with numerous structures ranging from simple to more complex ones like a multistoried frame building. All the structural members are subjected to various loads viz. concentrated loads, uniformly distributed loads, live loads, earthquake load, wind load, etc. The structure transfers the loads acting on it to the supports and ultimately to the ground. While transferring the loads acting on the structure, all members of the structure are subjected to the internal forces like axial forces, shear forces, bending and torsion moments.

Structural Analysis deals with analyzing these internal forces in the structural member developed as a result of various loading conditions or combinations.

Structural Design deals with sizing various members of the structures to resist the internal forces to which they are subjected in the course of their life cycle. Unless the proper structural detailing method is adopted the structural design will be no more effective. A Standard Code of practice (Indian Standard code in our case) should be thoroughly followed and implemented for proper analysis, design and detailing with respect to safety, economy, stability, strength.

Besides the scarcity of land, earthquake is one of the dominant constraints while designing the multistory frame building in earthquake prone zone like Kathmandu. According to IS

1893: 2016, Kathmandu lies on V zone, the severest one, hence the effect of earthquake is predominant to wind load. So, the building is analyzed for earthquake as lateral load. The seismic coefficient design method as stipulated in IS 1893:2016 is applied to analyze the building for earthquake. The 3 dimensional moment resistance frame with shear wall is considered as the main structural system of the building.

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 storey 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 Theme of Project work

This group under the project work has undertaken the Computer Aided Structural Analysis and Design of Multi Storey Building. The main aim of the project work under the title is to acquire knowledge and skill with an emphasis on practical application. Besides the utilization of analytical methods and design approaches, exposure and application of various available codes of practices is another aim of the work.

1.3 Objectives

The aim of reinforced concrete 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 the entire load and the deformation of normal construction and use and have adequate durability and adequate resistance to the effect of misuse and fire.

The specific objectives of the project work are:

a. To prepare plan of the building to meet the requirements for its intended use.
b. To identify the structural arrangement of the plan.
c. To modify the building for structural analysis.
d. To analyze the structure using structural analysis program.
e. To perform sectional design of the structural members.
f. To prepare detail structural drawing of the design.

1.4 Building Description

a. Building Type: Eight Storey RCC Framed Apartment Building
b. Structural System: RCC Space Frame
c. Plinth area covered: 413.646 m^2 (4452.448 sq.ft)
d. Type of Foundation: Mat Foundation
e. No. of Storey Floor: 8 + 1 Basements
f. Floor Height: 3.2 m
g. Type of Sub-Soil: Medium
h. Soil Seismic zone: V

According to IS 456-2000, Cl. 27, structures in which changes in plan dimensions take place abruptly shall be provided with expansion joints at the section where such changes occur. Reinforcement shall not extend across an expansion joint and the break between the sections shall be completed. Normally structure exceeding 45m in length is designed with one or more expansion joints.
The design is intended to serve quality service to the general public round the clock.

1.5 Identification of Load

a. Seismic load is calculated according to IS: 1893 (Part I) - 2016 assuming Kathmandu located at Zone V
b. Dead loads are calculated as per IS: 875 (Part I) -1987
c. Imposed loads according to IS: 875 (Part II) -1987

1.6 Code of Practices

Following codes of practices developed by Bureau of Indian Standards were followed in the analysis and design of building:

a. IS 456:2000 (Code of 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 (to assess dead loads)
e. IS 875 (part 2):1987 (to assess live loads)
f. SP 16 and SP 34 (design aids and hands book)

1.7 Method of Analysis

The building is modeled as a space frame. ETABS 16.0.2 is adopted as the basic tool for the execution of analysis. ETABS 16.0.2 program is based on Finite Element Method. Due to possible actions in the building, the stresses, displacements and fundamental time periods are obtained using ETABS 16.0.2 which are used for the design of the members.

Mat foundation, steel structures, staircase and slabs are analyzed separately.

1.8 Design

The following materials are adopted for the design of the elements:

- Concrete Grade:
    1. M25 for the beams, columns, slab and foundation
- Reinforcement Steel:
    1. 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 design moments, shear forces, axial forces and torsions are taken as computed by computer software program "ETABS 16.0.2" for the worst possible combination and a number of hand calculations are done so as to verify the reliability of the design results suggested by the software.

1.9 Scope

The project work is limited to the structural analysis and design only. Design and detailing of following structural elements are performed:

1. Slab
2. Beam
3. Column
    - Design and layout of the building services like pipeline, electrical appliances, sanitary and sewage system are not covered.
    - The required parking facilities are assumed to be provided in a underground basement; however, its design is not concerned with this project.
    - The project is not concerned with the existing soil condition of the locality.

- The bearing capacity of the soil is assumed.
- The environmental, social and economic condition of the locality is not considered.
- The project work is only related with the practical application of the studied courses in the field.

Detail cost estimate of the project is not included in this report

1.10 Structural Properties.

Total number of Column:	29
Section of column:	700 mm X 700mm
Depth of slab	150mm, 175mm
Foundation Depth	1100mm
Section of beam	400mm X 550mm 300mm x 400mm

Important values that are used in the analysis and calculation are:

Bearing Capacity of soil	: 150 KN/m^2
Unit weight of concrete	: 25 KN/m^3
Unit weight of masonry	: 20 KN/m^3
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	:1

2. Review of literature

2.1 Background

We are mainly dealing with earthquake resistant analysis and structural design of RCC framed concrete structure. Our main focus will be on obtaining design output by limit state method on basis of structural design incorporating seismic considerations.

Earthquake is the natural phenomena caused by release of seismic wave (p-wave and s- wave) from the earth surface from a faint tremor to a wild motion due to sudden release of energy stored to the rocks beneath the earth surface. It occurs in clusters. It dates as old as earths' history itself, however, our knowledge and interpretations about their behavior & ways to minimize damages due to them is recent. Most of the earthquake are minor and go un-noticed but the major ones, though occasional are responsible for huge loss of life and property.

The theoretical development of earthquake forces in structure reveals that the maximum elastic response acceleration during earthquake (range for which structure is designed) would be several times larger than the design acceleration i.e. the seismic coefficient specified in most of the codes. This situation is quite different to the approach made in codes for loads such as design loads are usually higher than the actual ones. It is based on the probability of the in-frequent 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 and there by dissipate the energy of motion using material and structural ductility. It is clear that, to achieve ductile behaviors, brittle modes of failure due to shear, anchorage and bond should be avoided. This concept is derived from a basic philosophy that damage of the building is permissible as long as the structure doesn't collapse catastrophically during a severe earthquake. This fact guides concept that vertical load-bearing member providing basic support of structure should be strong and can be achieved by applying 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 has been originated from ultimate or plastic design. The object of design based on the limit state concept is to achieve an acceptable probability that a structure will not become unserviceable in its life time for the use for which it is intended, i.e. it will not reach a limit state. A structure with appropriate degrees of reliability should be able to withstand safely all the loads that are liable to act on it through-out its life and it should satisfy the serviceability requirements. All relevant limit states must be considered in design to ensure an adequate degree of safety and serviceability.

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

d. In the structure shall be assumed to be 0.67 times the characteristic strength. The partial safety factor γm =1.5 shall be applied.

e. The tensile strength of concrete is ignored.

f. The stresses in the reinforcement are derived from the representative stressstrain curve for the type of steel used. For design purpose the partial safety factor γm =1.15 shall be applied.

g. 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, fy = characteristic strength of steel.

Es = modulus of elasticity of steel.

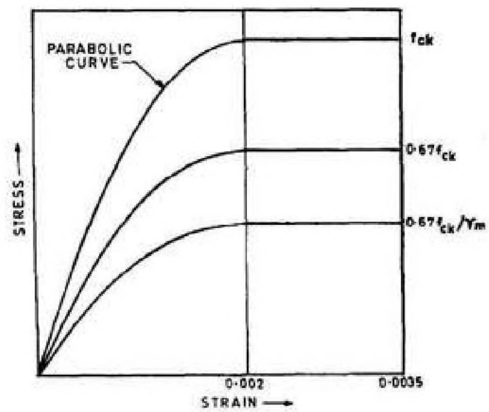


Figure 2. 1 Stress-Strain curve for Concrete

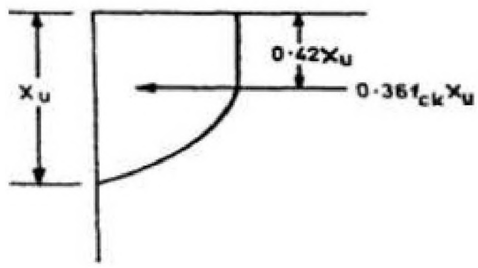


Figure 2. 2 Stress block parameters

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:

2.2.1 Limit state of collapse

This state corresponds to the maximum load carrying capacity. Violation of collapse limit state implies failure in sense that a clearly defined limit state of structural usefulness has been exceeded. However, it does not mean a complete collapse. This limit state may correspond to:

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

2.2.2 Limit state of serviceability

This state corresponds to development of the excessive deformation and is used for checking members in which magnitude of deformation may limit the use of the structure or its components. This state may correspond to:

2.2.2.1 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.

2.2.2.2 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 18:456-2000, 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.

2.2.2.3 Control of Vibration

A dynamic load is any load of which the magnitude, direction or position varies with the time and almost any RCC structural system may be subjected to one form or another loading during its life-time. Similarly, structural response i.e. resulting stresses or deflections is also time-varying or dynamic and is expressed in terms of displacements.

The limit state concept of design of reinforced concrete structures takes into account the probabilistically and structural variation in the material properties, loads and safety factors.

3. Load and Load Calculation

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.

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.

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.

3.2.1 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.

3.2.2 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.

3.2.3 Eccentricity of vertical loads

When transferring the loads from parapets, partition walls, cladding walls and facade 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.

3.2.4 Seismic Loads

These are the load resulting from the vibration of the ground underneath the superstructure 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 predominantly 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 3. 1 Seismic Design Criteria

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

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.

Lateral Strength:

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

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.

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.

3.3 Estimation of loads

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

3.3.1 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.

3.3.2 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 storey in the columns and that for calculating earthquake loads may be considered in the calculations later.

3.3.3 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.

Response Spectra:

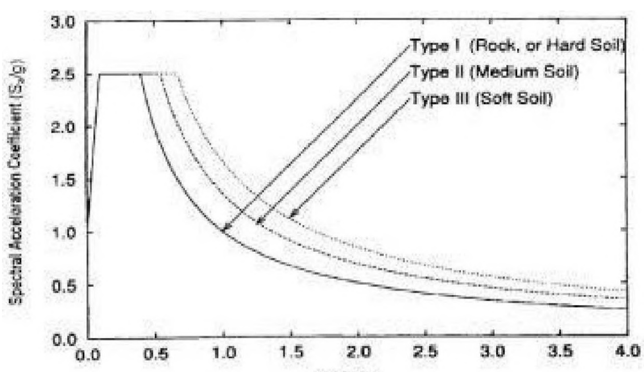


Figure 3. 1 Response spectra for rock and soils for 5% damping motion records of eight earthquakes.

The representation of the maximum response of idealized single degree of freedom

system having certain period of vibration and damping during given earthquake is referred to as response spectrum. The maximum response, that is, maximum absolute acceleration, maximum relative velocity or maximum relative displacement of single degree of freedom system is plotted against the damped natural period and for various damping values.

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 $(Vb) = A_h W$

Where, A_h=Design horizontal acceleration spectrum.

W= Seismic weight of building

$$A_h = \frac{ZIS_a}{2Rg}$$

Where, Z = Zone factor, From Table 2; clause 6.4.2

I = Importance factor, From Table 6; clause 6.4.2

R = Response reduction factor, From Table 7; clause 6.4.2

$\frac{S_a}{g}$ = Average response acceleration coefficient for rock or soil sites, Fig 2, Table 3

The fundamental time period of the vibration,

Clause 7.6.1 IS 1893:2002

$Ta = 0.075 * h^{0\ 75}$ (for RC frame building)

$= 0.080 * h^{0\ 75}$ (for steel frame building)

Clause 7.6.2 IS 1893:2002

$Ta = \frac{0.09h}{\sqrt{d}}$ (for other building)

Where, Ta= Fundamental natural time of vibration 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

TERMINOLOGIES

Response reduction factor:

The response reduction factor assigned to different types of structural systems reflects design and construction experience, as well as evaluation of performance of structure in major and moderate earthquakes. It endeavors to account for the energy absorption capacity of the structural system by damping (which is normally taken as 5% of critical damping for RCC structures) and in-elastic action through several load reversals. A building with a value R equal to 1.0 corresponds to a structural system exhibiting little or no ductility and value greater than 1.0 is presumed to be capable of undergoing inelastic cyclic deformation. 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 1S: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.

Storey drift:

The relative inter-storey horizontal displacement is referred to as storey drift. A limitation on storey 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 storey height in the elastic range is imposed by IS:1893-2002.

Regularity:

The regularity of a building can significantly affects its performance during a strong earthquake. Past earthquakes have repeatedly shown that buildings having irregular configurations suffer greater damage than buildings having regular configuration.

Regular structure has no significant physical discontinuities in plan, vertical configuration or in their lateral force resisting system whereas, irregular structure has significant physical discontinuities in configuration or in their lateral force resisting system. They may have; either plan irregularity or vertical irregularity or mass irregularity.

Load Combinations

The analysis was performed for various 19 combinations and time history separately. Following are those 19 combinations as suggested by IS: 1893-2002, Clause 6.3 .1.2

i. 1.5 (DL + IL)
ii. 1.2 (DL + IL ± ELx)
iii. 1.2 (DL + IL ±ELy)
iv. 1.5 (DL ±ELx)
v. 1.5 (DL±ELy)
vi. 0.9DL±1.5 EL
vii. 0.9DL± 1.5 Ely
viii. 1.2(DL + IL + RSX)
ix. 1.2(DL + IL + RSY)
x. 1.5(DL + RSX)
xi. 1.5(DL + RSY)
xii. 0.9DL + 1.5RSX
xiii. 0.9DL + 1.5RSY

4. Methodology

4.1 Structural System

Any structure is made up of structural elements (load carrying, such as beams and columns and non-structural elements (such as partitions, false ceilings, doors). The structural elements put together, constitute the structural systems. Its function is to resist effectively the action of gravitational and environmental loads, and to transmit the resulting forces to the supporting ground without significantly disturbing the geometry, integrity and serviceability of the structure.

4.2 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 are appropriately done and accordingly beam arrangement is carried out so that the whole building will be aesthetically, functionally and economically feasible. 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 Multistoried RCC framed building is described below:

4.3 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.

4.4 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.

4.5 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 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. Crack or yield pattern.

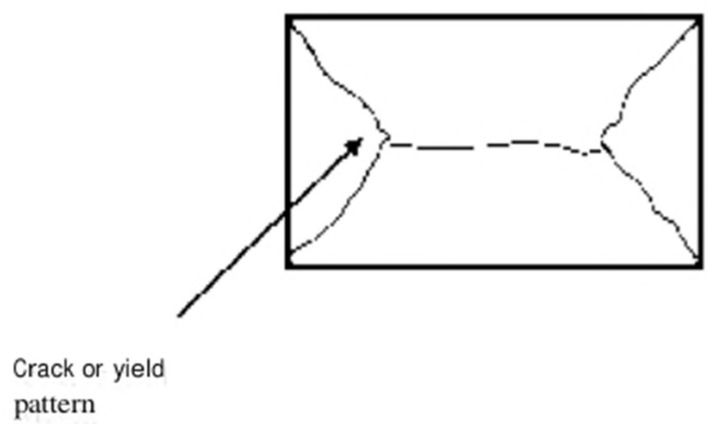


Figure 4. 1 Cracking in 2-way slab

4.6 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

5. Preliminary Design

The following remarks will be helpful in choosing the sections;

a. 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.
b. Richer concrete mixes can be used in lower storey elements to avoid frequent change in sections. Some size variation can also be avoided by reducing column steel upwards in building.
c. 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.
d. 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 25b, 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 approximate dimensions of structural elements were determined in preliminary design, verified from architect, so that they act as guidelines in analysis and aid to make final design safe and economical

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 1893:2002.

For design of slab: BC45

The project building has largest span of 4.91m X 5.17 m.

lx = 4910mm (i.e. the smallest of the two dimensions of the slab)

ly = 5170mm (i.e. the largest of the two dimensions of the slab)

$$\frac{ly}{lx} = \frac{5170}{4910} = 1.053 < 2$$

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

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

Here, l= lx= 4910 mm

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

(For continuous slab)

β =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)**

$d_{eff} = \frac{4910}{1*1*1.38*26}$= 136.84 mm < 150mm

Overall thickness of slab = 140+10/2 +15 =160mm

Overall depth will exceed 150mm, not valid.

Add secondary beam dividing slab to two equal halves.

After:

lx' = ly/2=2583mm (i.e. the smallest of the two dimensions of the slab)

ly = 4910mm (i.e. the largest of the two dimensions of the slab)

$$\frac{ly}{lx} = \frac{4910}{2583} = 1.901 < 2$$

Hence the slab is two way continuous

slab. From IS 456:2000 Clause 23.2

Here, l= lx'= 2583mm

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

(For continuous slab)

β = 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)**

$d_{eff} = \frac{2582.5}{1*1*1.38*26}$ = 71.97mm (=Approx.75mm)

D = 75+ 0.5*10+ 15 = 90.5mm

So provide the overall depth of 125mm.

Add second secondary beam dividing slab to two equal halves.

After:

lx = 4250mm (i.e. the smallest of the two dimensions of the slab)

ly = 6350mm (i.e. the largest of the two dimensions of the slab)

$$\frac{ly}{lx} = \frac{6350}{4250} = 1.495 < 2$$

Hence the slab is two way continuous

slab. <u>From IS 456:2000 Clause 23.2</u>

Here, l= lx= 4250 mm

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

(For continuous slab)

β = 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)**

$d_{eff} = \frac{4245}{1*1*1*1.38*26} = 118.31$ mm

Take the overall depth(D) as 125 mm

A clear cover of 15mm for main bar of diameter 10mm we get the overall depth of slab as:

Effective Depth of Slab (d_{eff}) = Effective Depth - 0.5x Dia. of main bar - Clear Cover

$= D - \frac{\emptyset}{2}$ -clear cover

= 125 - 0.5 X 10 - 15 (∅ can be either 10 mm or 8 mm not more or less)

d_{eff} = 105mm < 118.31mm

Secondary Beam will be required for all slab with span in shorter direction greater than
= 26* 1.38* 105mm = 3.767 m

Preliminary design of secondary beam:

Taking maximum span of beam = 6350mm

lx = 6350/2 = 3175mm

ly = 4245mm

From the deflection criteria:

$d \geq \frac{lx}{\alpha\beta\gamma\delta\lambda}$

$d_{eff} = \frac{lx}{\alpha\beta\gamma\delta\lambda} = \frac{3175}{12} = 264.58mm$

Provide d_{eff} = 300mm

Overall depth (D) = 350mm (50mm cover)

D/b = 1.5

$b = \frac{350}{1.5} = 233.33mm$

provide width of secondary beam as 250 mm.

Provide secondary beam as 350 x 250 mm.

Preliminary design of main beam:

Taking maximum span of beam = 6350mm

lx = 6350mm

From the deflection criteria:

d≥ $\frac{lx}{\alpha\beta\gamma\delta\lambda}$

d_{eff} = $\frac{lx}{\alpha\beta\gamma\delta\lambda}$ =$\frac{6350}{15}$ = 423.33mm

Provide d_{eff} = 430mm

Overall depth (D) = 480mm (50mm cover)

D/b = 1.5

b =$\frac{480}{1.5} = 320mm$

provide width of main beam as 350mm

Provide beam of 350 x 480 mm.

Preliminary design of Column

Dead load Calculation		
Load tpe	Calculation	Load (KN)
Self weight of slab	25*25.18*.125	78.69
Self weight of partition wall	19.6*3.05*(5.35+4.70)*.23	138.18
Floor Finish	25.18*1	25.18
Self-weight of beam	25*0.35*(0.48-0.125)*(5.35+4.705)	31.23
Self weight of secondary beam	25*0.25*(0.3-0.125)*(5.35+4.705)	11
Self weight of column	25*0.5*0.5*3.05	19.06
	Total	303.34
Live load Calculation		
Load tpe	Calculation	Load (KN)
Basement	25.18*4	100.72
Ground	25.18*4*0.9	90.648
1st	25.18*4*0.8	80.576
2nd	25.18*4*0.7	70.504
3rd	25.18*4*0.6	60.432
4th	25.18*4*0.6	60.432
5th	25.18*4*0.6	60.432
Roof	25.18*4*0.6	60.432
	Total	584.176
Floors	Dead Load	Live Load
Basement	303.34	100.72
Ground	303.34	90.648
1st	303.34	80.576
2nd	303.34	70.504
3rd	303.34	60.432
4th	303.34	60.432
5th	303.34	60.432
Roof	303.34	60.432
Total	2426.72	584.176

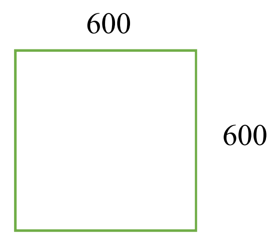
Calculation of dimensions required for above calculated load

Total Working load=2426.27+584.176=3010.896 KN

Total factored load = 1.5*3010.896=4516.344 KN

From IS 456:2000,CL 39.3

Assuming 2% steel and for fy=500N/mm^2 and M25 grade of concrete



$P_u = 0.4*f_{ck}\ A_g + 0.67 f_y A_{Sc}$

$4516.344 = 0.4*25*\ A_g(1-.002) + 0.67*500*A_g*0.02$

$A_g = 273712.8182 mm^2$

For square Column

$D^2 = 273712.8182 mm^2$

D=523.17 mm

Adopt D=600 mm

Therefore, size of column = 600mm x 600 mm

6. Modeling and structural analysis

6.1 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 ofthe element is based.

6.2 Analysis

I. Manual calculation

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

II. Use of standard software.

Two ofthe following analysis can be done by using standard software

- 2D-analysis
- 3D- analysis

Among various softwares, ETABS 16.0.2 has been used for our purpose.

6.3 Modeling and Analysis Tool

ETABS version 16.0.2 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.

6.4 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.

7. Idealization and analysis of structure

7.1 Idealization of Structure

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

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

1. Idealization of Load
2. Idealization of Structure

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

7.1.1 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.

7.1.2 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 ETABS2016 program for analysis.

7.1.3 Idealization of Staircase

Open welled staircase used in the building is idealized to behave as simply supported slabs, supported on beams at the floor and landing levels. This idealization helps us analyze the staircase slab in strips subjected to distributed loading on the landing strip and going of the slab. Detailing rules are then followed to address the negative bending moment that are induced on the joint of going and top flight in the staircase, the rigorous analysis of which is beyond or scope. Staircase being an area element is also assumed not to be a part of the integral load bearing frame structure. The loads from staircase are transferred to the supports as vertical reactions and moments.

7.1.4 Idealization of Beam and Column

Beam column idealization is one of the most critical aspects of structural 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 behavior 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 ofloading during earthquakes become ineffective in taking the tension induced in them and hence we ignore their contribution in design.

7.1.5 Idealization of Structural System

After idealizing individual elements, we idealize the structural system in its entirety to behave as our theoretical approximation for first order linear analysis and corresponding design. The building is idealized as unbraced space frame. This 3D space framework is modeled in ETABS 2016 for analysis. 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.

7.2 Structural Analysis

Analysis of Structure

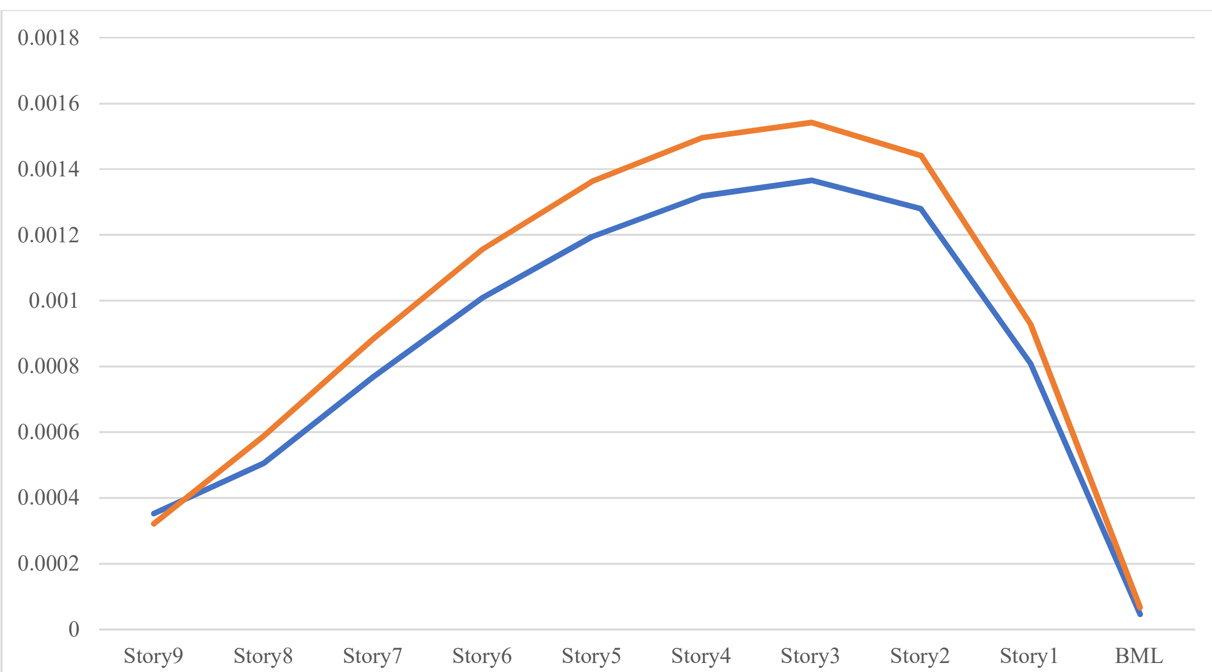
The analysis of structure is carried out in a commercial computer software ETABS 2016, the salient features of which are already explained in detail in methodology. The results of analysis are used according to our necessities in designing representative beams and columns sections. A detailed manual 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 125 mm, main beam of size 350 mm x 480 mm, secondary beam of size 350 mm x 250 mm and column of size 600 mm x 600 mm.

7.2.1 Storey Drift Computation from ETABS 2016 Analysis

Analysis is performed and the value of inter-storey drift for serviceability condition is computed from absolute displacements for earthquake loads in both horizontal directions. The relative inter-storey horizontal displacement is referred to as storey drift. A limitation on storey 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 storey height in the elastic range is imposed by IS 1893:2016.

Story	Drift in X-direction	Drift in Y-direction
Story9	0.000316	0.000343
Story8	0.000598	0.000516
Story7	0.000897	0.000786
Story6	0.001173	0.001034
Story5	0.001382	0.001222
Story4	0.001515	0.001346
Story3	0.001561	0.001392
Story2	0.001458	0.001302
Story1	0.000939	0.000819
BML	6.8E-05	4.5E-05

Table 7. 1 Storey Drift Check



As maximum drift ratio in our model is less than 0.004, it is okay.

7.2.2 Stiffness Irregularity Computation from ETABS 2016 Analysis

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

Soft Storey check in X and Y direction						
	X direction			Y direction		
Storey name	Lateral Stiffness	Lateral stiffness is NOT less than 80 percent of the average stiffness of three storeys	Lateral stiffness is NOT less than 70 percent of that in the storey above	Lateral Stiffness	Lateral stiffness is NOT less than 80 percent of the average stiffness of three storeys storey	Lateral stiffness is NOT
Roof	0			0		
8F	433560	ok	ok	0.044	ok	ok
7F	559014	ok	ok	500932.064	ok	ok
6F	604429	ok	ok	584940.741	ok	ok
5F	628950	ok	ok	613068.908	ok	ok
4F	647558	ok	ok	629088.312	ok	ok
3F	669440	ok	ok	639065.389	ok	ok
2F	730989	ok	ok	712276.427	ok	ok
1F	1298272	ok	ok	1210506.588	ok	ok
BML	1.2E+07	ok	ok	18573022.87	ok	ok

Table 7. 2 Soft storey check

7.2.3 Mass Irregularity Computation from ETABS 2016 Analysis

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

Mass irregularity check for X and Y direction			
Storey	Seismic Mass along X and Y axis	M_i/M_{i-1}	Seismic weight of any storey is less than 150 percent of that of floors below
Roof	15485.48	-	-
7F	537429.41	1.13	Regular
6F	608824.7	1.13	Regular
5F	630699.88	1.04	Regular
4F	630699.88	1.00	Regular
3F	630699.88	1.00	Regular
2F	630699.88	1.00	Regular
1F	630699.88	1.00	Regular
GF	630929.52	1.00	Regular
Basement	681586.95	1.08	Regular
Base	105227.06	0.15	Regular

Table 7. 3 Mass irregularity check

7.2.4 Torsion Irregularity Computation from ETABS 2016 Analysis

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/storey. 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 storey drift at one end of the structures transverse to an axis is more than 1.2 times the average of the storey drifts at the two ends of the structure. (IS 1893-2016).

Story	Load Case/Combo	Direction	Ratio		Story	Load Case/Combo	Direction	Ratio	
Story9	EQx	X	1.027	ok	Story9	EQy	Y	1.017	ok
Story8	EQx	X	1.129	ok	Story8	EQy	Y	1.001	ok
Story7	EQx	X	1.147	ok	Story7	EQy	Y	1.001	ok
Story6	EQx	X	1.155	ok	Story6	EQy	Y	1	ok
Story5	EQx	X	1.159	ok	Story5	EQy	Y	1.001	ok
Story4	EQx	X	1.16	ok	Story4	EQy	Y	1.002	ok
Story3	EQx	X	1.158	ok	Story3	EQy	Y	1.002	ok
Story2	EQx	X	1.151	ok	Story2	EQy	Y	1.003	ok
Story1	EQx	X	1.129	ok	Story1	EQy	Y	1.005	ok
BML	EQx	X	1.121	ok	BML	EQy	Y	1.089	ok

Table 7. 4 Torsional irregularity check

7.2.5 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)

Mode	Period	UX	UY	SumUX	SumUY	RZ
	sec					
1	0.954	0.0177	0.6274	0.0177	0.6274	0.0672
2	0.926	0.6931	0.0203	0.7109	0.6477	0.0002
3	0.823	0.0028	0.0630	0.7137	0.7107	0.6366
4	0.299	0.0046	0.0799	0.7182	0.7906	0.0093
5	0.291	0.0906	0.0056	0.8088	0.7962	0.0001
6	0.257	0.0012	0.0085	0.8100	0.8046	0.0847
7	0.164	0.0034	0.0301	0.8135	0.8348	0.0043
8	0.159	0.0335	0.0044	0.8470	0.8392	0.0001
9	0.14	0.0012	0.0033	0.8483	0.8424	0.0333
10	0.108	0.0030	0.0155	0.8513	0.8579	0.0029
11	0.105	0.0174	0.0040	0.8687	0.8619	0.0002
12	0.091	0.0019	0.0017	0.8706	0.8637	0.0181
13	0.08	0.0003	0.0083	0.8709	0.8719	0.0014
14	0.078	0.0082	0.0006	0.8792	0.8725	0.0017
15	0.068	0.0020	0.0059	0.8812	0.8784	0.0015
16	0.068	0.0071	0.0022	0.8882	0.8807	0.0005
17	0.064	0.0002	0.0009	0.8884	0.8815	0.0100
18	0.059	0.0000	0.0000	0.8884	0.8815	0.0022
19	0.055	0.0008	0.0073	0.8892	0.8888	0.0005
20	0.054	0.0086	0.0008	0.8978	0.8897	0.0001
21	0.048	0.0001	0.0005	0.8979	0.8902	0.0087
22	0.045	0.0012	0.0036	0.8991	0.8939	0.0004
23	0.045	0.0046	0.0012	0.9037	0.8951	0.0000
24	0.04	0.0004	0.0011	0.9041	0.8961	0.0002
25	0.039	0.0002	0.0003	0.9043	0.8964	0.0049
26	0.039	0.0014	0.0005	0.9057	0.8969	0.0000
27	0.035	0.0001	0.0001	0.9058	0.8969	0.0015
28	0.026	0.0940	0.0000	0.9998	0.8970	0.0001
29	0.023	0.0001	0.0855	0.9999	0.9825	0.0158
30	0.019	0.0000	0.0175	1.0000	1.0000	0.0934

Table 7. 5 Modal mass participation ratio

7.2.6 Response Spectra Analysis

Using Response Spectra Analysis storey drift in X and Y direction were determined as follows;

Story	Drift in X-direction	Drift in Y-direction
Story9	0.000251	0.000282
Story8	2.6E-05	0.000496
Story7	0.000418	0.000807
Story6	0.000643	0.00111
Story5	0.000868	0.001378
Story4	0.001066	0.001587
Story3	0.001221	0.001707
Story2	0.001315	0.00164
Story1	0.001279	0.001014
BML	0.000845	5.7E-05

Table 7. 6 Storey drift check Using RS

A limitation on storey 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 storey height in the elastic range is imposed by IS 1893:2002. And our storey drift is less than 0.004 so it is ok.

A building is said to be torsionally irregular when the maximum storey drift at one end of the structures transverse to an axis is more than 1.2 times the average of the storey drifts at the two ends of the structure. (IS 1893-2002). In our case the ratio is less than 1.2 so it is ok.

Story	Direction	Max Drift	Avg Drift	Ratio	Direction	Max Drift	Avg Drift	Ratio	Check
		mm	mm			mm	mm		
Story9	Y	0.079	0.077	1.025	Y	0.846	0.817	1.036	ok
Story8	X	1.337	1.273	1.05	Y	1.588	1.339	1.186	ok
Story7	X	2.056	1.954	1.052	Y	2.582	2.161	1.195	ok
Story6	X	2.779	2.646	1.05	Y	3.554	2.979	1.193	ok
Story5	X	3.41	3.251	1.049	Y	4.409	3.693	1.194	ok
Story4	X	3.908	3.722	1.05	Y	5.079	4.254	1.194	ok
Story3	X	4.207	3.995	1.053	Y	5.462	4.629	1.18	ok
Story2	X	4.093	3.867	1.058	Y	5.249	4.404	1.192	ok
Story1	X	2.706	2.56	1.057	Y	3.245	2.711	1.197	ok
BML	X	0.177	0.176	1.01	Y	0.17	0.142	1.198	ok

Table 7. 7 Torsion Irregularity Check Using RS

8. 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.

8.1 Design Procedure

8.1.1 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

8.1.2 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.

8.1.3 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 storey, so that, we generally provide symmetrical bar arrangement in a column section and the steel area is kept constant throughout a given storey.

8.1.4 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.

8.1.4.1 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:

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

Where,

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

$$\frac{P}{A} = \frac{\text{Mylx} - \text{Mxlxy}}{lxly - lxy^\wedge 2} y \pm \frac{\text{Mxly} - \text{Mylxy}}{lxly - lxy^\wedge 2} x$$

Where,
I,= M.O.I. of footing about X-axis
I=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, = moment about X-axis
M, = moment about Y -axis
I =product of inertia, may be +ve or ve

8.1.4.2 Raft foundation

If the loads transmitted by the column in a structure are so heavy or the allowable soil pressure so small that individual footings would cover more than about one• half of the area, it may be better to provide continuous footings under all columns and walls. Such footing is called mat or raft foundation. Raft foundations are also used to reduce the settlement of structures located above highly compressible deposits. When earthquake forces are considered along with other normal design force, the permissible stress in material and allowable bearing pressure of foundation soil can be increased by 33% &50% resp. in the elastic method of design
of mat foundation (Clause no.6.3.5, IS: 1893-2016).

If the columns are equally spaced and their loads are equal, the pressure on the soil will be uniform, otherwise moments of the loads may be taken about the same center of the base and pressure distribution is determined using above formula. Since these equations were derived from rigid member, and a raft is not a rigid member, the pressure and resulting internal stresses may be seriously in error if the eccentricity is very large. The weight of the raft is not considered in the structural design because it is assumed to be carried directly by the sub-soil.
While designing the footing of building, preference should be given to the isolated footing because of its simplicity & ease in construction, proper dealing of each column load. But in our building, we design the mat foundation because each individual footing would cover more than 50% of plinth area. The depth of footing is ascertained by considering bending moment, one-way shear &two-way shear. The depth required by one-way shear is excess. So, considering the economy, depth satisfying the B.M. & two-way shear is adopted and the deficiency in capacity to resist one-way shear is designed by providing shear reinforcement to mat design. The raft is designed by conventional method by dividing mat into a series of continuous strips centered on the appropriate column rows in both directions. The shear and B.M. are calculated as continuous beam analysis by 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.

Detailed slab calculation

S1

Two adjacent edge discontinuous

Step 1

Calculation of Effective depth

Short span length (l)	4.0900	m
long span length(l)	4.2450	m
l/d	35.8800	
Calculated Effective depth(d)	113.9911	
Effective depth(d)	130.0000	mm
∅	10.0000	mm
Clear cover	15.0000	mm
Calculated overall depth	150.0000	
Overall depth (D)	150.0000	
d_x	130.0000	mm
dy	120.0000	mm

Step 2

Calculation of Effective length

According to IS 456:2000, cl.22.2(a)

Along X-axis

i.	Clear span + eff depth	3.8200	m
ii,	c/c distance	4.0900	m
	l_{ex}	3.8200	m

According to IS 456:2000, cl.22.2(a)

Along Y-axis

i.	Clear span + eff depth	3.9650	m
ii,	c/c distance	4.2450	m
	l_{ey}	3.9650	m

$\frac{ley}{lex}$ 1.0380 **Two way slab**

Step 3

Load calculation

Live load	2.0000	kN/m
Floor finish	1.5000	kN/m
Partition wall	0.0000	kN/m
Self weight	3.7500	kN/m
Total Load	7.2500	kN/m
Factored load	10.8750	kN/m

Step 4

α_x^+	0.0369	**From IS**
α_x^-	0.0493	**456:2000**
α_y^+	0.0350	**, Annex**
α_y^-	0.0470	**D , Table**
$M_x^+ = \alpha_x^+ wl_x^2 =$	5.8557	kNm
$M_x^- = \alpha_x^- wl_x^2 =$	7.8204	kNm
$M_y^+ = \alpha_y^+ wl_x^2 =$	5.5542	kNm
$M_y^- = \alpha_y^- wl_x^2 =$	7.4585	kNm

Step 5

Check for depth from moment criteria

f_{ck}	25.0000		
f_y	500.0000		
b	1000.0000		
M_{max}	7.8204	kNm	
$M_{max} = 0.133 f_{ck} b d^2$			
$\therefore d_{req} =$	48.4973	mm	ok

Step 6

Calculation of area of reinforcement and spacing
According to Cl 26.5.2.1,IS 456:2000

$A_{st,mim} = 0.12\% of bD =$	180.0000	mm^2
Area of one rebar =	78.5714	

According to Cl G.1.1,IS 456:2000

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

$M_x^+ =$	5.8557	kNm
$\therefore A_{stx}^+ =$	105.3078	mm^2

$$Spacing_x^+ = \frac{1000}{\left(\frac{A_{stx}^+}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^+$ 746.1121 mm

$M_x^- =$	7.8204	kNm
$\therefore A_{stx}^- =$	141.4378	mm^2

$$Spacing_x^- = \frac{1000}{\left(\frac{A_{stx}^-}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^- =$ 555.5192 mm

$M_y^+ =$ 5.5542 kNm

$\therefore A_{sty}^+ =$ 108.4151 mm^2

$$Spacing_y^+ = \frac{1000}{(\frac{A_{sty}^+}{Area\ of\ one\ rebar})}$$

$Spacing_y^+ =$ 724.7278 mm

$M_y^- =$ 7.4585 kNm

$\therefore A_{sty}^- =$ 146.5341 mm^2

$$Spacing_y^- = \frac{1000}{(\frac{A_{sty}^-}{Area\ of\ one\ rebar})}$$

$Spacing_y^- =$ 536.1991 mm

At support of X-axis

Provide 10 mm diameter bar @ 200 c/c

At mid-span of X-axis

Provide 10 mm diameter bar @ 200 c/c

At support of Y-axis

Provide 10 mm diameter bar @ 200 c/c

At midspan of Y-axis

Provide 10 mm diameter bar @ 200 c/c

$\therefore A_{stx(provided)}^+ =$ 392.8571 mm^2

Step 7

Check for shear

$V_{max} = \frac{wl_{ex}}{2} =$ 20.7713 kN

$\tau = \frac{V_{max}}{bd} =$ 0.1598 N/mm^2

$100\frac{A_{stx(provided)}^+}{bd} =$ 0.3022 %

$\tau =$ 0.3871 **From table 19,**

Check for deflection

According to Cl.23.2.1, IS 456:2000

α= 26.0000

$\beta =$ 1.0000

$\gamma =$ 2.0000

$\lambda =$ 1.0000

$\delta =$ 1.0000

For γ

$100 \frac{A^{+}_{stx(provided)}}{bd} =$ 0.3022

$f_s = 0.58 * f_y * \frac{A_{st,req}}{A_{st,provided}} =$ 77.7363

$\gamma =$ 2.0000 From fig 4,

αβλγδ= 52.0000

$l/_d =$ 29.3846

$\therefore l/_d \leq \alpha\beta\lambda\gamma\delta$

Step 9
Check for development length
According to Cl.26.2.1, IS 456:2000

$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} =$ 485.4911

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

1E+07

$1.3 \frac{M_l}{V} =$ 653.1948

Assume l_0= 100.0000

$1.3 \frac{M_l}{V} + l_0 =$ 753.1948 m

$L_d \leq 1.3 \frac{M_l}{V} + l_0$ (ok)

Detailed slab calculation

S2

Two adjacent edge discontinuous

Step 1

Calculation of Effective depth

Short span length (l)	3.6150	m
long span length(l)	4.0900	m
l/d	35.8800	
Calculated Effective depth(d)	100.7525	
Effective depth(d)	130.0000	mm
∅	10.0000	mm
Clear cover	15.0000	mm
Calculated overall depth	150.0000	
Overall depth (D)	150.0000	
d_x	130.0000	mm
dy	120.0000	mm

Step 2

Calculation of Effective length

According to IS 456:2000, cl.22.2(a)

Along X-axis

i.	Clear span + eff depth	3.3450	m
ii,	c/c distance	3.6150	m
	l_{ex}	3.3450	m

According to IS 456:2000, cl.22.2(a)

Along Y-axis

i.	Clear span + eff depth	3.8100	m
ii,	c/c distance	4.0900	m
	l_{ey}	3.8100	m

$\frac{ley}{lex}$ 1.1390 **Two way slab**

Step 3

Load calculation

Live load	2.0000	kN/m
Floor finish	1.5000	kN/m
Partition wall	0.0000	kN/m
Self weight	3.7500	kN/m
Total Load	7.2500	kN/m
Factored load	10.8750	kN/m

Step 4

α_x^+	0.0419	**From IS**
α_x^-	0.0557	**456:2000 ,**
α_y^+	0.0350	**Annex D ,**
α_y^-	0.0470	**Table 26**
$M_x^+ = \alpha_x^+ \mathrm{w} l_x^2 =$	5.0984	kNm
$M_x^- = \alpha_x^- \mathrm{w} l_x^2 =$	6.7776	kNm
$M_y^+ = \alpha_y^+ \mathrm{w} l_x^2 =$	4.2588	kNm
$M_y^- = \alpha_y^- \mathrm{w} l_x^2 =$	5.7190	kNm

Step 5

Check for depth from moment criteria

f_{ck}	25.0000	
f_y	500.0000	
b	1000.0000	
M_{max}	6.7776	kNm
$M_{max} = 0.133 f_{ck} b d^2$		
$\therefore d_{req} =$	45.1484	mm

Step 6

Calculation of area of reinforcement and spacing

According to Cl 26.5.2.1,IS 456:2000

$A_{st,mim} = 0.12\% of bD =$ 180.0000 mm^2

Area of one rebar = 78.5714

According to Cl G.1.1,IS 456:2000

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

$M_x^+ =$ 5.0984 kNm

$\therefore A_{stx}^+ =$ 91.4906 mm^2

$$Spacing_x^+ = \frac{1000}{\left(\frac{A_{stx}^+}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^+$ 858.7926 mm

$M_x^- =$ 6.7776 kNm

$\therefore A_{stx}^- =$ 122.2093 mm^2

$$Spacing_x^- = \frac{1000}{\left(\frac{A_{stx}^-}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^- =$ 642.9251 mm

$M_y^+ =$ 4.2588 kNm

$$\therefore A_{sty}^{+} = \quad 82.7692 \text{ mm}^2$$

$$Spacing_y^{+} = \frac{1000}{\left(\frac{A_{sty}^{+}}{Area\ of\ one\ rebar}\right)}$$

$$Spacing_y^{+} = \quad 949.2831 \text{ mm}$$

$$M_y^{-} = \quad 5.7190 \text{ kNm}$$

$$\therefore A_{sty}^{-} = \quad 111.6932 \text{ mm}^2$$

$$Spacing_y^{-} = \frac{1000}{\left(\frac{A_{sty}^{-}}{Area\ of\ one\ rebar}\right)}$$

$$Spacing_y^{-} = \quad 703.4575 \text{ mm}$$

At support of X-axis

Provide 10 mm diameter bar @ 200 c/c

At mid-span of X-axis

Provide 10 mm diameter bar @ 200 c/c

At support of Y-axis

Provide 10 mm diameter bar @ 200 c/c

At midspan of Y-axis

Provide 10 mm diameter bar @ 200 c/c

$$\therefore A_{stx(provided)}^{+} = \quad 392.8571 \text{ mm}^2$$

Step 7

Check for shear

$$V_{max} = \frac{wl_{ex}}{2} = \quad 18.1884 \text{ kN}$$

$$\tau = \frac{V_{max}}{bd} = \quad 0.1399 \text{ N/mm}^2$$

$$100\frac{A_{stx(provided)}^{+}}{bd} = \quad 0.3022 \ \%$$

$$\tau = \quad 0.3871$$ **From table 19,**

Step 8

Check for deflection

According to Cl.23.2.1, IS 456:2000

$$\alpha = \quad 26.0000$$

$$\beta = \quad 1.0000$$

$$\gamma = \quad 2.0000$$

$\lambda =$ 1.0000

$\delta =$ 1.0000

For γ

$100\,\frac{A^{+}_{stx(provided)}}{bd} =$ 0.3022

$f_s = 0.58 * f_y * \frac{A_{st,req}}{A_{st,provided}} =$ 67.5367

$\gamma =$ 2.0000 From fig 4,

αβλγδ= 52.0000

$l/_d =$ 25.7308

$\therefore l/_d \leq \alpha\beta\lambda\gamma\delta$

Step 9
Check for development length
According to Cl.26.2.1, IS 456:2000

$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} =$

$M_l = 0.87 f_y * A_{st} * d * \left(1 - \frac{f_y * A_{st}}{f_{ck} * bd}\right) =$ 10436671

$1.3\frac{M_l}{V} =$ 745.9504

Assume l_0= 100.0000

$1.3\frac{M_l}{V} + l_0 =$ 845.9504

$L_d \leq 1.3\frac{M_l}{V} + l_0$ (ok)

<u>Detailed slab calculation</u>

S3

<u>One long edge discontinuous</u>

Step 1

Calculation of Effective depth

Short span length (l)	4.2450	m
long span length(l)	6.3500	m
l/d	35.8800	
Calculated Effective depth(d)	118.3110	
Effective depth(d)	130.0000	mm
∅	10.0000	mm
Clear cover	15.0000	mm
Calculated overall depth	150.0000	
Overall depth (D)	150.0000	
d_x	130.0000	mm
dy	120.0000	mm

Step 2

Calculation of Effective length

According to IS 456:2000, cl.22.2(a)

Along X-axis

i.	Clear span + eff depth	4.0250	m
ii,	c/c distance	4.2450	m
	l_{ex}	4.0250	m

According to IS 456:2000, cl.22.2(a)

Along Y-axis

i.	Clear span + eff depth	6.0700	m
ii,	c/c distance	6.3500	m
	l_{ey}	6.0700	m

$\frac{ley}{lex}$ 1.5081 **Two way slab**

Step 3

Load calculation

Live load	2.0000	kN/m
Floor finish	1.5000	kN/m
Partition wall	0.0000	kN/m
Self weight	3.7500	kN/m
Total Load	7.2500	kN/m
Factored load	10.8750	kN/m

Step 4

α_x^+	0.0513	**From IS**
α_x^-	0.0673	**456:2000 ,**
α_y^+	0.0280	**Annex D ,**
α_y^-	0.0370	**Table 26**
$M_x^+ = \alpha_x^+ \mathrm{w} l_x^2 =$	9.0381	kNm
$M_x^- = \alpha_x^- \mathrm{w} l_x^2 =$	11.8613	kNm
$M_y^+ = \alpha_y^+ \mathrm{w} l_x^2 =$	4.9331	kNm
$M_y^- = \alpha_y^- \mathrm{w} l_x^2 =$	6.5187	kNm

Step 5

Check for depth from moment criteria

f_{ck}	25.0000	
f_y	500.0000	
b	1000.0000	
M_{max}	11.8613	kNm
$M_{max} = 0.133 f_{ck} b d^2$		
$\therefore d_{req} =$	59.7269	mm

Step 6

Calculation of area of reinforcement and spacing

According to Cl 26.5.2.1,IS 456:2000

$A_{st,mim} = 0.12\% of bD =$ 180.0000 mm^2

Area of one rebar = 78.5714

According to Cl G.1.1,IS 456:2000

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

$M_x^+ =$ 9.0381 kNm

$\therefore A_{stx}^+ =$ 164.0454 mm^2

$$Spacing_x^+ = \frac{1000}{\left(\frac{A_{stx}^+}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^+$ 478.9613 mm

$M_x^- =$ 11.8613 kNm

$\therefore A_{stx}^- =$ 217.1046 mm^2

$$Spacing_x^- = \frac{1000}{\left(\frac{A_{stx}^-}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^- =$ 361.9059 mm

$M_y^+ =$ 4.9331 kNm

$$\therefore A_{sty}^{+} = \quad 96.0898 \text{ mm}^2$$

$$Spacing_y^{+} = \frac{1000}{\left(\frac{A_{sty}^{+}}{Area\ of\ one\ rebar}\right)}$$

$$Spacing_y^{+} = \quad 817.6877 \text{ mm}$$

$$M_y^{-} = \quad 6.5187 \text{ kNm}$$

$$\therefore A_{sty}^{-} = \quad 127.6584 \text{ mm}^2$$

$$Spacing_y^{-} = \frac{1000}{\left(\frac{A_{sty}^{-}}{Area\ of\ one\ rebar}\right)}$$

$$Spacing_y^{-} = \quad 615.4820 \text{ mm}$$

At support of X-axis

Provide 10 mm diameter bar @ 200 c/c

At mid-span of X-axis

Provide 10 mm diameter bar @ 200 c/c

At support of Y-axis

Provide 10 mm diameter bar @ 200 c/c

At midspan of Y-axis

Provide 10 mm diameter bar @ 200 c/c

$$\therefore A_{stx(provided)}^{+} = \quad 392.8571 \text{ mm}^2$$

Step 7

Check for shear

$$V_{max} = \frac{wl_{ex}}{2} = \quad 21.8859 \text{ kN}$$

$$\tau = \frac{V_{max}}{bd} = \quad 0.1684 \text{ N/mm}^2$$

$$100\frac{A_{stx(provided)}^{+}}{bd} = \quad 0.3022 \%$$

$$\tau = \quad 0.3871$$ **From table 19, IS**

Step 8

Check for deflection

According to Cl.23.2.1, IS

456:2000

α=	26.0000
$\beta =$	1.0000
$\gamma =$	2.0000
$\lambda =$	1.0000
$\delta =$	1.0000

For γ

$100\,\frac{A^{+}_{stx(provided)}}{bd} =$	0.3022	
$f_s = 0.58 * f_y * \frac{A_{st,req}}{A_{st,provided}} =$	121.0954	
$\gamma =$	2.0000	From fig 4,
αβλγδ=	52.0000	
$l/_d =$	30.9615	

$\therefore l/_d \leq \alpha\beta\lambda\gamma\delta$

Step 9

Check for development length

According to Cl.26.2.1, IS 456:2000

$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} =$ 485.4911

$M_l = 0.87 f_y * A_{st} * d * \left(1 - \frac{f_y * A_{st}}{f_{ck} * bd}\right) =$ 10436670.9

$1.3\frac{M_l}{V} =$ 619.9265

Assume l_0= 100.0000

$1.3\frac{M_l}{V} + l_0 =$ 719.9265

$L_d \leq 1.3\frac{M_l}{V} + l_0$ (ok)

<u>Detailed slab calculation</u>

S5

a

<u>Two adjacent edge discontinuous</u>

Step 1

Calculation of Effective depth

Short span length (l)	2.4550	m
long span length(l)	2.4967	m
l/d	35.8800	
Calculated Effective depth(d)	68.4225	
Effective depth(d)	130.0000	mm
∅	10.0000	mm
Clear cover	15.0000	mm
Calculated overall depth	150.0000	
Overall depth (D)	150.0000	
d_x	130.0000	mm
dy	120.0000	mm

Step 2

Calculation of Effective length

According to IS 456:2000, cl.22.2(a)

Along X-axis

i.	Clear span + eff depth	2.2350	m
ii,	c/c distance	2.4550	m
	l_{ex}	2.2350	m

According to IS 456:2000, cl.22.2(a)

Along Y-axis

i.	Clear span + eff depth	2.2167	m
ii,	c/c distance	2.4967	m
	l_{ey}	2.2167	m

$\frac{ley}{lex}$ 0.9918 **Two way slab**

Step 3

Load calculation

Live load	2.0000	kN/m
Floor finish	1.5000	kN/m
Partition wall	4.0600	kN/m
Self weight	3.7500	kN/m
Total Load	11.3100	
Factored load	16.9650	

Step 4

α_x^+	0.0349	**From IS**
α_x^-	0.0446	**456:2000 ,**
α_y^+	0.0350	**Annex D ,**
α_y^-	0.0470	**Table 26**
$M_x^+ = \alpha_x^+ \mathrm{w} l_x^2 =$	2.9604	kNm
$M_x^- = \alpha_x^- \mathrm{w} l_x^2 =$	3.7796	kNm
$M_y^+ = \alpha_y^+ \mathrm{w} l_x^2 =$	2.9660	kNm
$M_y^- = \alpha_y^- \mathrm{w} l_x^2 =$	3.9830	kNm

Step 5

Check for depth from moment criteria

f_{ck}	25.0000	
f_y	500.0000	
b	1000.0000	
M_{max}	3.9830	
$M_{max} = 0.133 f_{ck} b d^2$		
$\therefore d_{req} =$	34.6105	mm

Step 6

Calculation of area of reinforcement and spacing

According to Cl 26.5.2.1,IS 456:2000

$A_{st,mim} = 0.12\% of bD =$	180.0000	mm^2
Area of one rebar =	78.5714	

According to Cl G.1.1,IS 456:2000

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

$M_x^+ =$	2.9604	kNm
$\therefore A_{stx}^+ =$	52.8054	mm^2

$$Spacing_x^+ = \frac{1000}{\left(\frac{A_{stx}^+}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^+$	1487.9439	mm
$M_x^- =$	3.7796	kNm
$\therefore A_{stx}^- =$	67.5720	mm^2

$$Spacing_x^- = \frac{1000}{\left(\frac{A_{stx}^-}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^- =$	1162.7811	mm
$M_y^+ =$	2.9660	kNm

$$\therefore A^{+}_{sty} = \quad 57.3982 \text{ mm}^2$$

$$Spacing^{+}_{y} = \frac{1000}{\left(\frac{A^{+}_{sty}}{Area\ of\ one\ rebar}\right)}$$

$$Spacing^{+}_{y} = \quad 1368.8835 \text{ mm}$$

$$M^{-}_{y} = \quad 3.9830 \text{ kNm}$$

$$\therefore A^{-}_{sty} = \quad 77.3370 \text{ mm}^2$$

$$Spacing^{-}_{y} = \frac{1000}{\left(\frac{A^{-}_{sty}}{Area\ of\ one\ rebar}\right)}$$

$$Spacing^{-}_{y} = \quad 1015.9610 \text{ mm}$$

At support of X-axis

Provide 10 mm diameter bar @ 200 c/c

At mid-span of X-axis

Provide 10 mm diameter bar @ 200 c/c

At support of Y-axis

Provide 10 mm diameter bar @ 200 c/c

At midspan of Y-axis

Provide 10 mm diameter bar @ 200 c/c

$$\therefore A^{+}_{stx(provided)} = \quad 392.8571 \text{ mm}^2$$

Step 7

Check for shear

$$V_{max} = \frac{wl_{ex}}{2} = \quad 18.9584 \text{ kN}$$

$$\tau = \frac{V_{max}}{bd} = \quad 0.1458 \text{ N/mm}^2$$

$$100\frac{A^{+}_{stx(provided)}}{bd} = \quad 0.3022 \%$$

$$\tau = \quad 0.3871$$ **From table 19,**

Step 8

Check for deflection

According to Cl.23.2.1, IS 456:2000

$$\alpha = \quad 26.0000$$

$$\beta = \quad 1.0000$$

$$\gamma = \quad 2.0000$$

$\lambda =$ 1.0000

$\delta =$ 1.0000

For γ

$100\frac{A^{+}_{stx(provided)}}{bd} =$ 0.3022

$f_s = 0.58 * f_y * \frac{A_{st,req}}{A_{st,provided}} =$ 38.9800

$\gamma =$ 2.0000 From fig 4,

$\alpha\beta\lambda\gamma\delta =$ 52.0000

$l/_d =$ 17.1923

$\therefore l/_d \leq \alpha\beta\lambda\gamma\delta$

Step 9

Check for development length

According to Cl.26.2.1, IS 456:2000

$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} =$ 485.4911

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

10436671

$1.3\frac{M_l}{V} =$ 715.6554

Assume $l_0 =$ 100.0000

$1.3\frac{M_l}{V} + l_0 =$ 815.6554

$L_d \leq 1.3\frac{M_l}{V} + l_0$ (ok)

Detailed slab calculation

S5

b

One long edge discontinuous

Step 1

Calculation of Effective depth

Short span length (l)	2.4550	m
long span length(l)	2.4967	m
l/d	35.8800	
Calculated Effective depth(d)	68.4225	
Effective depth(d)	130.0000	mm
∅	10.0000	mm
Clear cover	15.0000	mm
Calculated overall depth	150.0000	
Overall depth (D)	150.0000	
d_x	130.0000	mm
dy	120.0000	mm

Step 2

Calculation of Effective length

According to IS 456:2000, cl.22.2(a)

Along X-axis

i.	Clear span + eff depth	2.2350	m
ii,	c/c distance	2.4550	m
	l_{ex}	2.2350	m

According to IS 456:2000, cl.22.2(a)

Along Y-axis

i.	Clear span + eff depth	2.2167	m
ii,	c/c distance	2.4967	m
	l_{ey}	2.2167	m

$\frac{ley}{lex}$ 0.9918 **Two way slab**

Step 3

Load calculation

Live load	2.0000	kN/m
Floor finish	1.5000	kN/m
Partition wall	4.0600	kN/m
Self weight	3.7500	kN/m
Total Load	11.3100	
Factored load	16.9650	

Step 4

α_x^+	0.0280	**From IS**
α_x^-	0.0370	**456:2000 ,**
α_y^+	0.0280	**Annex D ,**
α_y^-	0.0370	**Table 26**
$M_x^+ = \alpha_x^+ \mathrm{w} l_x^2 =$	2.3728	kNm
$M_x^- = \alpha_x^- \mathrm{w} l_x^2 =$	3.1355	kNm
$M_y^+ = \alpha_y^+ \mathrm{w} l_x^2 =$	2.3728	kNm
$M_y^- = \alpha_y^- \mathrm{w} l_x^2 =$	3.1355	kNm

Step 5

Check for depth from moment criteria

f_{ck}	25.0000	
f_y	500.0000	
b	1000.0000	
M_{max}	3.1355	kNm
$M_{max} = 0.133 f_{ck} b d^2$		
$\therefore d_{req} =$	30.7086	ok

Step 6

Calculation of area of reinforcement and spacing

According to Cl 26.5.2.1,IS 456:2000

$A_{st,mim} = 0.12\% of bD =$	180.0000	mm^2
Area of one rebar =	78.5714	

According to Cl G.1.1,IS 456:2000

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

$M_x^+ =$	2.3728	kNm
$\therefore A_{stx}^+ =$	42.2556	mm^2

$$Spacing_x^+ = \frac{1000}{\left(\frac{A_{stx}^+}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^+$	1859.4338	mm
$M_x^- =$	3.1355	kNm
$\therefore A_{stx}^- =$	55.9564	mm^2

$$Spacing_x^- = \frac{1000}{\left(\frac{A_{stx}^-}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^- =$	1404.1537	mm
$M_y^+ =$	2.3728	kNm

$\therefore A^+_{sty} =$ 45.8293 mm^2

$$Spacing^+_y = \frac{1000}{\left(\frac{A^+_{sty}}{Area\ of\ one\ rebar}\right)}$$

$Spacing^+_y =$ 1714.4354 mm

$M^-_y =$ 3.1355 kNm

$\therefore A^-_{sty} =$ 60.7119 mm^2

$$Spacing^-_y = \frac{1000}{\left(\frac{A^-_{sty}}{Area\ of\ one\ rebar}\right)}$$

$Spacing^-_y =$ 1294.1677 mm

At support of X-axis

Provide	10 mm	diameter	bar @	200 c/c

At mid-span of X-axis

Provide	10 mm	diameter	bar @	200 c/c

At support of Y-axis

Provide	10 mm	diameter	bar @	200 c/c

At midspan of Y-axis

Provide	10 mm	diameter	bar @	200 c/c

$\therefore A^+_{stx(provided)} =$ 392.8571 mm^2

Step 7

Check for shear

$V_{max} = \frac{wl_{ex}}{2} =$ 18.9584 kN

$\tau = \frac{V_{max}}{bd} =$ 0.1458 N/mm^2

$100\frac{A^+_{stx(provided)}}{bd} =$ 0.3022 %

$\tau =$ 0.3871 **From table 19,**

Step 8

Check for deflection

According to Cl.23.2.1, IS 456:2000

$\alpha =$ 26.0000

$\beta =$ 1.0000

$\gamma =$ 2.0000

$\lambda =$ 1.0000

$\delta =$ 1.0000

For γ

$100\frac{A^{+}_{stx(provided)}}{bd} =$ 0.3022

$f_s = 0.58 * f_y * \frac{A_{st,req}}{A_{st,provided}} =$ 31.1923

$\gamma =$ 2.0000 From fig 4,

$\alpha\beta\lambda\gamma\delta =$ 52.0000

$l/_d =$ 17.1923

$\therefore l/_d \leq \alpha\beta\lambda\gamma\delta$ ok

Step 9

Check for development length

According to Cl.26.2.1, IS 456:2000

$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} =$ 485.4911

$M_l = 0.87 f_y * A_{st} * d * \left(1 - \frac{f_y * A_{st}}{f_{ck} * bd}\right) =$ 10436671

$1.3\frac{M_l}{V} =$ 715.6554

Assume $l_0 =$ 100.0000

$1.3\frac{M_l}{V} + l_0 =$ 815.6554

$L_d \leq 1.3\frac{M_l}{V} + l_0$ ok

Detailed slab calculation

S5

c

Two adjacent edge discontinuous

Step 1

Calculation of Effective depth

Short span length (l)	2.4550	m
long span length(l)	2.4967	m
l/d	35.8800	
Calculated Effective depth(d)	68.4225	
Effective depth(d)	130.0000	mm
∅	10.0000	mm
Clear cover	15.0000	mm
Calculated overall depth	150.0000	
Overall depth (D)	150.0000	
d_x	130.0000	mm
dy	120.0000	mm

Step 2

Calculation of Effective length

According to IS 456:2000, cl.22.2(a)

Along X-axis

i.	Clear span + eff depth	2.2350	m
ii,	c/c distance	2.4550	m
	l_{ex}	2.2350	m

According to IS 456:2000, cl.22.2(a)

Along Y-axis

i.	Clear span + eff depth	2.2167	m
ii,	c/c distance	2.4967	m
	l_{ey}	2.2167	m

$\frac{ley}{lex}$ 0.9918 **Two way slab**

Step 3

Load calculation

Live load	2.0000	kN/m
Floor finish	1.5000	kN/m
Partition wall	4.0600	kN/m
Self weight	3.7500	kN/m
Total Load	11.3100	
Factored load	16.9650	

Step 4

α_x^+	0.0349	**From IS**
α_x^-	0.0446	**456:2000**
α_y^+	0.0350	**, Annex**
α_y^-	0.0470	**D , Table**
$M_x^+ = \alpha_x^+ \mathrm{w} l_x^2 =$	2.9604	kNm
$M_x^- = \alpha_x^- \mathrm{w} l_x^2 =$	3.7796	kNm
$M_y^+ = \alpha_y^+ \mathrm{w} l_x^2 =$	2.9660	kNm
$M_y^- = \alpha_y^- \mathrm{w} l_x^2 =$	3.9830	kNm

Step 5

Check for depth from moment criteria

f_{ck}	25.0000	
f_y	500.0000	
b	1000.0000	
M_{max}	3.9830	kNm
$M_{max} = 0.133 f_{ck} b d^2$		
$\therefore d_{req} =$	34.6105	ok

Step 6

Calculation of area of reinforcement and spacing

According to Cl 26.5.2.1,IS 456:2000

$A_{st,mim} = 0.12\% of bD =$ 180.0000 mm^2

Area of one rebar = 78.5714

According to Cl G.1.1,IS 456:2000

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

$M_x^+ =$ 2.9604 kNm

$\therefore A_{stx}^+ =$ 52.8054 mm^2

$$Spacing_x^+ = \frac{1000}{\left(\frac{A_{stx}^+}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^+$ 1487.9439 mm

$M_x^- =$ 3.7796 kNm

$\therefore A_{stx}^- =$ 67.5720 mm^2

$$Spacing_x^- = \frac{1000}{\left(\frac{A_{stx}^-}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^- =$ 1162.7811 mm

$M_y^+ =$ 2.9660 kNm

$\therefore A^{+}_{sty} =$ 57.3982 mm^2

$$Spacing^{+}_{y} = \frac{1000}{\left(\frac{A^{+}_{sty}}{Area\ of\ one\ rebar}\right)}$$

$Spacing^{+}_{y} =$ 1368.8835 mm

$M^{-}_{y} =$ 3.9830 kNm

$\therefore A^{-}_{sty} =$ 77.3370 mm^2

$$Spacing^{-}_{y} = \frac{1000}{\left(\frac{A^{-}_{sty}}{Area\ of\ one\ rebar}\right)}$$

$Spacing^{-}_{y} =$ 1015.9610 mm

At support of X-axis

Provide	10 mm	diameter	bar @	200 c/c

At mid-span of X-axis

Provide	10 mm	diameter	bar @	200 c/c

At support of Y-axis

Provide	10 mm	diameter	bar @	200 c/c

At midspan of Y-axis

Provide	10 mm	diameter	bar @	200 c/c

$\therefore A^{+}_{stx(provided)} =$ 392.8571 mm^2

Step 7
Check for shear

$V_{max} = \frac{wl_{ex}}{2} =$ 18.9584 kN

$\tau = \frac{V_{max}}{bd} =$ 0.1458 N/mm^2

$100\frac{A^{+}_{stx(provided)}}{bd} =$ 0.3022 %

$\tau =$ 0.3871 **From table 19,**

Step 8
Check for deflection
According to Cl.23.2.1, IS 456:2000

$\alpha =$ 26.0000

$\beta =$ 1.0000

$\gamma =$ 2.0000

$\lambda =$ 1.0000

$\delta =$ 1.0000

For γ

$100\frac{A^{+}_{stx(provided)}}{bd} =$ 0.3022

$f_s = 0.58 * f_y * \frac{A_{st,req}}{A_{st,provided}} =$ 38.9800

$\gamma =$ 2.0000 From fig 4,

αβλγδ= 52.0000

$l/_d =$ 17.1923

$\therefore l/_d \leq \alpha\beta\lambda\gamma\delta$ ok

Step 9
Check for development length
According to Cl.26.2.1, IS 456:2000

$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} =$ 485.4911

$M_l = 0.87 f_y * A_{st} * d * \left(1 - \frac{f_y * A_{st}}{f_{ck} * bd}\right) =$ 1E+07

$1.3\frac{M_l}{V} =$ 715.6554

Assume l_0= 100.0000

$1.3\frac{M_l}{V} + l_0 =$ 815.6554

$L_d \leq 1.3\frac{M_l}{V} + l_0$ ok

<u>Detailed slab calculation</u>

S5

d

<u>Two adjacent edge discontinuous</u>

Step 1

Calculation of Effective depth

Short span length (l)	2.4550	m
long span length(l)	2.4967	m
l/d	35.8800	
Calculated Effective depth(d)	68.4225	
Effective depth(d)	130.0000	mm
∅	10.0000	mm
Clear cover	15.0000	mm
Calculated overall depth	150.0000	
Overall depth (D)	150.0000	
d_x	130.0000	mm
dy	120.0000	mm

Step 2

Calculation of Effective length

According to IS 456:2000, cl.22.2(a)

Along X-axis

i.	Clear span + eff depth	2.2350	m
ii,	c/c distance	2.4550	m
	l_{ex}	2.2350	m

According to IS 456:2000, cl.22.2(a)

Along Y-axis

i.	Clear span + eff depth	2.2167	m
ii,	c/c distance	2.4967	m
	l_{ey}	2.2167	m

$\frac{ley}{lex}$ 0.9918 **Two way slab**

Step 3

Load calculation

Live load	2.0000	kN/m
Floor finish	1.5000	kN/m
Partition wall	4.0600	kN/m
Self weight	3.7500	kN/m
Total Load	11.3100	kN/m
Factored load	16.9650	kN/m

Step 4

α_x^+	0.0240	**From IS 456:2000 ,**
α_x^-	0.0320	**Annex D , Table 26**
α_y^+	0.0240	
α_y^-	0.0320	
$M_x^+ = \alpha_x^+ \mathrm{w} l_x^2 =$	2.0339	kNm
$M_x^- = \alpha_x^- \mathrm{w} l_x^2 =$	2.7118	kNm
$M_y^+ = \alpha_y^+ \mathrm{w} l_x^2 =$	2.0339	kNm
$M_y^- = \alpha_y^- \mathrm{w} l_x^2 =$	2.7118	kNm

Step 5

Check for depth from moment criteria

f_{ck}	25.0000	
f_y	500.0000	
b	1000.0000	
M_{max}	2.7118	kNm
$M_{max} = 0.133 f_{ck} b d^2$		
$\therefore d_{req} =$	28.5584	ok

Step 6

Calculation of area of reinforcement and spacing

According to Cl 26.5.2.1,IS 456:2000

$A_{st,mim} = 0.12\% of bD =$ 180.0000 mm^2

Area of one rebar = 78.5714

According to Cl G.1.1,IS 456:2000

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

$M_x^+ =$ 2.0339 kNm

$\therefore A_{stx}^+ =$ 36.1850 mm^2

$$Spacing_x^+ = \frac{1000}{\left(\frac{A_{stx}^+}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^+$ 2171.3787 mm

$M_x^- =$ 2.7118 kNm

$\therefore A_{stx}^- =$ 48.3376 mm^2

$$Spacing_x^- = \frac{1000}{\left(\frac{A_{stx}^-}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^- =$ 1625.4722 mm

$M_y^+ =$ 2.0339 kNm

$$\therefore A_{sty}^{+} =$$ 39.2389 mm^2

$$Spacing_y^{+} = \frac{1000}{\left(\frac{A_{sty}^{+}}{Area\ of\ one\ rebar}\right)}$$

$Spacing_y^{+} =$ 2002.3886 mm

$M_y^{-} =$ 2.7118 kNm

$$\therefore A_{sty}^{-} =$$ 52.4345 mm^2

$$Spacing_y^{-} = \frac{1000}{\left(\frac{A_{sty}^{-}}{Area\ of\ one\ rebar}\right)}$$

$Spacing_y^{-} =$ 1498.4669 mm

At support of X-axis

Provide 10 mm diameter bar @ 200 c/c

At mid-span of X-axis

Provide 10 mm diameter bar @ 200 c/c

At support of Y-axis

Provide 10 mm diameter bar @ 200 c/c

At midspan of Y-axis

Provide 10 mm diameter bar @ 200 c/c

$\therefore A_{stx(provided)}^{+} =$ 392.8571 mm^2

Step 7

Check for shear

$V_{max} = \frac{wl_{ex}}{2} =$ 18.9584 kN

$\tau = \frac{V_{max}}{bd} =$ 0.1458 N/mm^2

$100\frac{A_{stx(provided)}^{+}}{bd} =$ 0.3022 %

$\tau =$ 0.3871 **From table 19, IS 456:2000**

Step 8

Check for deflection

According to Cl.23.2.1, IS 456:2000

α= 26.0000

$\beta =$ 1.0000

$\gamma =$ 2.0000

$\lambda =$	1.0000	
$\delta =$	1.0000	
For γ		
$100\frac{A^{+}_{stx(provided)}}{bd} =$	0.3022	
$f_s = 0.58 * f_y * \frac{A_{st,req}}{A_{st,provided}} =$	26.7111	
$\gamma =$	2.0000	From fig 4,
$\alpha\beta\lambda\gamma\delta=$	52.0000	
$l/_d =$	17.1923	
$\therefore l/_d \leq \alpha\beta\lambda\gamma\delta$		ok

Step 9
Check for development length
According to Cl.26.2.1, IS 456:2000

$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} =$ 485.4911

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

10436670.92

$1.3\frac{M_l}{V} =$ 715.6554

Assume l_0= 100.0000

$1.3\frac{M_l}{V} + l_0 =$ 815.6554

$L_d \leq 1.3\frac{M_l}{V} + l_0$ ok

Detailed slab calculation

S6

b

Two adjacent edge discontinuous

Step 1

Calculation of Effective depth

Short span length (l)	2.4550	m
long span length(l)	2.4967	m
l/d	35.8800	
Calculated Effective depth(d)	68.4225	
Effective depth(d)	130.0000	mm
∅	10.0000	mm
Clear cover	15.0000	mm
Calculated overall depth	150.0000	
Overall depth (D)	150.0000	
d_x	130.0000	mm
dy	120.0000	mm

Step 2

Calculation of Effective length

According to IS 456:2000, cl.22.2(a)

Along X-axis

i.	Clear span + eff depth	2.2350	m
ii,	c/c distance	2.4550	m
	l_{ex}	2.2350	m

According to IS 456:2000, cl.22.2(a)

Along Y-axis

i.	Clear span + eff depth	2.2167	m
ii,	c/c distance	2.4967	m
	l_{ey}	2.2167	m

$\frac{ley}{lex}$ 0.9918 **Two way slab**

Step 3

Load calculation

Live load	2.0000	kN/m
Floor finish	1.5000	kN/m
Partition wall	4.0600	kN/m
Self weight	3.7500	kN/m
Total Load	11.3100	kN/m
Factored load	16.9650	kN/m

Step 4

α_x^+	0.0240	**From IS 456:2000 ,**
α_x^-	0.0320	**Annex D , Table 26**
α_y^+	0.0240	
α_y^-	0.0320	
$M_x^+ = \alpha_x^+ \mathrm{w} l_x^2 =$	2.0339	kNm
$M_x^- = \alpha_x^- \mathrm{w} l_x^2 =$	2.7118	kNm
$M_y^+ = \alpha_y^+ \mathrm{w} l_x^2 =$	2.0339	kNm
$M_y^- = \alpha_y^- \mathrm{w} l_x^2 =$	2.7118	kNm

Step 5

Check for depth from moment criteria

f_{ck}	25.0000	
f_y	500.0000	
b	1000.0000	
M_{max}	2.7118	kNm

$$M_{max} = 0.133 f_{ck} b d^2$$

$\therefore d_{req} =$	28.5584	ok

Step 6

Calculation of area of reinforcement and spacing

According to Cl 26.5.2.1,IS 456:2000

$A_{st,mim} = 0.12\% of bD =$	180.0000	mm^2
Area of one rebar =	78.5714	

According to Cl G.1.1,IS 456:2000

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

$M_x^+ =$	2.0339	kNm
$\therefore A_{stx}^+ =$	36.1850	mm^2

$$Spacing_x^+ = \frac{1000}{\left(\frac{A_{stx}^+}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^+$	2171.3787	mm

$M_x^- =$	2.7118	kNm
$\therefore A_{stx}^- =$	48.3376	mm^2

$$Spacing_x^- = \frac{1000}{\left(\frac{A_{stx}^-}{Area\ of\ one\ rebar}\right)}$$

$Spacing_x^- =$	1625.4722	mm

$M_y^+ =$ 2.0339 kNm

$\therefore A_{sty}^+ =$ 39.2389 mm^2

$$Spacing_y^+ = \frac{1000}{\left(\frac{A_{sty}^+}{Area\ of\ one\ rebar}\right)}$$

$Spacing_y^+ =$ 2002.3886 mm

$M_y^- =$ 2.7118 kNm

$\therefore A_{sty}^- =$ 52.4345 mm^2

$$Spacing_y^- = \frac{1000}{\left(\frac{A_{sty}^-}{Area\ of\ one\ rebar}\right)}$$

$Spacing_y^- =$ 1498.4669 mm

At support of X-axis

Provide 10 mm diameter bar @ 200 c/c

At mid-span of X-axis

Provide 10 mm diameter bar @ 200 c/c

At support of Y-axis

Provide 10 mm diameter bar @ 200 c/c

At midspan of Y-axis

Provide 10 mm diameter bar @ 200 c/c

$\therefore A_{stx(provided)}^+ =$ 392.8571 mm^2

Step 7

Check for shear

$V_{max} = \frac{wl_{ex}}{2} =$ 18.9584 kN

$\tau = \frac{V_{max}}{bd} =$ 0.1458 N/mm^2

$100\frac{A_{stx(provided)}^+}{bd} =$ 0.3022 %

$\tau =$ 0.3871 **From table 19, IS 456:2000**

Step 8

Check for deflection

According to Cl.23.2.1, IS 456:2000

$\alpha=$ 26.0000

$\beta =$ 1.0000

$\gamma =$ 2.0000

$\lambda =$ 1.0000

$\delta =$ 1.0000

For γ

$100\frac{A^{+}_{stx(provided)}}{bd} =$ 0.3022

$f_s = 0.58 * f_y * \frac{A_{st,req}}{A_{st,provided}} =$ 26.7111

$\gamma =$ 2.0000 From fig 4,

$\alpha\beta\lambda\gamma\delta=$ 52.0000

$l/_d =$ 17.1923

$\therefore l/_d \leq \alpha\beta\lambda\gamma\delta$ ok

Step 9

Check for development length

According to Cl.26.2.1, IS 456:2000

$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} =$ 485.4911

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

10436670.92

$1.3\frac{M_l}{V} =$ 715.6554

Assume $l_0=$ 100.0000

$1.3\frac{M_l}{V} + l_0 =$ 815.6554

$L_d \leq 1.3\frac{M_l}{V} + l_0$ ok

Cantilever Slab

Span length C/C (Lex) =	1.2	m	In X-direction
Span length C/C (Ley) =	2.56	m	In Y-direction

We have,

$$\frac{L_{efx}}{d} = 14$$

So, d =	85.71	mm
Adopt d =	130	mm
φ=	10	mm
cc=	15	mm
D =	150	mm

Now,

d_x =	130	mm
d_y =	120	mm

Load Calculation:

Self weight =	3.75	KN/m^2
Wall Load	3.26	KN/m^2
Live Load (L.L.) =	3	KN/m^2
Floor Finish (F.F.) =	1.5	KN/m^2
Total Load =	11.51	KN/m^2
Factored Load (F.L.) =	17.265	KN/m^2
Consider unit width,w=	17.265	KN/m

$$\frac{L_y}{L_x} = 2.1333 > 2$$

(Design as One Way Slab)

Checking for depth from Moment Consideration

M_{max}=	2.0718	KN-m
b=	1000	mm
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
φ=	10	mm
A_b=	78.54	mm^2

IS 456-2000 Annex G.1.1

$$\text{depth(d)} = \sqrt{\frac{M_{max}}{0.133 * fck * b}} = 24.962 \text{ mm}$$

< dx OK

Calculation of Area of Steel

Min A_{st} = 0.12 % of bD = 180 mm^2

IS 456-2000 G-1.1.b

$$A_{st} = 0.5\frac{f_{ck}}{f_y}\left(1-\sqrt{1-\frac{4.6M_u}{f_{ck}bd^2}}\right)bd$$

$$S_v = \frac{A_b * 1000}{Ast}$$

IS 456-2000 Cl. 26.3.3, b,1

Smax =	300	mm
3*deff	390	mm

We get,

Mmax =	2.0718	KN-m		
A_{st} =	36.864	mm^2	< 150 mm^2	**Not OK**

So, take Ast = 150 mm2

S_v=	523.6	mm	> 300mm, > 3d

Provide 10mm φ bar @ 200 mm C/C

S_v=	200	mm
A_{st} provided =	392.7	mm^2

Check for Deflection

IS 456-2000 cl.23.2.1

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

L_x=	1.2	mm
α=	7	
β=	1	span less than 10 m
γ=	1	no compression reinforcement
δ=	1	not a flanged section

For λ

$$fs = 0.58\ f_y \frac{\text{Area of Steel Required}}{\text{Area of Steel Provided}}$$

Area of steel required=	36.864	mm^2
Area of steel provided=	392.7	mm^2

So,

fs= 27.223 N/mm^2

%st= 0.374 %

IS 456-2000 cl.23.2.1 fig 4

λ= 2

So,

αβγδλ= 14

$\frac{L_x}{d}$= 9.2308 ≤ αβγδλ **(OK)**

Check for shear:

$$Vu = \frac{w * Lx}{2} = \quad 10.359 \text{ KN}$$

IS 456-2000 cl.40.1

$$\tau v = \frac{Vu}{b * d} = \quad 0.0797 \ N/mm^2$$

IS 456-2000 cl.40.2.1.1

Overall Depth 150 mm

k= 1.3

IS 456-2000 cl.40.4 Table 19

Now,

% steel =	0.374
τ_c for M25	0.4245

Design Shear Strength (Kτc)= 0.5518 N/mm^2 **OK**

Check for Development Length:

IS 456-2000 cl.26.2.1

$$Ld = \frac{\Phi \sigma_s}{1.6 \ x \ 4 \ x \ \tau_{bd}} = \quad 485.49 \text{ mm}$$

Also,

$$L_d \leq 1.3 \frac{M_1}{V} + L_o$$

Where, $M_1 = 0.87 * f_y * 0.5 * A_{stprvd} * (d - \frac{f_y * A_{stprvd}}{f_{ck} * b * 2})$

M_1=	1E+07	N-mm
V=	10359	N
L_o=	185	mm

So,

$1.3 * \frac{M_1}{V} + L_o =$ 1536.3 **> Ld OK**

Design of Column

Columns are the vertical members that are subjected to axial loads and moment acting from two directions (biaxial). All columns are subjected to some moment which may be due to accidental eccentricity or due to end restraint imposed by monolithically placed beams or slabs. The strength of

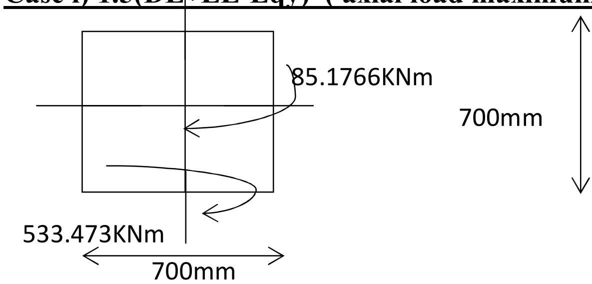
a) Axially Loaded Column
b) A Column Subjected to Axial Load and Uni-axial Bending
c) A Column Subjected to Axial Load and Bi-axial Bending

The design of the column section for given axial load and biaxial bending moments can be made by pre assigning the section and checking accuracy of assumption. The design of the column depends upon the eccentricity of loading and the moment acting in different directions

FLEXURAL ANALYSIS OF COLUMN

For column (C15) **Storey 1**

Case i) 1.5(DL+LL-Eqy) (axial load maximum case)



Unsupported Length(Lx)=	2650	mm
Unsupported Length(Ly)=	2650	mm
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	5283.069	KN

Calculated previously

M_{uy}=	533.4725	KN-m
M_{ux}=	85.1766	KN-m

Check for Axial Stress:

IS 13920:2016 cl.7.1.1

$0.08*f_{ck}$=	2	N/mm^2	
Factored Axial Load(P_u) =	5283.069	KN	
Factored Axial Stress $\frac{P_u}{B*D}$=	10.782	N/mm^2	> 3 N/mm^2

Hence, design as Column Member

Calculated Previously

$\frac{l_{effx}}{l}=$ 0.899153

$\frac{l_{effy}}{l}=$ 0.949372

So,

l_{effx}=	2382.755	mm
l_{effy}=	2515.836	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x=\frac{l_{effx}}{l}=$	3.40	**<12 Short Column**
$\lambda_x=\frac{l_{effy}}{l}=$	3.59	**<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

28.63333333	mm	**>20 OK**
28.63333333	mm	**>20 OK**

Also,

0.05*D =	35	mm	**>e_{minx}**
0.05*B =	35	mm	**>e_{miny}**

$M_{minx}= P_u*e_{minx}$
= 151.2718757 KN-m

$M_{miny}= P_u*e_{miny}$
= 151.2718757 KN-m

So,

M_{ux}=	533.4725	KN-m	(maximum of M_{ux} and M_{minx})
M_{uy}=	151.2718757	KN-m	(maximum of M_{uy} and M_{miny})

Check for minimun eccentricity

$e_{minx}=\frac{l}{500}+\frac{D}{30}=$ **31.96667** **ok**

$e_{minx}=\frac{l}{500}+\frac{D}{30}=$ **28.63333** **ok**

Since, S.R. <12, short column.

Now,

Assume p%= 1.20%

$$\frac{d'}{D} = 0.075$$

$$\frac{d'}{B} = 0.075$$

$$\frac{p}{f_{ck}} = 0.048$$

$$\frac{P_u}{f_{ck}BD} = 0.43$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = 0.085$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = 0.085$$

Thus,

$M_{ux,1} =$ 728.875 KN-m

$M_{uy,1} =$ 728.875 KN-m

IS 456 : 2000 cl.39.6

$$P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}$$

We get,

$P_{uz} =$ 7651.35 KN

$$\frac{P_u}{P_{uz}} = 0.690$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.690	1.817

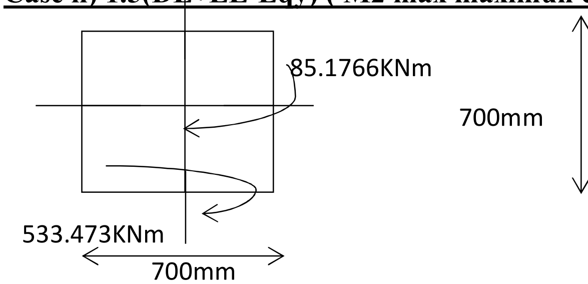
Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = 0.587 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

Case ii) 1.5(DL+LL-Eqy) (M2 max maximun case)



Unsupported Length(Lx)=	2650	m
Unsupported Length(Ly)=	2650	
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	5283.069	KN

Calculated previously

M_{uy}=	533.4725	KN-m
M_{ux}=	85.1766	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) =	5283.069	KN	
Factored Axial Stress $\frac{P_u}{B*D}$=	10.782	N/mm^2	> 3 N/mm^2

Hence, design as Column Member

Calculated Previously

$$\frac{l_{effx}}{l} = 0.8992$$

$$\frac{l_{effy}}{l} = 0.949372$$

So,

l_{effx}=	2382.755	mm
l_{effy}=	2515.836	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l} =$ 3.40 **<12 Short Column**

$\lambda_x = \frac{l_{effy}}{l} =$ 3.59 **<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

28.63333333 mm **>20 OK**

28.63333333 mm **>20 OK**

Also,

0.05*D = 35 mm **>e_{minx}**

0.05*B = 35 mm **>e_{miny}**

$M_{minx} = P_u * e_{minx}$

= 151.2718757 KN-m

$M_{miny} = P_u * e_{miny}$

= 151.2718757 KN-m

So,

M_{ux}= 533.4725 KN-m (maximum of M_{ux} and M_{minx})

M_{uy}= 151.2718757 KN-m (maximum of M_{uy} and M_{miny})

Check for minimun eccentricity

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **31.96667** **ok**

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **28.63333** **ok**

Since, S.R. <12 short column.

Now,

Assume p%= 1.20%

$\frac{d'}{D} =$ 0.075

$\frac{d'}{B} =$ 0.075

$\frac{p}{f_{ck}} =$ 0.048

$$\frac{P_u}{f_{ck}BD} = \quad 0.43$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = \quad 0.085$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = \quad 0.085$$

Thus,

$M_{ux,1}$ =	728.875	KN-m
$M_{uy,1}$ =	728.875	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} = 7651.35 KN

$$\frac{P_u}{P_{uz}} = \quad 0.690$$

IS 456 : 2000 CL 39.6

For,

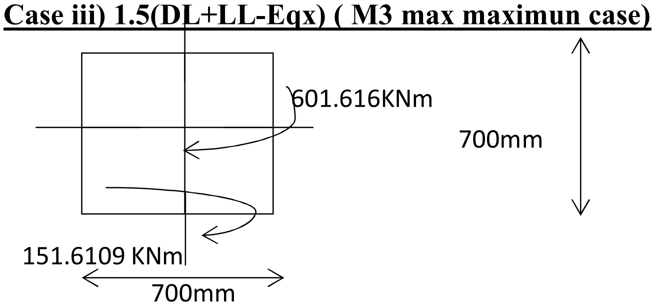
P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.690	1.817

Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = \quad 0.587 \quad \textbf{OK}$$

Area of Steel (A_{sc})=	5880	mm^2
Area of each Bar (A_b) =	490.88	mm^2

<u>Case iii) 1.5(DL+LL-Eqx) (M3 max maximun case)</u>



Unsupported Length(Lx)= 2650 m

Unsupported Length(Ly)= 2650

Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	4527.683	KN

Calculated previously

M_{uy}=	151.6109	KN-m
M_{ux}=	601.6155	KN-m

Check for Axial Stress:

IS 13920:2016 cl.7.1.1

$0.08*f_{ck}$=	2.4	N/mm^2
Factored Axial Load(P_u) =	4527.683	KN

Factored Axial Stress

$\frac{P_u}{B*D}$= 9.240 N/mm^2 > 3 N/mm^2

Hence, design as Column Member

Calculated Previously

$\frac{l_{effx}}{l}$= 0.8992

$\frac{l_{effy}}{l}$= 0.949372

So,

l_{effx}=	2382.755	mm
l_{effy}=	2515.836	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x=\frac{l_{effx}}{l}$ = 3.40 **<12 Short Column**

$\lambda_x=\frac{l_{effy}}{l}$= 3.59 **<12 Short Column**

l= Least lateral dimension

Check for minimun eccentricity

$e_{minx}=\frac{l}{500}+\frac{D}{30}$= **31.96667** **ok**

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **28.63333** **ok**

Since, S.R. <12 short column.

Now,

Assume p%= **1.20%**

$\frac{d'}{D} =$ 0.075

$\frac{d'}{B} =$ 0.075

$\frac{p}{f_{ck}} =$ 0.048

$\frac{P_u}{f_{ck}BD} =$ 0.37

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$\frac{M_{ux,1}}{f_{ck}BD^2} =$ 0.09

$\frac{M_{uy,1}}{f_{ck}DB^2} =$ 0.12

Thus,

$M_{ux,1} =$ 771.75 KN-m

$M_{uy,1} =$ 1029 KN-m

IS 456 : 2000 cl.39.6

$P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}$

We get,

$P_{uz} =$ 7651.35 KN

$\frac{P_u}{P_{uz}} =$ 0.592

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.592	1.653

Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n}+\left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = \quad 0.705 \quad \textbf{OK}$$

Area of Steel (A_{sc})=	5880	mm^2
Area of each Bar (A_b) =	490.88	mm^2

Among three cases Pmax,M2 max & M3 max, minimun value of longitudinal reinforcement is obtained in all three cases. So we adopt the minimum longitudinal reinforcement for column.

From Etabs

Area of Steel (A_{sc})= 5880 mm^2

Provide 12-25 φmm bars equally distributed on four faces.

% A_{sc} provided =	1.2	%	Range = 0.8% - 6%	**OK**
A_{sc} provided =	5890.486225	mm^2		

For Transverse Reinforcement,

IS 456 : 2000 CL 26.5.3.2

Diameter,

$\varphi_t \geq$	6	mm	6 mm
$\varphi_t \geq$	6.25	mm	1/4 of φ_l

Adopt,

$\varphi_t =$ 8 mm

Pitch should not be more than

1)	700	mm	least lateral dimension
2)	400	mm	$16*\varphi_l$
3)	300	mm	300mm

Provide 8mm φ bar @ 200 mm C/C

Spacing of longitudinal bars

In X-direction,

Space between end bars =	595	mm	> $48*\varphi_t$(384)
Space between bars =	148.75	mm	>75mm

In Y-direction,

Space between end bars =	595	mm	> $48*\varphi_t$(384)
Space between bars =	148.75	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 close type

Design of Column

Columns are the vertical members that are subjected to axial loads and moment acting from two directions (biaxial). All columns are subjected to

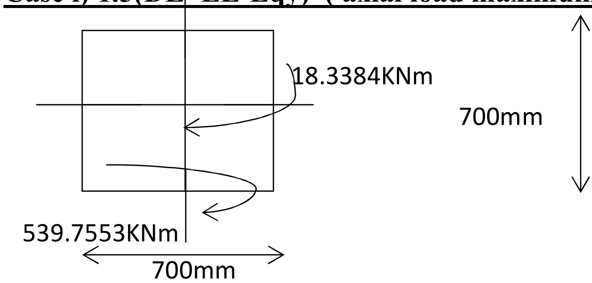
a) Axially Loaded Column
b) A Column Subjected to Axial Load and Uni-axial Bending
c) A Column Subjected to Axial Load and Bi-axial Bending

The design of the column section for given axial load and biaxial bending moments can be made by pre assigning the section and checking accuracy

FLEXURAL ANALYSIS OF COLUMN

<u>For column (C18)</u> **Storey 1**

<u>Case i) 1.5(DL+LL-Eqy) (axial load maximum case)</u>



Unsupported Length(Lx)=	2650	mm
Unsupported Length(Ly)=	2650	mm
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	3040.446	KN

<u>Calculated previously</u>

M_{uy}=	539.7553	KN-m
M_{ux}=	18.3384	KN-m

<u>Check for Axial Stress:</u>

IS 13920:2016 cl.7.1.1

$0.08*f_{ck}$=	2	N/mm^2	
Factored Axial Load(P_u) =	3040.446	KN	
Factored Axial Stress			
$\frac{P_u}{B*D}$=	6.205	N/mm^2	> 3 N/mm^2

Hence, design as Column Member

<u>Calculated Previously</u>

$\frac{l_{effx}}{l}$= 0.934819

$\frac{l_{effy}}{l} = \boxed{0.927082}$

So,

l_{effx}=	2477.270	mm
l_{effy}=	2456.767	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l} =$	3.54	**<12 Short Column**
$\lambda_x = \frac{l_{effy}}{l} =$	3.51	**<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

28.63333333	mm	**>20 OK**
28.63333333	mm	**>20 OK**

Also,

0.05*D =	35	mm	**>e_{minx}**
0.05*B =	35	mm	**>e_{miny}**

M_{minx}= P_u*e_{minx}

= 87.05809521 KN-m

M_{miny}= P_u*e_{miny}

= 87.05809521 KN-m

So,

M_{ux}=	539.7553	KN-m	(maximum of M_{ux} and M_{minx})
M_{uy}=	87.05809521	KN-m	(maximum of M_{uy} and M_{miny})

Check for minimun eccentricity

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **31.96667** **ok**

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **28.63333** **ok**

Since, S.R. <12, short column.

Now,

Assume p%= **1.20%**

$$\frac{d'}{D} = 0.075$$

$$\frac{d'}{B} = 0.075$$

$$\frac{p}{f_{ck}} = 0.048$$

$$\frac{P_u}{f_{ck}BD} = 0.25$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = 0.12$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = 0.12$$

Thus,

$M_{ux,1}$ = 1029 KN-m

$M_{uy,1}$ = 1029 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} = 7651.35 KN

$$\frac{P_u}{P_{uz}} = 0.397$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		**α_n**
<=	0.2	1
>=	0.8	2
	0.397	1.329

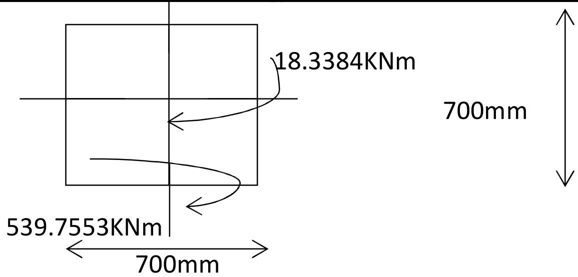
Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = 0.429 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

Case ii) 1.5(DL+LL-Eqy) (M2 max maximun case)



Unsupported Length(Lx)=	2650	m
Unsupported Length(Ly)=	2650	
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	3040.446	KN

Calculated previously

M_{uy}=	539.7553	KN-m
M_{ux}=	18.3384	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) =	3040.446	KN

Factored Axial Stress

$\frac{P_u}{B*D}$= 6.205 N/mm^2 > 3 N/mm^2

Hence, design as Column Member

Calculated Previously

$\frac{l_{effx}}{l}$= 0.9348

$\frac{l_{effy}}{l}$= 0.927082

So,

l_{effx}=	2477.270	mm
l_{effy}=	2456.767	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l} =$	3.54	**<12 Short Column**
$\lambda_x = \frac{l_{effy}}{l} =$	3.51	**<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

28.63333333	mm	**>20 OK**
28.63333333	mm	**>20 OK**

Also,

0.05*D =	35	mm	**>e_{minx}**
0.05*B =	35	mm	**>e_{miny}**

M_{minx}= P_u*e_{minx}
= 87.05809521 KN-m

M_{miny}= P_u*e_{miny}
= 87.05809521 KN-m

So,

M_{ux}=	539.7553	KN-m	(maximum of M_{ux} and M_{minx})
M_{uy}=	87.05809521	KN-m	(maximum of M_{uy} and M_{miny})

Check for minimun eccentricity

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$	**31.96667**	**ok**
$e_{minx} = \frac{l}{500} + \frac{D}{30} =$	**28.63333**	**ok**

Since, S.R. <12 short column.

Now,

Assume p%= 1.20%

$\frac{d'}{D} =$ 0.075

$\frac{d'}{B} =$ 0.075

$\frac{p}{f_{ck}} =$ 0.048

$$\frac{P_u}{f_{ck}BD} = \quad 0.25$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = \quad 0.12$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = \quad 0.12$$

Thus,

$M_{ux,1}$ = 1029 KN-m

$M_{uy,1}$ = 1029 KN-m

IS 456 : 2000 cl.39.6

P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}

We get,

P_{uz} = 7651.35 KN

$$\frac{P_u}{P_{uz}} = \quad 0.397$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.397	1.329

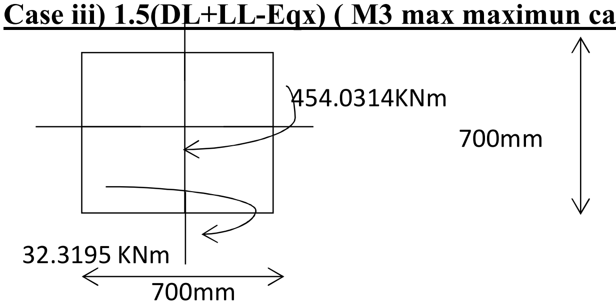
Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = \quad 0.429 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

Case iii) 1.5(DL+LL-Eqx) (M3 max maximun case)



Unsupported Length(Lx)=	2650	m
Unsupported Length(Ly)=	2650	
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	2685.586	KN

Calculated previously

M_{uy}=	32.3195	KN-m
M_{ux}=	454.0314	KN-m

Check for Axial Stress:

IS 13920:2016 cl.7.1.1

$0.08*f_{ck}$=	2.4	N/mm^2
Factored Axial Load(P_u) =	2685.586	KN

Factored Axial Stress

$$\frac{P_u}{B*D} = 5.481 \; N/mm^2 \quad > 3\ N/mm^2$$

Hence, design as Column Member

Calculated Previously

$$\frac{l_{effx}}{l} = 0.9348$$

$$\frac{l_{effy}}{l} = 0.927082$$

So,

l_{effx}=	2477.270	mm
l_{effy}=	2456.767	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l} =$ 3.54 **<12 Short Column**

$\lambda_x = \frac{l_{effy}}{l} =$ 3.51 **<12 Short Column**

l= Least lateral dimension

Check for minimun eccentricity

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **31.96667** **ok**

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **28.63333** **ok**

Since, S.R. <12 short column.

Now,

Assume p%= **1.20%**

$\frac{d'}{D} =$ 0.075

$\frac{d'}{B} =$ 0.075

$\frac{p}{f_{ck}} =$ 0.048

$\frac{P_u}{f_{ck}BD} =$ 0.22

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$\frac{M_{ux,1}}{f_{ck}BD^2} =$ 0.12

$\frac{M_{uy,1}}{f_{ck}DB^2} =$ 0.12

Thus,

$M_{ux,1} =$ 1029 KN-m

$M_{uy,1} =$ 1029 KN-m

IS 456 : 2000 cl.39.6

$P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}$

We get,

$P_{uz} =$ 7651.35 KN

$$\frac{P_u}{P_{uz}} = \quad 0.351$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.351	1.252

Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = \quad 0.372 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

Among three cases Pmax,M2 max & M3 max, minimun value of longitudinal reinforcement is obtained in all three cases. So we adopt the minimum longitudinal reinforcement for column.

From Etabs

Area of Steel (A_{sc})= 5880 mm^2

Provide 12-25 φmm bars equally distributed on four faces.

% A_{sc} provided = 1.2 % Range = 0.8% - 6% **OK**

A_{sc} provided = 5890.486225 mm^2

For Transverse Reinforcement,

IS 456 : 2000 CL 26.5.3.2

Diameter,

$\varphi_t \geq$ 6 mm 6 mm

$\varphi_t \geq$ 6.25 mm 1/4 of φ_l

Adopt,

$\varphi_t =$ 8 mm

Pitch should not be more than

1) 700 mm least lateral dimension

2) 400 mm 16*φ_l

3) 300 mm 300mm

Provide 8mm φ bar @ 200 mm C/C

Spacing of longitudinal bars

In X-direction,

Space between end bars =	595	mm	> 48*φ_t(384)
Space between bars =	148.75	mm	>75mm

In Y-direction,

Space between end bars =	595	mm	> 48*φ_t(384)
Space between bars =	148.75	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 close type

Design of Column

Columns are the vertical members that are subjected to axial loads and moment acting from two directions (biaxial). All columns are subjected to

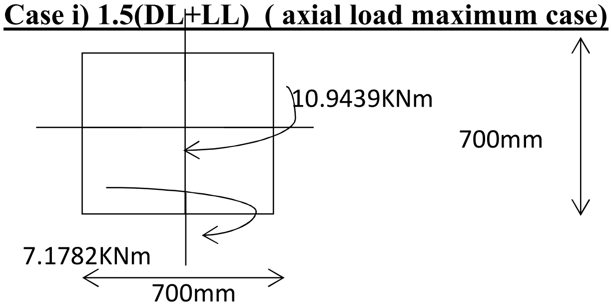
a) Axially Loaded Column
b) A Column Subjected to Axial Load and Uni-axial Bending
c) A Column Subjected to Axial Load and Bi-axial Bending

The design of the column section for given axial load and biaxial bending moments can be made by pre assigning the section and checking

FLEXURAL ANALYSIS OF COLUMN

For column (C21) **Storey 5**

Case i) 1.5(DL+LL) (axial load maximum case)



Unsupported Length(Lx)=	2650	mm
Unsupported Length(Ly)=	2650	mm
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	1926.791	KN

Calculated previously

M_{uy}=	7.1782	KN-m
M_{ux}=	10.9493	KN-m

Check for Axial Stress:

IS 13920:2016 cl.7.1.1

$0.08*f_{ck}$=	2	N/mm^2
Factored Axial Load(P_u) =	1926.791	KN

Factored Axial Stress

$\frac{P_u}{B*D}$ = 3.932 N/mm^2 > 3 N/mm^2

Hence, design as Column Member

Calculated Previously

$\frac{l_{effx}}{l}=$ 0.890518

$\frac{l_{effy}}{l}=$ 0.896396

So,

$l_{effx}=$	2359.873	mm
$l_{effy}=$	2375.449	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x=\frac{l_{effx}}{l}=$	3.37	**<12 Short Column**
$\lambda_x=\frac{l_{effy}}{l}=$	3.39	**<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

28.63333333	mm	**>20 OK**
28.63333333	mm	**>20 OK**

Also,

0.05*D =	35	mm	**>e_{minx}**
0.05*B =	35	mm	**>e_{miny}**

$M_{minx}= P_u*e_{minx}$

= 55.1704461 KN-m

$M_{miny}= P_u*e_{miny}$

= 55.1704461 KN-m

So,

$M_{ux}=$	55.1704461	KN-m	(maximum of M_{ux} and M_{minx})
$M_{uy}=$	55.1704461	KN-m	(maximum of M_{uy} and M_{miny})

Check for minimun eccentricity

$e_{minx}=\frac{l}{500}+\frac{D}{30}=$ **31.96667** **ok**

$e_{minx}=\frac{l}{500}+\frac{D}{30}=$ **28.63333** **ok**

Since, S.R. <12, short column.

Now,

Assume p%= 1.20%

$$\frac{d'}{D} = 0.075$$

$$\frac{d'}{B} = 0.075$$

$$\frac{p}{f_{ck}} = 0.048$$

$$\frac{P_u}{f_{ck}BD} = 0.16$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = 0.11$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = 0.11$$

Thus,

$M_{ux,1} =$ 943.25 KN-m

$M_{uy,1} =$ 943.25 KN-m

IS 456 : 2000 cl.39.6

$P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}$

We get,

$P_{uz} =$ 7651.35 KN

$$\frac{P_u}{P_{uz}} = 0.252$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.252	1.086

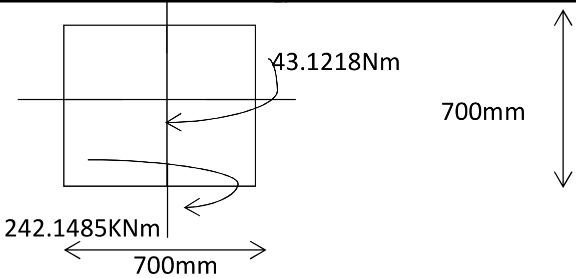
Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = 0.013 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

Case ii) 1.5(DL+LL+Eqy) (M2 max maximun case)



Unsupported Length(Lx)=	2650	m
Unsupported Length(Ly)=	2650	
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	1559.259	KN

<u>Calculated previously</u>

M_{uy}=	242.1485	KN-m
M_{ux}=	43.1218	KN-m

<u>Check for Axial Stress:</u>

IS 13920:1993 cl.7.1.1

$0.08*f_{ck}$=	2.4	N/mm^2
Factored Axial Load(P_u) =	1559.259	KN

Factored Axial Stress

$\frac{P_u}{B*D}$= 3.182 N/mm^2 > 3 N/mm^2

Hence, design as Column Member

<u>Calculated Previously</u>

$\frac{l_{effx}}{l}$= 0.8905

$\frac{l_{effy}}{l}$= 0.896396

So,

l_{effx}=	2359.873	mm
l_{effy}=	2375.449	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l} =$	3.37	**<12 Short Column**
$\lambda_x = \frac{l_{effy}}{l} =$	3.39	**<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

28.63333333	mm	**>20 OK**
28.63333333	mm	**>20 OK**

Also,

0.05*D =	35	mm	**>e_{minx}**
0.05*B =	35	mm	**>e_{miny}**

M_{minx}= P_u*e_{minx}

= 55.1704461 KN-m

M_{miny}= P_u*e_{miny}

= 44.64677984 KN-m

So,

M_{ux}=	242.1485	KN-m	(maximum of M_{ux} and M_{minx})
M_{uy}=	44.64677984	KN-m	(maximum of M_{uy} and M_{miny})

Check for minimun eccentricity

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **31.96667** **ok**

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **28.63333** **ok**

Since, S.R. <12 short column.

Now,

Assume p%= 1.20%

$\frac{d'}{D} =$ 0.075

$\frac{d'}{B} =$ 0.075

$$\frac{p}{f_{ck}} = \quad 0.048$$

$$\frac{P_u}{f_{ck}BD} = \quad 0.13$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = \quad 0.108$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = \quad 0.108$$

Thus,

$M_{ux,1} =$ 926.1 KN-m

$M_{uy,1} =$ 926.1 KN-m

IS 456 : 2000 cl.39.6

$P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}$

We get,

$P_{uz} =$ 7651.35 KN

$$\frac{P_u}{P_{uz}} = \quad 0.204$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.204	1.006

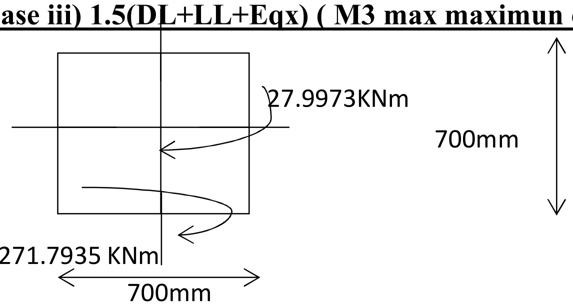
Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = \quad 0.305 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

Case iii) 1.5(DL+LL+Eqx) (M3 max maximun case)



Unsupported Length(Lx)=	2650	m
Unsupported Length(Ly)=	2650	
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	1493.116	KN

Calculated previously

M_{uy}=	27.9973	KN-m
M_{ux}=	271.7935	KN-m

Check for Axial Stress:

IS 13920:2016 cl.7.1.1

$0.08*f_{ck}$=	2.4	N/mm^2
Factored Axial Load(P_u) =	1493.116	KN

Factored Axial Stress

$\frac{P_u}{B*D}$= 3.047 N/mm^2 > 3 N/mm^2

Hence, design as Column Member

Calculated Previously

$\frac{l_{effx}}{l}$= 0.8905

$\frac{l_{effy}}{l}$= 0.896396

So,

l_{effx}=	2359.873	mm
l_{effy}=	2375.449	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l} =$ 3.37 **<12 Short Column**

$\lambda_x = \frac{l_{effy}}{l} =$ 3.39 **<12 Short Column**

l= Least lateral dimension

Check for minimun eccentricity

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **31.96667** **ok**

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **28.63333** **ok**

Since, S.R. <12 short column.

Now,

Assume p%= **1.20%**

$\frac{d'}{D} =$ 0.075

$\frac{d'}{B} =$ 0.075

$\frac{p}{f_{ck}} =$ 0.048

$\frac{P_u}{f_{ck}BD} =$ 0.12

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$\frac{M_{ux,1}}{f_{ck}BD^2} =$ 0.105

$\frac{M_{uy,1}}{f_{ck}DB^2} =$ 0.105

Thus,

$M_{ux,1} =$ 900.375 KN-m

$M_{uy,1} =$ 900.375 KN-m

IS 456 : 2000 cl.39.6

$P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}$

We get,

P_{uz} = 7651.35 KN

$$\frac{P_u}{P_{uz}} = 0.195$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.195	1.000

Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = 0.333 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

Among three cases Pmax,M2 max & M3 max, minimun value of longitudinal reinforcement is obtained in all three cases. So we adopt the minimum longitudinal reinforcement for column.

From Etabs

Area of Steel (A_{sc})= 5880 mm^2

Provide 12-25 φmm bars equally distributed on four faces.

% A_{sc} provided = 1.2 % Range = 0.8% - 6% **OK**

A_{sc} provided = 5890.486225 mm^2

For Transverse Reinforcement,

IS 456 : 2000 CL 26.5.3.2

Diameter,

$\varphi_t \geq$ 6 mm 6 mm

$\varphi_t \geq$ 6.25 mm 1/4 of φ_l

Adopt,

φ_t = 8 mm

Pitch should not be more than

1) 700 mm least lateral dimension

2) 400 mm $16*\varphi_l$

3) 300 mm 300mm

Provide 8mm φ bar @ 200 mm C/C

Spacing of longitudinal bars

In X-direction,

Space between end bars =	595	mm	> 48*φ_t(384)
Space between bars =	148.75	mm	>75mm

In Y-direction,

Space between end bars =	595	mm	> 48*φ_t(384)
Space between bars =	148.75	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 close type

Design of Column

Columns are the vertical members that are subjected to axial loads and moment acting from two directions (biaxial). All columns are subjected to

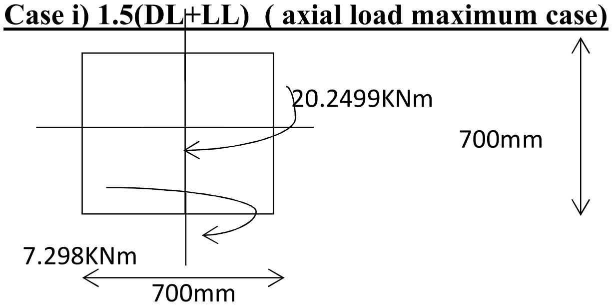
a) Axially Loaded Column
b) A Column Subjected to Axial Load and Uni-axial Bending
c) A Column Subjected to Axial Load and Bi-axial Bending

The design of the column section for given axial load and biaxial bending moments can be made by pre assigning the section and checking

FLEXURAL ANALYSIS OF COLUMN

For column (C24) **Storey BML**

Case i) 1.5(DL+LL) (axial load maximum case)



Unsupported Length(Lx)=	2650	mm
Unsupported Length(Ly)=	2650	mm
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	3696.462	KN

Calculated previously

M_{uy}=	7.298	KN-m
M_{ux}=	20.2499	KN-m

Check for Axial Stress:

IS 13920:2016 cl.7.1.1

$0.08*f_{ck}$=	2	N/mm^2
Factored Axial Load(P_u) =	3696.462	KN

Factored Axial Stress

$\frac{P_u}{B*D}$ = 7.544 N/mm^2 > 3 N/mm^2

Hence, design as Column Member

Calculated Previously

$\frac{l_{effx}}{l}$ = 0.780591

$\frac{l_{effy}}{l}$ = 0.792569

So,

l_{effx}=	2068.566	mm
l_{effy}=	2100.308	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l}$ =	2.96	**<12 Short Column**
$\lambda_x = \frac{l_{effy}}{l}$ =	3.00	**<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

28.63333333	mm	**>20 OK**
28.63333333	mm	**>20 OK**

Also,

0.05*D =	35	mm	**>e_{minx}**
0.05*B =	35	mm	**>e_{miny}**

M_{minx}= P_u*e_{minx}
= 105.8420257 KN-m

M_{miny}= P_u*e_{miny}
= 105.8420257 KN-m

So,

M_{ux}=	105.8420257	KN-m	(maximum of M_{ux} and M_{minx})
M_{uy}=	105.8420257	KN-m	(maximum of M_{uy} and M_{miny})

Check for minimun eccentricity

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **31.96667** **ok**

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **28.63333** **ok**

Since, S.R. <12, short column.

Now,

Assume p%= **1.20%**

$$\frac{d'}{D} = \quad 0.075$$

$$\frac{d'}{B} = \quad 0.075$$

$$\frac{p}{f_{ck}} = \quad 0.048$$

$$\frac{P_u}{f_{ck}BD} = \quad 0.30$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = \quad 0.1$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = \quad 0.1$$

Thus,

$M_{ux,1} =$ 857.5 KN-m

$M_{uy,1} =$ 857.5 KN-m

IS 456 : 2000 cl.39.6

P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}

We get,

$P_{uz} =$ 7651.35 KN

$$\frac{P_u}{P_{uz}} = \quad 0.483$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.483	1.472

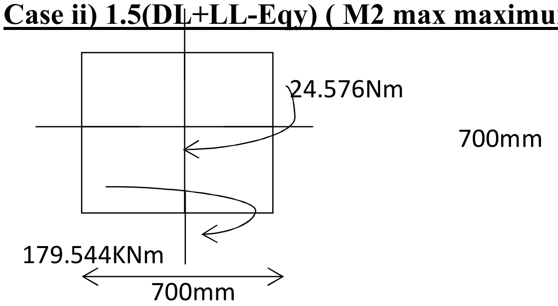
Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = \quad 0.005 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

Case ii) 1.5(DL+LL-Eqy) (M2 max maximun case)



Unsupported Length(Lx)=	2650	m
Unsupported Length(Ly)=	2650	
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	3140.254	KN

Calculated previously

M_{uy}=	179.544	KN-m
M_{ux}=	24.576	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) =	3140.254	KN

Factored Axial Stress

$\frac{P_u}{B*D}$= 6.409 N/mm^2 > 3 N/mm^2

Hence, design as Column Member

Calculated Previously

$\frac{l_{effx}}{l}$= 0.7806

$\frac{l_{effy}}{l}$= 0.792569

So,

l_{effx}=	2068.566	mm
l_{effy}=	2100.308	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l} =$	2.96	**<12 Short Column**
$\lambda_x = \frac{l_{effy}}{l} =$	3.00	**<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

28.63333333	mm	**>20 OK**
28.63333333	mm	**>20 OK**

Also,

0.05*D =	35	mm	**>e_{minx}**
0.05*B =	35	mm	**>e_{miny}**

$M_{minx} = P_u * e_{minx}$

= 105.8420257 KN-m

$M_{miny} = P_u * e_{miny}$

= 89.91593381 KN-m

So,

M_{ux}=	179.544	KN-m	(maximum of M_{ux} and M_{minx})
M_{uy}=	89.91593381	KN-m	(maximum of M_{uy} and M_{miny})

Check for minimun eccentricity

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **31.96667** **ok**

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **28.63333** **ok**

Since, S.R. <12 short column.

Now,

Assume p%= 1.20%

$\frac{d'}{D} =$ 0.075

$\frac{d'}{B} =$ 0.075

$$\frac{p}{f_{ck}} = \quad 0.048$$

$$\frac{P_u}{f_{ck}BD} = \quad 0.26$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = \quad 0.11$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = \quad 0.11$$

Thus,

$M_{ux,1}$ = 943.25 KN-m

$M_{uy,1}$ = 943.25 KN-m

IS 456 : 2000 cl.39.6

P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}

We get,

P_{uz} = 7651.35 KN

$$\frac{P_u}{P_{uz}} = \quad 0.410$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.410	1.351

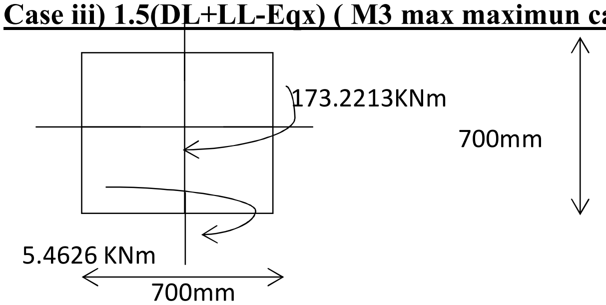
Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = \quad 0.114 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

<u>Case iii) 1.5(DL+LL-Eqx) (M3 max maximun case)</u>



Unsupported Length(Lx)=	2650	m
Unsupported Length(Ly)=	2650	
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	3170.272	KN

Calculated previously

M_{uy}=	5.4626	KN-m
M_{ux}=	173.2213	KN-m

Check for Axial Stress:

IS 13920:2016 cl.7.1.1

$0.08*f_{ck}$=	2.4	N/mm^2
Factored Axial Load(P_u) =	3170.272	KN

Factored Axial Stress

$\frac{P_u}{B*D}$= 6.470 N/mm^2 > 3 N/mm^2

Hence, design as Column Member

Calculated Previously

$\frac{l_{effx}}{l}$= 0.7806

$\frac{l_{effy}}{l}$= 0.792569

So,

l_{effx}=	2068.566	mm
l_{effy}=	2100.308	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x=\frac{l_{effx}}{l}$ = 2.96 **<12 Short Column**

$\lambda_x=\frac{l_{effy}}{l}$= 3.00 **<12 Short Column**

l= Least lateral dimension

Check for minimun eccentricity

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **31.96667** **ok**

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **28.63333** **ok**

Since, S.R. <12 short column.

Now,

Assume p%= **1.20%**

$\frac{d'}{D} =$ 0.075

$\frac{d'}{B} =$ 0.075

$\frac{p}{f_{ck}} =$ 0.048

$\frac{P_u}{f_{ck}BD} =$ 0.26

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$\frac{M_{ux,1}}{f_{ck}BD^2} =$ 0.11

$\frac{M_{uy,1}}{f_{ck}DB^2} =$ 0.11

Thus,

$M_{ux,1} =$ 943.25 KN-m

$M_{uy,1} =$ 943.25 KN-m

IS 456 : 2000 cl.39.6

$\mathbf{P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}}$

We get,

$P_{uz} =$ 7651.35 KN

$\frac{P_u}{P_{uz}} =$ 0.414

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.414	1.357

Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = \quad 0.101 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

Among three cases Pmax,M2 max & M3 max, minimun value of longitudinal reinforcement is obtained in all three cases. So we adopt the minimum longitudinal reinforcement for column.

From Etabs

Area of Steel (A_{sc})= 5880 mm^2

Provide 12-25 φmm bars equally distributed on four faces.

% A_{sc} provided = 1.2 % Range = 0.8% - 6% **OK**

A_{sc} provided = 5890.486225 mm^2

For Transverse Reinforcement,

IS 456 : 2000 CL 26.5.3.2

Diameter,

$\varphi_t \geq$	6	mm	6 mm
$\varphi_t \geq$	6.25	mm	1/4 of φ_l

Adopt,

φ_t = 8 mm

Pitch should not be more than

1)	700	mm	least lateral dimension
2)	400	mm	$16*\varphi_l$
3)	300	mm	300mm

Provide 8mm φ bar @ 200 mm C/C

Spacing of longitudinal bars

In X-direction,

Space between end bars =	595	mm	> $48*\varphi_t$(384)
Space between bars =	148.75	mm	>75mm

In Y-direction,

Space between end bars =	595	mm	> 48*φ_t(384)
Space between bars =	148.75	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 close type

Design of Column

Columns are the vertical members that are subjected to axial loads and moment acting from two directions (biaxial). All columns are subjected to

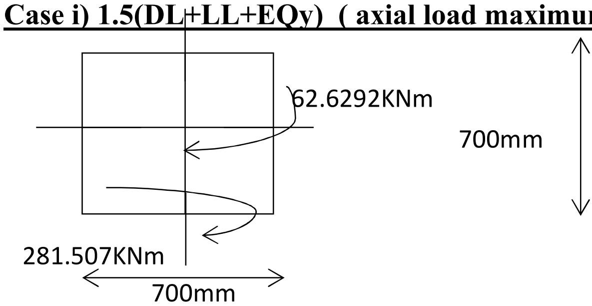
a) Axially Loaded Column
b) A Column Subjected to Axial Load and Uni-axial Bending
c) A Column Subjected to Axial Load and Bi-axial Bending

The design of the column section for given axial load and biaxial bending moments can be made by pre assigning the section and checking

FLEXURAL ANALYSIS OF COLUMN

For column (C31) **Storey 2**

Case i) 1.5(DL+LL+EQy) (axial load maximum case)



Unsupported Length(Lx)=	2650	mm
Unsupported Length(Ly)=	2650	mm
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	2564.830	KN

Calculated previously

M_{uy}=	281.5071	KN-m
M_{ux}=	62.6292	KN-m

Check for Axial Stress:

IS 13920:2016 cl.7.1.1

$0.08*f_{ck}$=	2	N/mm^2
Factored Axial Load(P_u) =	2564.830	KN

Factored Axial Stress

$$\frac{P_u}{B*D} = 5.234 \ N/mm^2 \quad > 3\ N/mm^2$$

Hence, design as Column Member

Calculated Previously

$$\frac{l_{effx}}{l} = 0.942735$$

$$\frac{l_{effy}}{l} = 0.941886$$

So,

l_{effx}=	2498.248	mm
l_{effy}=	2495.998	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l}$ =	3.57	**<12 Short Column**
$\lambda_x = \frac{l_{effy}}{l}$ =	3.57	**<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

28.63333333	mm	**>20 OK**
28.63333333	mm	**>20 OK**

Also,

0.05*D =	35	mm	**>e_{minx}**
0.05*B =	35	mm	**>e_{miny}**

M_{minx}= P_u*e_{minx}

= 73.43961802 KN-m

M_{miny}= P_u*e_{miny}

= 73.43961802 KN-m

So,

M_{ux}=	281.5071	KN-m	(maximum of M_{ux} and M_{minx})
M_{uy}=	73.43961802	KN-m	(maximum of M_{uy} and M_{miny})

Check for minimun eccentricity

$$e_{minx} = \frac{l}{500} + \frac{D}{30} = \quad \mathbf{31.96667} \quad \textbf{ok}$$

$$e_{minx} = \frac{l}{500} + \frac{D}{30} = \quad \mathbf{28.63333} \quad \textbf{ok}$$

Since, S.R. <12, short column.

Now,

Assume p%= 1.20%

$$\frac{d'}{D} = 0.075$$

$$\frac{d'}{B} = 0.075$$

$$\frac{p}{f_{ck}} = 0.048$$

$$\frac{P_u}{f_{ck}BD} = 0.21$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = 0.12$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = 0.12$$

Thus,

$M_{ux,1} =$ 1029 KN-m

$M_{uy,1} =$ 1029 KN-m

IS 456 : 2000 cl.39.6

P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}

We get,

$P_{uz} =$ 7651.35 KN

$$\frac{P_u}{P_{uz}} = 0.335$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.335	1.225

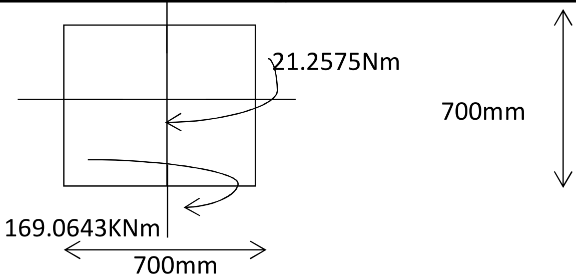
Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = 0.237 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

Case ii) 1.5(DL+LL+Eqy) (M2 max maximun case)



Unsupported Length(Lx)=	2650	m
Unsupported Length(Ly)=	2650	
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	558.835	KN

Calculated previously

M_{uy}=	169.0643	KN-m
M_{ux}=	21.2575	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) =	558.835	KN

Factored Axial Stress

$\frac{P_u}{B*D}$= 1.140 N/mm^2 > 3 N/mm^2

Hence, design as Column Member

Calculated Previously

$\frac{l_{effx}}{l}$= 0.9427

$\frac{l_{effy}}{l}$= 0.941886

So,

l_{effx}=	2498.248	mm
l_{effy}=	2495.998	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l}$ =	3.57	**<12 Short Column**
$\lambda_x = \frac{l_{effy}}{l}$ =	3.57	**<12 Short Column**

l= Least lateral dimension

IS 456 : 2000 cl.25.4

28.63333333	mm	**>20 OK**
28.63333333	mm	**>20 OK**

Also,

0.05*D =	35	mm	**>e_{minx}**
0.05*B =	35	mm	**>e_{miny}**

M_{minx}= P_u*e_{minx}

= 73.43961802 KN-m

M_{miny}= P_u*e_{miny}

= 16.00129452 KN-m

So,

M_{ux}=	169.0643	KN-m	(maximum of M_{ux} and M_{minx})
M_{uy}=	21.2575	KN-m	(maximum of M_{uy} and M_{miny})

Check for minimun eccentricity

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **31.96667** **ok**

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **28.63333** **ok**

Since, S.R. <12 short column.

Now,

Assume p%= **1.20%**

$\frac{d'}{D}$ = 0.075

$$\frac{d'}{B} = \quad 0.075$$

$$\frac{p}{f_{ck}} = \quad 0.048$$

$$\frac{P_u}{f_{ck}BD} = \quad 0.05$$

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$$\frac{M_{ux,1}}{f_{ck}BD^2} = \quad 0.095$$

$$\frac{M_{uy,1}}{f_{ck}DB^2} = \quad 0.095$$

Thus,

$M_{ux,1} =$ 814.625 KN-m

$M_{uy,1} =$ 814.625 KN-m

IS 456 : 2000 cl.39.6

$P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}$

We get,

$P_{uz} =$ 7651.35 KN

$$\frac{P_u}{P_{uz}} = \quad 0.073$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.073	1.000

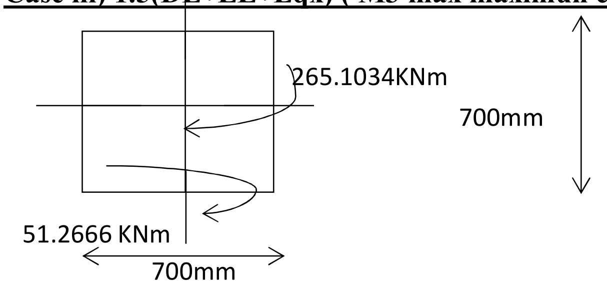
Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = \quad 0.234 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

Case iii) 1.5(DL+LL+Eqx) (M3 max maximun case)



Unsupported Length(Lx)=	2650	m
Unsupported Length(Ly)=	2650	
Depth of column: D=	700	mm
Width of column: B=	700	mm
Bar Dia(φ)=	25	mm
d'=	52.5	mm
Concrete Grade=	M25	
Steel Grade=	Fe500	
f_{ck}=	25	N/mm^2
f_y=	500	N/mm^2
P_u=	2514.420	KN

Calculated previously

M_{uy}=	51.2666	KN-m
M_{ux}=	265.1034	KN-m

Check for Axial Stress:

IS 13920:2016 cl.7.1.1

$0.08*f_{ck}$=	2.4	N/mm^2
Factored Axial Load(P_u) =	2514.420	KN

Factored Axial Stress

$\frac{P_u}{B*D}$= 5.131 N/mm^2 > 3 N/mm^2

Hence, design as Column Member

Calculated Previously

$\frac{l_{effx}}{l}$= 0.9427

$\frac{l_{effy}}{l}$= 0.941886

So,

l_{effx}=	2498.248	mm
l_{effy}=	2495.998	mm

IS 456 : 2000 cl.25.1.2

$\lambda_x = \frac{l_{effx}}{l} =$ 3.57 **<12 Short Column**

$\lambda_x = \frac{l_{effy}}{l} =$ 3.57 **<12 Short Column**

l= Least lateral dimension

Check for minimun eccentricity

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **31.96667** **ok**

$e_{minx} = \frac{l}{500} + \frac{D}{30} =$ **28.63333** **ok**

Since, S.R. <12 short column.

Now,

Assume p%= **1.20%**

$\frac{d'}{D} =$ 0.075

$\frac{d'}{B} =$ 0.075

$\frac{p}{f_{ck}} =$ 0.048

$\frac{P_u}{f_{ck}BD} =$ 0.21

Assume reinforcement is uniformly distributed on four sides,

SP16, chart 48

$\frac{M_{ux,1}}{f_{ck}BD^2} =$ 0.106

$\frac{M_{uy,1}}{f_{ck}DB^2} =$ 0.106

Thus,

$M_{ux,1} =$ 908.95 KN-m

$M_{uy,1} =$ 908.95 KN-m

IS 456 : 2000 cl.39.6

$\mathbf{P_{uz} = 0.45*fck*A_c + 0.75*fy*A_{st}}$

We get,

$P_{uz} =$ 7651.35 KN

$$\frac{P_u}{P_{uz}} = 0.329$$

IS 456 : 2000 CL 39.6

For,

P_u/P_{uz}		α_n
<=	0.2	1
>=	0.8	2
	0.329	1.214

Now,

$$\left(\frac{M_{ux}}{M_{uxl}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} = 0.254 \quad \textbf{OK}$$

Area of Steel (A_{sc})= 5880 mm^2

Area of each Bar (A_b) = 490.88 mm^2

Among three cases Pmax,M2 max & M3 max, minimun value of longitudinal reinforcement is obtained in all three cases. So we adopt the minimum longitudinal reinforcement for column.

From Etabs

Area of Steel (A_{sc})= 5880 mm^2

Provide 12-25 φmm bars equally distributed on four faces.

% A_{sc} provided = 1.2 % Range = 0.8% - 6% **OK**

A_{sc} provided = 5890.486225 mm^2

For Transverse Reinforcement,

IS 456 : 2000 CL 26.5.3.2

Diameter,

$\varphi_t \geq$ 6 mm 6 mm

$\varphi_t \geq$ 6.25 mm 1/4 of φ_l

Adopt,

$\varphi_t =$ 8 mm

Pitch should not be more than

1) 700 mm least lateral dimension

2) 400 mm 16*φ_l

3) 300 mm 300mm

Provide 8mm φ bar @ 200 mm C/C

Spacing of longitudinal bars

In X-direction,

Space between end bars =	595	mm	> 48*φ_t(384)
Space between bars =	148.75	mm	>75mm

In Y-direction,

Space between end bars =	595	mm	> 48*φ_t(384)
Space between bars =	148.75	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 close type

BEAM DETAILS

Width (b) =	400	mm
Overall Depth (D) =	550	mm
Effective Clear Cover (d') =	40	mm
Effective Depth (d) =	510	mm
Grade of Concrete (f_{ck}) =	25	N/mm^2
Yield Strength of Steel (f_y) =	500	N/mm^2
Rebar Diameter (Ø) =	20	mm
Beam Name :	B26	Story 4
Length of Beam =	4.2461	m

Minimum Area Reqd(A_{st},min)

IS 456:2000 26.5.1.1

$$A_{st}, \min = \left(\frac{0.85}{f_y}\right) * b * d$$

A_{st}, min= 346.80 mm^2

IS 13920 6.2.1

$$A_{st}, \min = 0.24 \frac{\sqrt{f_{ck}}}{f_y} * b * D$$

A_{st}, min= 528.0 mm^2

So adopt maximum of above A_{st},min

A_{st}, min= 528.0 mm^2

Maximum Area Required (A_{st},max)

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

A_{st}, max= 8800 mm^2

Maximum Area Required (A_{sc},max)

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

A_{sc}, max= 8800 mm^2

IS 456:2000 Cl 38

Formula

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

$\frac{X_{u,max}}{d}$ = 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 ($M_{u,lim}$)

$$M_{u,lim} = 0.36 * \left(\frac{X_u}{d}\right) * \left\{1 - 0.4\left(\frac{X_u}{d}\right)\right\} * b\, d^2 f_{ck}$$

$M_{u,lim}$ = 349.486 KN-m

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

From ETABS

Factored Moment (M_u) = 240.3022 KN-m (negative)

Factored Torsion (T_u) = 28.3587 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u\left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 39.619 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 279.9210 KN-m

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

$$e_{sc} = \frac{0.0035}{x_{ul}}(x_{ul} - d')$$

Where,

x_{ul}= 232.8918 mm

d'= 40 mm

We get,

e_{sc}= 0.0029

SP16 (Table A)

Interpolating,

e_{sc}	f_{sc}
0.00277	413
0.0029	?
0.00312	423.9

We,get

f_{sc} = 417.0131 N/mm^2

From IS 456:2000 Cl G-1.1 b)

$$A_{sc} = \frac{M_{e1} - M_{ul}}{f_{sc} * (d - d')}$$

A_{sc}= 0 mm^2

$$M_{ul} = 0.87 * f_y * A_{st1} * (d - \frac{f_y * A_{st1}}{f_{ck} * b})$$

Sloving,

A_{st1}= 1948.23 mm^2

$$A_{st2} = \frac{A_{sc} * f_{sc}}{0.87 * f_y}$$

= 0 mm^2

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

= 1948.23 mm^2

Area of Steel (A_{st}) = 1948.230 mm^2 Top

Percentage of Steel (%) = 0.886 %

Area of compression steel(A_{sc}) = 974.115 mm^2 Bottom

% of compression steel= 0.443 %

Position ***I (bottom)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 225.1108 KN-m (positive)

Factored Torsion (T_u) = 28.3587 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1+\frac{D}{b}}{1.7}\right)$$

$M_t =$ 39.619 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t =$ 264.7296 KN-m

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

$$Ast = 0.5 \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} bd^2}}\right) bd$$

$A_{st} =$ 1380.801 mm^2

Provide Ast min

Ast provided= 1380.8 mm^2 Bottom

Area of compression steel= 690.4006 mm^2 Top

Percentage of Steel (%) = 0.628 %

% of compression steel= 0.314 %

Position	***Middle (Top)***
Governing Combination =	Envelope

From ETABS

Factored Moment (M_u) = 0 KN-m (negative)

Factored Torsion (T_u) = 7.4168 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u\left(\frac{1+\frac{D}{b}}{1.7}\right)$$

$M_t =$ 10.362 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t =$ 10.3617 KN-m

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

$$A_{st} = 0.5 \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} bd^2}}\right) bd$$

Solving,

$A_{st} =$ 46.94533 mm^2

Area of Steel (A_{st}) provoded = 528.000 mm^2 Top

Percentage of Steel (%) = 0.240 %

Area of compression steel(A_{sc}) = 264.000 mm^2 Bottom

% of compression steel= 0.120 %

Position ***Middle (Bottom)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 41.4698 KN-m (positive)

Factored Torsion (T_u) = 7.4168 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u\left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 10.362 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 51.8315 KN-m

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

$$A_{st} = 0.5\frac{f_{ck}}{f_y}\left(1-\sqrt{1-\frac{4.6\,M_u}{f_{ck}\,bd^2}}\right)bd$$

Solving,

A_{st} = 239.3673 mm^2

Area of Steel (A_{st}) =	239.367	mm^2	Bottom
Percentage of Steel (%) =	0.109	%	
Area of compression steel =	119.684	mm^2	
Area of compression steel(A_{sc}) =	119.684	mm^2	Top
% of compression steel=	0.054	%	
%Asc=	0.054		

Position ***J (top)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 278.1513 KN-m (negative)

Factored Torsion (T_u) = 25.0372 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u\left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 34.978 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 313.1297 KN-m

Since $M>M_{ul}$, design as doubly reinforced section

$$e_{sc} = \frac{0.0035}{x_{ul}}(x_{ul} - d')$$

Where,

x_{ul}= 232.8918 mm

d'= 40 mm

We get,

e_{sc}= 0.0029

SP16 (Table A)

Interpolating,

e_{sc}	f_{sc}
0.00277	413
0.0029	?
0.00312	423.9

We,get

f_{sc} = 417.0131 N/mm^2

From IS 456:2000 Cl G-1.1 b)

$$A_{sc} = \frac{M_{e1} - M_{ul}}{f_{sc} * (d - d')}$$

A_{sc}= 0 mm^2

$$M_{ul} = 0.87 * f_y * A_{st1} * (d - \frac{f_y * A_{st1}}{f_{ck} * b})$$

Solving,

A_{st1}= 1948.23 mm^2

$$A_{st2} = \frac{A_{sc} * f_{sc}}{0.87 * f_y}$$

= 0 mm^2

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

= 1948.23 mm^2

Area of Steel (A_{st}) = 1948.230 mm^2 Top

Percentage of Steel (%) = 0.886 %

Area of compression steel(A_{sc}) = 974.115 mm^2 Bottom

% of compression steel= 0.443 %

Position ***J (Bottom)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 198.278 KN-m (positive)

Factored Torsion (T_u) = 25.0372 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 34.978 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 233.2564 KN-m

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

$$A_{st} = 0.5\frac{f_{ck}}{f_y}\left(1-\sqrt{1-\frac{4.6M_u}{f_{ck}bd^2}}\right)bd$$

Sloving,

A_{st}= 1191.01 mm^2

Area of Steel (A_{st}) = 1191.010 mm^2 Bottom

Percentage of Steel (%) = 0.541 %

Area of compression steel(A_{sc}) = 595.505 mm^2 Top

Area of Steel (A_{st}) = 1191.010 mm^2 Bottom

Percentage of Steel (%) = 0.541 %

Area of compression steel(A_{sc}) = 595.505 mm^2 Top

% of compression steel= 0.271 %

Area of compression steel = 595.505 mm^2 Top

Longitudinal Detailing

Position	Manually Calculated Area of Steel mm^2	Bars	Area Provided (mm^2)
I (top)	1948.230	7-20 Φ	2199.114858
I (bottom)	1380.801	5-20 Φ	1570.796327

Mid (top)	528.000	4 - 20 Φ	1256.637061
Mid (bot)	264.000	4-20 Φ	1256.637061
j (top)	1948.230	7-20 Φ	2199.114858
j (bottom)	1191.010	4-20 Φ	1256.637061

Check for shear and design of vertical stirrups

Width (b) =	400	mm
Overall Depth (D) =	550	mm
Effective Clear Cover (d') =	40	mm
Effective Depth (d) =	510	mm
Grade of Concrete (f_{ck}) =	25	N/mm^2
Yield Strength of Steel (f_y) =	500	N/mm^2
Stirrup Diameter (Ø) =	10	mm
longitudinal rebar Diameter ($Ø_l$) =	20	mm

At Ends

Vui=	199.34	kN
Vuj=	199.34	kN

(Clause 40.1,40.2.3,26.5.1.6, 40.3.1 and 40.4,Table-19 and 20,IS:456-2000)

Max design shear force at end i (V_{dui}) = 199.34 *kN*
Maximum designed shear at end j (Vduj)= 199.34 *kN*
Tortional moment at end i (T_{ui})= 28.461 kN
Equivalent factored shear force at end I (V_{ei})

$$V_{ei} = V_{dui} + 1.6\frac{T_u}{b}$$

V_e= ###### KN

Equivalent nominal shear stress of concrete (τ_{ue})

$$\tau_{ue} = \frac{V_e}{b*d}$$

τ_{ue}= 1.535 N/mm^2

Percentage of tensile steel of end(A_{st})= 1.078 %

Now,
Design shear strength of M25 concrete (τ_c)
Interpolating,

$\%A_{st,prov}$	τ_c
1	0.64
1.078	?
1.25	0.7

we get,

τ_c= 0.659 N/mm^2

Max shear stregth of M30 concrete ($\tau_{c,max}$)= 3.1 N/mm^2

Since,

$\tau_{ue} > \tau_c$

$\tau_{ue} < \tau_{c,max}$

Hence,shear reinforcement is to be designed

Required capacity of shear reinforcement(V_{us})=V_e-τ_c*b*d

V_{us}= 178.80 KN

Take 2-legged 10 mm Φ vertical stirrups

Area of vertical stirrups (A_{sv})= 157.08 mm^2

Spacing of Stirrups (S_v)

(Clause 40.4 (a) IS:456-2000)

$$S_v = \frac{0.87 * f_y * A_{sv} * d}{V_{us}}$$

S_v = 194.90 mm

where,

S_v over the distance of 2*d= 1020 mm should be

(Clause 6.3.5,IS:13920-1993) $S_v \le d/4$ 128 mm

$S_v \le 8*\varphi_l$ 160 mm

$S_v \ge$ 100 mm

Adopt **S_v = 120 mm**

Minimum area of vertical stirrups($A_{sv,min}$)

$$A_{sv,min} \ge \frac{0.4 * b * S_v}{0.87 * f_y}$$

$A_{sv,min}$ = 44.138 mm^2 < 157.08 sq mm

Hence, ok

At mid span

Governing Equation Envelope

Designed shear force at the mid span= 128.22 kN

Torsional moment at mid span (T_u)= 7.3935 kNm

Equivalent factored shear force at mid (Vmid)

$$\boxed{V_{mid\ equvalent} = Vmid + 1.6\frac{T_u}{b}}$$

Vue at mid= 157.8 kN

$$\tau_{ue} = \frac{V_e}{b * d}$$

τ_{ue} = 0.7735

rcentage of tensile steel of end mid (Ast mid)= 0.616 %

Design shear strength of M25 concrete (τc)

Interpolating,

$\%A_{st,prov}$	τ_c
0.25	0.36
0.616	?
0.5	0.49

τ_c= **0.550** **N/mm^2**

Max shear stregth of M30 concrete ($\tau_{c,max}$)= 3.1 N/mm^2

τue>τc

Hence, shear reinforcement is to be designed

Take 2-legged 10 mm Φ vertical stirrups

Area of vertical stirrups (A_{sv})= 157.08 mm^2

Required capacity of shear reinforcement(V_{us})=V_e-τ_c*b*d

V_{us}= 45.53 KN

Spacing of Stirrups (S_v)

(Clause 40.4 (a) IS:456-2000)

$$S_v = \frac{0.87 * f_y * A_{sv} * d}{V_{us}}$$

S_v = 765.33 mm

(Clause 6.3.5,IS:13920-1993) $S_v \leq d/2$ **255**

Adopt Sv =200mm

Hence, 10 mm Φ two legged vertical stirrups @ 250 mm C/C is to be adopted at mid span.

At j end

Maximum designed shear at end j (Vduj)= 199.34 kN

Torsional moment at mid span (T_u)= 25.0372 kNm

Equivalent factored shear force at end I

$$V_{ei} = V_{dui} + 1.6\frac{T_u}{b}$$

V_e= 299.484 KN

Equivalent nominal shear stress of concrete (τ_{ue})

$$\tau_{ue} = \frac{V_e}{b*d}$$

τ_{ue}= 1.468 N/mm^2

Percentage of tensile steel of end j (A_{stj})= 1.078 %

Now,

Design shear strength of M25 concrete (τ_c)

Interpolating,

v	τ_c
1	0.64
1.078	?
1.25	0.7

we get,

τ_c= 0.659 N/mm^2

Max shear stregth of M30 concrete ($\tau_{c,max}$)= 3.5 N/mm^2

Since,

τue>τc

τue<tcmax

Hence,shear reinforcement is to be designed

Required capacity of shear reinforcement(V_{us})=V_e-τ_c*b*d

V_{us}= 165.11 KN

Take 2-legged 10 mm Φ vertical stirrups

Area of vertical stirrups (A_{sv})= 157.08 mm^2

Spacing of Stirrups (S_v)

(Clause 40.4 (a) IS:456-2000)

$$S_v = \frac{0.87 * f_y * A_{sv} * d}{V_{us}}$$

S_v = 211.07 mm

Also

Min shear reinforcement from IS 456:2000 Clause 26.5.1.6

$$S_v = \frac{0.87 * f_y * A_{sv}}{0.4b}$$

$S_{v=}$ 427.06 mm

S_v over the distance of 2*d= 1020 mm shoul

(Clause 6.3.5,IS:13920-1993)	S_v≤d/4	128	mm
	S_v≤ 8*φ_l	144	mm
	S_v≥	100	mm
Adopt	**S_v =**	120	mm

Provision of shear reinforcement

At I end 10 mm Φ two legged vertical stirrups @ 120 mm C/C is provided up to the distance of 1020 mm from I end

At mid span	10 mm Φ two legged vertical stirrups @ 200mm C/C is provided
At j end	10 mm Φ two legged vertical stirrups @ 120 mm C/C is provided upto the distance of 1020 mm from the j end

Check for Deflection

IS 456-2000 cl.23.2.1

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

clear span=	4246.1	mm
width of support=	700	mm
1/12 of clear span=	353.84	mm

Since,

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

effective length(L_x)=	4246.1	mm
α=	26	
β=	1	span less than 10 m
δ=	1	not a flanged section

For γ

A_{sc} provided=	1256.637061	mm^2
% A_{sc} provided=	0.62	%

IS 456-2000 cl.23.2.1 fig 5

So,

γ= 1.15

For λ

$$fs = 0.58\, f_y \frac{Area\ of\ Steel\ Required}{Area\ of\ Steel\ Provided}$$

A_{st} required=	264.000	mm^2
A_{st} provided=	1256.637061	mm^2

So,

fs=	60.925	N/mm^2
%st=	0.616	%

IS 456-2000 cl.23.2.1 fig 4

λ= 1.3

So,

$\alpha\beta\gamma\delta\lambda$= 38.87

$\frac{L_x}{d}$= 8.326 $\leq \alpha\beta\gamma\delta\lambda$ **(OK)**

Check for Development Length:

IS 456-2000 cl.26.2.1

$$Ld = \frac{\Phi \sigma_s}{4 \times \tau_{bd}} = 970.98 \text{ mm} \quad \text{for tension}$$

$$Ld = \frac{\Phi \sigma_s}{4 \times \tau_{bd}} = 776.7857 \text{ mm} \quad \text{for compression}$$

φ=	20	mm	(nominal diameter of bar)
$\sigma_s=0.87*f_y$=	435	N/mm^2	(stress in bars)
τ_{bd}=	1.4*1.6	N/mm^2	(design bond stress for tension
τ_{bd}=	1.4*1.6*1.25	N/mm^2	(design bond stress for compre

Also,

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

Where,

$$M = 0.87 * f_y * A_{stprvd} * (d - \frac{f_y * A_{stprvd}}{f_{ck} * b})$$

A_{st} provided=	2199.114858	mm^2	
M_1=	382688322.2	N-mm	(MOR offered by tension steel
V=	199334.5	N	(maximum shear force at that f
L_o=(sb/2-cc-3Φ) +8Φ			(additional anchorage length)
L_o=	275	mm	

So,

$1.3 * \frac{M}{V} + L_o$ = 2770.778798 > **Ld OK**

BEAM DETAILS

Width (b) =	400	mm
Overall Depth (D) =	550	mm
Effective Clear Cover (d') =	40	mm
Effective Depth (d) =	510	mm
Grade of Concrete (f_{ck}) =	25	N/mm^2
Yield Strength of Steel (f_y) =	500	N/mm^2
Rebar Diameter (Ø) =	20	mm
Beam Name :	B32	BML
Length of Beam =	5.0765	m

Minimum Area Reqd(A_{st},min)

IS 456:2000 26.5.1.1

$$A_{st}, \min = \left(\frac{0.85}{f_y}\right) * b * d$$

A_{st}, min= 346.80 mm^2

IS 13920 6.2.1

$$A_{st}, \min = 0.24 \frac{\sqrt{f_{ck}}}{f_y} * b * D$$

A_{st}, min= 528.0 mm^2

So adopt maximum of above A_{st},min

A_{st}, min= 528.0 mm^2

Maximum Area Required (A_{st},max)

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

A_{st}, max= 8800 mm^2

Maximum Area Required (A_{sc},max)

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

A_{sc}, max= 8800 mm^2

IS 456:2000 Cl 38

Formula

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

v

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

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 ($M_{u,lim}$)

$$M_{u,lim} = 0.36 * \left(\frac{X_u}{d}\right) * \left\{1 - 0.4\left(\frac{X_u}{d}\right)\right\} * b\, d^2 f_{ck}$$

$M_{u,lim}$ = 349.486 KN-m

Position ***I (top)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 110.6287 KN-m (negative)

Factored Torsion (T_u) = 15.8009 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u\left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 22.075 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 132.7035 KN-m

Since M<Mul, design as singly reinforced section

$$e_{sc} = \frac{0.0035}{x_{ul}}\,(x_{ul} - d')$$

Where,

x_{ul}= 232.8918 mm

d'= 40 mm

We get,

e_{sc}= 0.0029

SP16 (Table A)

Interpolating,

e_{sc}	f_{sc}
0.00226	391.3

0.0029	?
0.00277	413

We,get

f_{sc} = 418.483 N/mm^2

From IS 456:2000 Cl G-1.1 b)

$$A_{sc} = \frac{M_{e1} - M_{ul}}{f_{sc} * (d - d')}$$

A_{sc}= 0 mm^2

$$M_{ul} = 0.87 * f_y * A_{st1} * \left(d - \frac{f_y * A_{st1}}{f_{ck} * b}\right)$$

Sloving,

A_{st1}= 1948.23 mm^2

$$A_{st2} = \frac{A_{sc} * f_{sc}}{0.87 * f_y}$$

= 0 mm^2

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

= 1948.23 mm^2

Area of Steel (A_{st}) = 1948.230 mm^2 Top

Percentage of Steel (%) = 0.886 %

Area of compression steel(A_{sc}) = 974.115 mm^2 Bottom

% of compression steel= 0.443 %

Position ***I (bottom)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 34.0507 KN-m (positive)

Factored Torsion (T_u) = 15.8009 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1 + \frac{D}{b}}{1.7}\right)$$

M_t = 22.075 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 56.1255 KN-m

Since M<Mul, design as singly reinforced section

$$Ast = 0.5\frac{f_{ck}}{f_y}\left(1-\sqrt{1-\frac{4.6M_u}{f_{ck}bd^2}}\right)bd$$

A_{st}= 259.7286 mm^2

Provide Ast min

Ast provided= 528.0 mm^2 Bottom

Area of compression steel= 264.0000 mm^2 Top

Percentage of Steel (%) = 0.240 %

% of compression steel= 0.120 %

Position ***Middle (Top)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 0 KN-m (negative)

Factored Torsion (T_u) = 0.5062 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u\left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 0.707 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 0.7072 KN-m

Since M<Mul, design as singly reinforced section

$$A_{st} = 0.5\frac{f_{ck}}{f_y}\left(1-\sqrt{1-\frac{4.6M_u}{f_{ck}bd^2}}\right)bd$$

Solving,

A_{st}= 3.190291 mm^2

Area of Steel (A_{st}) provoded = 528.000 mm^2 Top

Percentage of Steel (%) = 0.240 %

Area of compression steel(A_{sc}) = 264.000 mm^2 Bottom

% of compression steel= 0.120 %

Position ***Middle (Bottom)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 49.177 KN-m (positive)

Factored Torsion (T_u) = 0.5062 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 0.707 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 49.8842 KN-m

Since M<Mul, design as singly reinforced section

$$A_{st} = 0.5 \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck}\, bd^2}}\right) bd$$

Solving,

A_{st}= 230.1615 mm^2

Area of Steel (A_{st}) = 230.161 mm^2 Bottom

Percentage of Steel (%) = 0.105 %

Area of compression steel = 115.081 mm^2

Area of compression steel(A_{sc}) = 115.081 mm^2 Top

% of compression steel= 0.052 %

%Asc= 0.052

Position ***J (top)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 110.3978 KN-m (negative)

Factored Torsion (T_u) = 16.766 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 23.423 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 133.8209 KN-m

Since M<Mul, design as singly reinforced section

$$e_{sc} = \frac{0.0035}{x_{ul}}(x_{ul} - d')$$

Where,

x_{ul}= 232.8918 mm

d'= 40 mm

We get,

e_{sc}= 0.0029

SP16 (Table A)

Interpolating,

e_{sc}	f_{sc}
0.00226	391.3
0.0029	?
0.00277	413

We,get

f_{sc} = 418.483 N/mm^2

From IS 456:2000 Cl G-1.1 b)

$$A_{sc} = \frac{M_{e1} - M_{ul}}{f_{sc} * (d-d')}$$

A_{sc}= 0 mm^2

$$M_{ul} = 0.87 * f_y * A_{st1} * (d - \frac{f_y * A_{st1}}{f_{ck} * b})$$

Solving,

A_{st1}= 1948.23 mm^2

$$A_{st2} = \frac{A_{sc} * f_{sc}}{0.87 * f_y}$$

= 0 mm^2

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

= 1948.23 mm^2

Area of Steel (A_{st}) = 1948.230 mm^2 Top

Percentage of Steel (%) = 0.886 %

Area of compression steel(A_{sc}) = 974.115 mm^2 Bottom

% of compression steel= 0.443 %

Position ***J (Bottom)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 34.2161 KN-m (positive)

Factored Torsion (T_u) = 16.766 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1+\frac{D}{b}}{1.7} \right)$$

M_t = 23.423 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 57.6392 KN-m

Since M<Mul, design as singly reinforced section

$$A_{st} = 0.5 \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck}\, bd^2}}\right) bd$$

Sloving,

A_{st}= 266.9267 mm^2

Area of Steel (A_{st}) = 266.927 mm^2 Bottom

Percentage of Steel (%) = 0.121 %

Area of compression steel(A_{sc}) = 133.463 mm^2 Top

Area of Steel (A_{st}) = 266.927 mm^2 Bottom

Percentage of Steel (%) = 0.121 %

Area of compression steel(A_{sc}) = 133.463 mm^2 Top

% of compression steel= 0.061 %

Area of compression steel = 133.463 mm^2 Top

Longitudinal Detailing

Position	Manually Calculated Area of Steel mm^2	Bars	Area Provided (mm^2)	%Ast provided
I (top)	1948.230	7-20 Φ	2199.114858	1.0780
I (bottom)	974.115	5-20 Φ	1256.637061	0.6160
Mid (top)	528.000	2 - 20 Φ	942.4777961	0.4620
Mid (bot)	264.000	4-20 Φ	942.4777961	0.4620

j (top)	1948.230	22 Φ +3-20 Φ-2 layer	2199.114858	1.0780
j (bottom)	974.115	2- 22 Φ +3-20 Φ	1256.637061	0.6160

Check for shear and design of vertical stirrups

Width (b) = 400 mm
Overall Depth (D) = 550 mm
Effective Clear Cover (d') = 40 mm
Effective Depth (d) = 510 mm
Grade of Concrete (f_{ck}) = 25 N/mm^2
Yield Strength of Steel (f_y) = 500 N/mm^2
Stirrup Diameter (Ø) = 10 mm
longitudinal rebar Diameter ($Ø_l$) = 20 mm

At Ends

Vui= 42.5195 kN
Vuj= 42.5195 kN

(Clause 40.1,40.2.3,26.5.1.6, 40.3.1 and 40.4,Table-19 and 20,IS:456-2000)

Max design shear force at end i (V_{dui}) = 42.5195 *kN*

Maximum designed shear at end j (Vduj)= 42.5195 *kN*

Torsional moment at end i (T_{ui})= 2.423 kN

Equivalent factored shear force at end I (V_{ei})

$$V_{ei} = V_{dui} + 1.6\frac{T_u}{b}$$

V_e= 52.210 KN

Equivalent nominal shear stress of concrete (τ_{ue}) $\tau_{ue} = \frac{V_e}{b*d}$

τ_{ue}= 0.256 N/mm^2

Percentage of tensile steel of end(A_{st})= 1.078 %

Now,

Design shear strength of M25 concrete (τ_c)

Interpolating,

$\%A_{st,prov}$	τ_c
1	0.64
1.078	?
1.25	0.7

we get,

τ_c= 0.659 N/mm^2

Max shear stregth of M25 concrete ($\tau_{c,max}$)= 3.1 N/mm^2

Since,

$\tau_{ue} < \tau_c$

Hence,nominal shear reinforcement is to be designed

According to clause 26.5.1.6,IS456:2000

26.5.1.6 ***Minimum shear reinforcement***

Minimum shear reinforcement in the form of stirrups shall be provided such that:

$$\frac{A_{sv}}{bs_v} \geq \frac{0.4}{0.87 f_y}$$

where

A_{sv} = **total cross-sectional area of stirrup legs effective in shear,**

s_v = **stirrup spacing along the length of the member,**

b = **breadth of the beam or breadth of the web of flanged beam, and**

f_y = **characteristic strength of the stirrup reinforcement in N/mm^2 which shall not be taken greater than 415 N/mm^2.**

Take 2-legged 10 mm Φ vertical stirrups

Area of vertical stirrups (A_{sv})= 157.08 mm^2

Spacing of Stirrups (S_v)

(Clause 26.5.1.6 IS:456-2000)

$$A_{sv,min} \geq \frac{0.4 * b * S_v}{0.87 * f_y}$$

S_v = 427.06 mm

where,

S_v over the distance of 2*d= 1020 mm should be

(Clause 6.3.5,IS:13920-1993) $S_v \leq d/4$ 128 mm

$S_v \leq 8*\varphi_l$ 160 mm

$S_v \geq$ 100 mm

Adopt **S_v = 120 mm**

Minimum area of vertical stirrups($A_{sv,min}$)

$$A_{sv,min} \geq \frac{0.4 * b * S_v}{0.87 * f_y}$$

$A_{sv,min}$ = 44.138 mm^2 < 157.08 sq mm

Hence, ok

At mid span

Governing Equation Envelope

Designed shear force at the mid span= 2.443 kN

Torsional moment at mid span (T_u)= 1.5918 kNm

Equivalent factored shear force at mid (Vmid)

$$V_{mid\ equivalent} = Vmid + 1.6 \frac{T_u}{b}$$

Vue at mid= 8.8102 kN

$$\tau_{ue} = \frac{V_e}{b * d}$$

τ_{ue} = 0.043187

rcentage of tensile steel of end mid (Ast mid)= 0.462 %

Design shear strength of M25 concrete (τc)

Interpolating,

$\%A_{st,prov}$	τ_c
0.5	0.49
0.462	
0.75	0.57

τ_c= **0.478** **N/mm^2**

Max shear stregth of M25 concrete ($\tau_{c,max}$)= 3.1 N/mm^2

τue<τc

Hence, nominal shear reinforcement is to be designed

Take 2-legged 10 mm Φ vertical stirrups

Area of vertical stirrups (A_{sv})= 157.08 mm^2

<u>Spacing of Stirrups (S_v)</u>

(Clause 26.5.1.6 IS:456-2000)

$$A_{sv,min} \geq \frac{0.4 * b * S_v}{0.87 * f_y}$$

S_v = 427.06 mm

(Clause 6.3.5,IS:13920-1993) $S_v \leq d/2$ **255** **mm**

Adopt Sv =150mm

Hence, 10 mm Φ two legged vertical stirrups @ 150 mm C/C is to be adopted at mid span.

At j end

Maximum designed shear at end j (Vduj)= 42.52 kN

Torsional moment at mid span (T_u)= 2.423 kNm

Equivalent factored shear force at end I (V_{ei})

$$V_{ei} = V_{dui} + 1.6 \frac{T_u}{b}$$

V_e= 52.212 KN

Equivalent nominal shear stress of concrete (τ_{ue})

$$\tau_{ue} = \frac{V_e}{b*d}$$

τ_{ue}= 0.256 N/mm^2

Percentage of tensile steel of end j (A_{stj})= 1.078 %

Now,

Design shear strength of M25 concrete (τ_c)

Interpolating,

$\%A_{st,prov}$	τ_c
1	0.64
1.078	?
1.25	0.7

we get,

τ_c= 0.659 N/mm^2

Max shear stregth of M25 concrete ($\tau_{c,max}$)= 3.1 N/mm^2

Since,

τue<τc

τue<tcmax

Hence,nominal shear reinforcement is to be designed

Take 2-legged 10 mm Φ vertical stirrups

Area of vertical stirrups (A_{sv})= 157.08 mm^2

<u>Spacing of Stirrups (S_v)</u>

(Clause 26.5.6.1 (a) IS:456-2000)

$$S_v = \frac{0.87* f_y * A_{sv} *d}{V_{us}}$$

S_v = 427.06 mm

Also

S_v over the distance of 2*d= 1020 mm should be

(Clause 6.3.5,IS:13920-1993) $S_v \le d/4$ 128 mm

$S_v \le 8*\varphi_l$ 144 mm

	$S_v \geq$	100	mm
Adopt	S_v =	120	mm

Provision of shear reinforcement

At I end	10 mm Φ two legged vertical stirrups @ 120 mm C/C is provided up to the distance of 720 mm from I end
At mid span	10 mm Φ two legged vertical stirrups @ 150mm C/C is provided
At j end	10 mm Φ two legged vertical stirrups @ 120 mm C/C is provided upto the distance of 720 mm from the j end

Check for Deflection

IS 456-2000 cl.23.2.1

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

clear span=	5076.5	mm
width of support=	700	mm
1/12 of clear span=	423.04	mm

Since,

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

effective length(L_x)=	5076.5	mm
α=	26	
β=	1	span less than 10 m
δ=	1	not a flanged section

For γ

A_{sc} provided=	942.4777961	mm^2
% A_{sc} provided=	0.46	%

IS 456-2000 cl.23.2.1 fig 5

So,

γ= 1.15

For λ

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

A_{st} required= 264.000 mm^2

A_{st} provided= 942.4777961 mm^2

So,

fs= 81.233 N/mm^2

%st= 0.462 %

IS 456-2000 cl.23.2.1 fig 4

λ= 1.4

So,

$\alpha\beta\gamma\delta\lambda$= 41.86

$\frac{L_x}{d}$= 9.954 $\leq \alpha\beta\gamma\delta\lambda$ **(OK)**

<u>Check for Development Length:</u>

IS 456-2000 cl.26.2.1

$$Ld = \frac{\Phi \sigma_s}{4 \ x \ \tau_{bd}} = 970.98 \text{ mm} \quad \text{for tension}$$

$$Ld = \frac{\Phi \sigma_s}{4 \ x \ \tau_{bd}} = 776.7857 \text{ mm} \quad \text{for compression}$$

φ=	20	mm	(nominal diameter of bar)
$\sigma_s=0.87*f_y$=	435	N/mm^2	(stress in bars)
τ_{bd}=	1.4*1.6	N/mm^2	(design bond stress for tension)
τ_{bd}=	1.4*1.6*1.25	N/mm^2	(design bond stress for compression)

Also,

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

Where,

$$M = 0.87 * f_y * A_{stprvd} * (d - \frac{f_y * A_{stprvd}}{f_{ck} * b})$$

A_{st} provided=	2199.114858	mm^2	
M_1=	382688322.2	N-mm	(MOR offered by tension steel provided)
V=	199334.5	N	(maximum shear force at that face)
L_o=(sb/2-cc-3Φ) +8Φ			(additional anchorage length)
L_o=	275	mm	

So,

$$1.3 * \frac{M}{V} + L_o = 2770.778798 > \textbf{Ld} \quad \textbf{OK}$$

BEAM DETAILS

Width (b) =	400	mm
Overall Depth (D) =	550	mm
Effective Clear Cover (d') =	40	mm
Effective Depth (d) =	510	mm
Grade of Concrete (f_{ck}) =	25	N/mm^2
Yield Strength of Steel (f_y) =	500	N/mm^2
Rebar Diameter (Ø) =	20	mm
Beam Name :	B48	Story 2
Length of Beam =	4.912	m

Minimum Area Reqd(A_{st},min)

IS 456:2000 26.5.1.1

$$A_{st}, \min = \left(\frac{0.85}{f_y}\right) * b * d$$

A_{st}, min= 346.80 mm^2

IS 13920 6.2.1

$$A_{st}, \min = 0.24 \frac{\sqrt{f_{ck}}}{f_y} * b * D$$

A_{st}, min= 528.0 mm^2

So adopt maximum of above A_{st},min

A_{st}, min= 528.0 mm^2

Maximum Area Required (A_{st},max)

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

A_{st}, max= 8800 mm^2

Maximum Area Required (A_{sc},max)

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

A_{sc}, max= 8800 mm^2

IS 456:2000 Cl 38

Formula

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

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

a

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 ($M_{u,lim}$)

$$M_{u,lim} = 0.36 * \left(\frac{X_u}{d}\right) * \left\{1 - 0.4\left(\frac{X_u}{d}\right)\right\} * b\, d^2 f_{ck}$$

$M_{u,lim}$ = 349.486 KN-m

Position ***I (top)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 285.453 KN-m (negative)

Factored Torsion (T_u) = 40.153 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 56.096 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 341.5491 KN-m

Since M<Mul, design as singly reinforced section

$$e_{sc} = \frac{0.0035}{x_{ul}} (x_{ul} - d')$$

Where,

x_{ul}= 232.8918 mm

d'= 40 mm

We get,

e_{sc}= 0.0029

SP16 (Table A)

Interpolating,

e_{sc}	f_{sc}
0.00277	413
0.0029	?
0.00312	423.9

We,get

$f_{sc} =$ 417.0131 N/mm^2

From IS 456:2000 Cl G-1.1 b)

$$A_{sc} = \frac{M_{e1} - M_{ul}}{f_{sc} * (d - d')}$$

$A_{sc} =$ 0 mm^2

$$M_{ul} = 0.87 * f_y * A_{st1} * (d - \frac{f_y * A_{st1}}{f_{ck} * b})$$

Sloving,

$A_{st1} =$ 1948.23 mm^2

$$A_{st2} = \frac{A_{sc} * f_{sc}}{0.87 * f_y}$$

= 0 mm^2

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

= 1948.23 mm^2

Area of Steel (A_{st}) =	1948.230	mm^2	Top
Percentage of Steel (%) =	0.886	%	
Area of compression steel(A_{sc}) =	974.115	mm^2	Bottom
% of compression steel=	0.443	%	

Position ***I (bottom)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 187.1363 KN-m (positive)

Factored Torsion (T_u) = 40.1891 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1 + \frac{D}{b}}{1.7} \right)$$

$M_t =$ 56.147 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t =$ 243.2828 KN-m

Since M<Mul, design as singly reinforced section

$$Ast= 0.5\frac{f_{ck}}{f_y}(1-\sqrt{1-\frac{4.6\,M}{f_{ck}\,bc}}$$

$A_{st}=$ 1250.456 mm^2

Provide Ast min

Ast provided= 1250.5 mm^2 Bottom

Area of compression steel= 625.2280 mm^2 Top

Percentage of Steel (%) = 0.568 %

% of compression steel= 0.284 %

<u>Position</u> ***<u>Middle (Top)</u>***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 0 KN-m (negative)

Factored Torsion (T_u) = 28.101 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u\left(\frac{1+\frac{D}{b}}{1.7}\right)$$

$M_t =$ 39.259 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t =$ 39.2588 KN-m

Since M<Mul, design as singly reinforced section

$$A_{st}= 0.5\frac{f_{ck}}{f_y}(1-\sqrt{1-\frac{4.6\,M_u}{f_{ck}\,bd^2}})bd$$

Solving,

$A_{st}=$ 180.234 mm^2

Area of Steel (A_{st}) provoded = 528.000 mm^2 Top

Percentage of Steel (%) = 0.240 %

Area of compression steel(A_{sc}) = 264.000 mm^2 Bottom

% of compression steel= 0.120 %

<u>Position</u> ***<u>Middle (Bottom)</u>***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 79.2564 KN-m (positive)

Factored Torsion (T_u) = 28.101 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 39.259 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 118.5152 KN-m

Since M<Mul, design as singly reinforced section

$$A_{st} = 0.5\frac{f_{ck}}{f_y}\left(1-\sqrt{1-\frac{4.6M_u}{f_{ck}\,bd^2}}\right)bd$$

Solving,

A_{st}= 565.8735 mm^2

Area of Steel (A_{st}) =	565.874	mm^2	Bottom
Percentage of Steel (%) =	0.257	%	
Area of compression steel =	282.937	mm^2	
Area of compression steel(A_{sc}) =	282.937	mm^2	Top
% of compression steel=	0.129	%	
%Asc=	0.129		

<u>Position</u> ***<u>J (top)</u>***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 305.5504 KN-m (negative)

Factored Torsion (T_u) = 40.7458 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 56.924 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 362.4747 KN-m

Since M>Mul design as doubly reinforced section

$$e_{sc} = \frac{0.0035}{x_{ul}}(x_{ul} - d')$$

Where,

x_{ul}= 232.8918 mm

$d' = 40$ mm

We get,

$e_{sc} = 0.0029$

SP16 (Table A)

Interpolating,

e_{sc}	f_{sc}
0.00277	413
0.0029	?
0.00312	423.9

We,get

$f_{sc} = 417.0131$ N/mm^2

From IS 456:2000 Cl G-1.1 b)

$$A_{sc} = \frac{M_{e1} - M_{ul}}{f_{sc} * (d - d')}$$

$A_{sc} = 66.27099$ mm^2

$$M_{ul} = 0.87 * f_y * A_{st1} * (d - \frac{f_y * A_{st1}}{f_{ck} * b})$$

Solving,

$A_{st1} = 1948.23$ mm^2

$$A_{st2} = \frac{A_{sc} * f_{sc}}{0.87 * f_y}$$

$= 63.53074$ mm^2

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

$= 2011.761$ mm^2

Area of Steel (A_{st}) = 2011.761 mm^2 Top

Percentage of Steel (%) = 0.914 %

Area of compression steel(A_{sc}) = 1005.881 mm^2 Bottom

% of compression steel= 0.457 %

Position ***J (Bottom)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 190.7392 KN-m (positive)

Factored Torsion (T_u) = 40.7458 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u\left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 56.924 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 247.6635 KN-m

Since M<Mul, design as singly reinforced section

$$A_{st} = 0.5\frac{f_{ck}}{f_y}\left(1-\sqrt{1-\frac{4.6M_u}{f_{ck}\,bd^2}}\right)bd$$

Sloving,

A_{st}= 1276.719 mm^2

Area of Steel (A_{st}) = 1276.719 mm^2 Bottom

Percentage of Steel (%) = 0.580 %

Area of compression steel(A_{sc}) = 638.359 mm^2 Top

Area of Steel (A_{st}) = 1276.719 mm^2 Bottom

Percentage of Steel (%) = 0.580 %

Area of compression steel(A_{sc}) = 638.359 mm^2 Top

% of compression steel= 0.290 %

Area of compression steel = 638.359 mm^2 Top

Longitudinal Detailing

Position	Manually Calculated Area of Steel mm^2	Bars	Area Provided (mm^2)
I (top)	1948.230	7-20 Φ	2199.114858
I (bottom)	1250.456	5-20 Φ	1570.796327
Mid (top)	528.000	4-20 Φ	1256.637061
Mid (bot)	565.874	4-20 Φ	1256.637061
j (top)	2011.761	7-20 Φ	2199.114858
j (bottom)	1276.719	5-20 Φ	1570.796327

Check for shear and design of vertical stirrups

Width (b) = 400 mm

Overall Depth (D) = 550 mm

Effective Clear Cover (d') =	40	mm
Effective Depth (d) =	510	mm
Grade of Concrete (f_{ck}) =	25	N/mm^2
Yield Strength of Steel (f_y) =	500	N/mm^2
Stirrup Diameter (Ø) =	10	mm
longitudinal rebar Diameter ($Ø_l$) =	20	mm

<u>At Ends</u>

Vui=	224.9437	kN
Vuj=	224.9437	kN

(Clause 40.1,40.2.3,26.5.1.6, 40.3.1 and 40.4,Table-19 and 20,IS:456-2000)

Max design shear force at end i (V_{dui}) =	224.9437	*kN*
Maximum designed shear at end j (Vduj)=	224.9437	*kN*

Torsional moment at end i (T_{ui})= 40.746 kN

Equivalent factored shear force at end I (V_{ei})

$$V_{ei} = V_{dui} + 1.6\frac{T_u}{b}$$

V_e= 387.927 KN

Equivalent nominal shear stress of concrete (τ_{ue})

$$\tau_{ue} = \frac{V_e}{b*d}$$

τ_{ue}= 1.902 N/mm^2

Percentage of tensile steel of end(A_{st})= 1.078 %

Now,

Design shear strength of M25 concrete (τ_c)

Interpolating,

$\%A_{st,prov}$	τ_c
1	0.64
1.078	?
1.25	0.7

we get,

τ_c= 0.659 N/mm^2

Max shear stregth of M30 concrete ($\tau_{c,max}$)= 3.1 N/mm^2

Since,

$\tau_{ue} > \tau_c$

$\tau_{ue} < \tau_{c,max}$

Hence,shear reinforcement is to be designed

Required capacity of shear reinforcement(V_{us})=V_e-τ_c*b*d

V_{us}= 253.55 KN

Take 2-legged 10 mm Φ vertical stirrups

Area of vertical stirrups (A_{sv})= 157.08 mm^2

Spacing of Stirrups (S_v)

(Clause 40.4 (a) IS:456-2000)

$$S_v = \frac{0.87* f_y * A_{sv} *d}{V_{us}}$$

$S_v =$ 137.44 mm

where,

S_v over the distance of 2*d= 1020 mm should be

(Clause 6.3.5,IS:13920-1993) $S_v \le d/4$ 128 mm

$S_v \le 8*\varphi_l$ 160 mm

$S_v \ge$ 100 mm

Adopt **$S_v =$ 120 mm**

Minimum area of vertical stirrups($A_{sv,min}$)

$$A_{sv,min} \ge \frac{0.4*b*S_v}{0.87*f_y}$$

$A_{sv,min} =$ 44.138 mm^2 < 157.08 sq mm

Hence, ok

At mid span

Governing Equation Envelope

Designed shear force at the mid span= 8.4803 kN

Torsional moment at mid span (T_u)= 28.101 kNm

Equivalent factored shear force at mid (Vmid)

$$V_{mid\ equvalent} = Vmid + 1.6\frac{T_u}{b}$$

Vue at mid= 120.8843 kN

$$\tau_{ue} = \frac{V_e}{b*d}$$

$\tau_{ue} =$ 0.59257

rcentage of tensile steel of end mid (Ast mid)= 0.616 %

Design shear strength of M25 concrete (τc)

Interpolating,

$\%A_{st,prov}$	τ_c
0.5	0.49
0.616	
0.75	0.57

$\tau_c =$ **0.527 N/mm^2**

Max shear stregth of M25 concrete ($\tau_{c,max}$)= 3.1 N/mm^2

$\tau ue > \tau c$

Hence, shear reinforcement is to be designed

Take 2-legged 10 mm Φ vertical stirrups

Area of vertical stirrups (A_{sv})= 157.08 mm^2

Required capacity of shear reinforcement(V_{us})=V_e-τ_c*b*d

V_{us}= 13.35 KN

Spacing of Stirrups (S_v)

(Clause 40.4 (a) IS:456-2000)

$$S_v = \frac{0.87 * f_y * A_{sv} * d}{V_{us}}$$

S_v = 2609.98 mm

(Clause 6.3.5,IS:13920-1993) $S_v \leq d/2$ **255**

Adopt Sv =200mm

Hence, 10 mm Φ two legged vertical stirrups @ 200 mm C

At j end

Maximum designed shear at end j (Vduj)= 224.94 kN

Torsional moment at mid span (T_u)= 40.746 kNm

Equivalent factored shear force at end I (V_{ei})

$$V_{ei} = V_{dui} + 1.6\frac{T_u}{b}$$

V_e= 387.928 KN

Equivalent nominal shear stress of concrete (τ_{ue})

$$\tau_{ue} = \frac{V_e}{b*d}$$

τ_{ue}= 1.902 N/mm^2

Percentage of tensile steel of end j (A_{stj})= 1.078 %

Now,

Design shear strength of M25 concrete (τ_c)

Interpolating,

$\%A_{st,prov}$	τ_c
1	0.64
1.078	?
1.25	0.7

we get,

τ_c= 0.659 N/mm^2

Max shear stregth of M25 concrete ($\tau_{c,max}$)= 3.1 N/mm^2

Since,

τue>τc

τue<tcmax

Hence,shear reinforcement is to be designed

Required capacity of shear reinforcement(V_{us})=V_e-τ_c*b*d

V_{us}= 253.55 KN

Take 2-legged 10 mm Φ vertical stirrups

Area of vertical stirrups (A_{sv})= 157.08 mm^2

Spacing of Stirrups (S_v)

(Clause 40.4 (a) IS:456-2000)

$$S_v = \frac{0.87 * f_y * A_{sv} * d}{V_{us}}$$

S_v = 137.44 mm

Also

Min shear reinforcement from IS 456:2000

$$S_v = \frac{0.87 * f_y * A_{sv}}{0.4b}$$

$S_{v=}$ 427.06 mm

S_v over the distance of 2*d= 1020 mm shoul

(Clause 6.3.5,IS:13920-1993) S_v≤d/4 128 mm

S_v≤ 8*φ_l 144 mm

S_v≥ 100 mm

Adopt **S_v =** 120 mm

Provision of shear reinforcement

At I end	10 mm Φ two legged vertical stirrups @ 120 mm C/C is provi distance of 1020 mm from I end
At mid span	10 mm Φ two legged vertical stirrups @ 200mm C/C is provided
At j end	10 mm Φ two legged vertical stirrups @ 120 mm C/C is prov distance of 1020 mm from the j end

Check for Deflection

IS 456-2000 cl.23.2.1

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

clear span=	4912	mm
width of support=	700	mm
1/12 of clear span=	409.33	mm

Since,

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

effective length($L_{x)}$=	4912	mm
α=	26	
β=	1	span less than 10 m
δ=	1	not a flanged section

For γ

A_{sc} provided=	1256.637061	mm^2
% A_{sc} provided=	0.62	%

IS 456-2000 cl.23.2.1 fig 5

So,

γ= 1.15

For λ

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

A_{st} required=	565.874	mm^2
A_{st} provided=	1256.637061	mm^2

So,

fs=	130.589	N/mm^2
%st=	0.616	%

IS 456-2000 cl.23.2.1 fig 4

λ= 1.3

So,

$\alpha\beta\gamma\delta\lambda$= 38.87

$\frac{L_x}{d}$= 9.631 $\leq \alpha\beta\gamma\delta\lambda$ **(OK)**

Check for Development Length:

IS 456-2000 cl.26.2.1

$$Ld = \frac{\Phi \sigma_s}{4 \ x \ \tau_{bd}} = 970.98 \text{ mm} \quad \text{for tension}$$

$$Ld = \frac{\Phi \sigma_s}{4 \ x \ \tau_{bd}} = 776.7857 \text{ mm} \quad \text{for compression}$$

φ=	20	mm	(nominal diameter of bar)
$\sigma_s=0.87*f_y$=	435	N/mm^2	(stress in bars)
τ_{bd}=	1.4*1.6	N/mm^2	(design bond stress for tension)
τ_{bd}=	1.4*1.6*1.25	N/mm^2	(design bond stress for compress

Also,

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

Where,

$$M = 0.87 * f_y * A_{stprvd} *(d- \frac{f_y * A_{stprvd}}{f_{ck} * b})$$

A_{st} provided=	2199.114858	mm^2	
M_1=	382688322.2	N-mm	(MOR offered by tension steel pr
V=	199334.5	N	(maximum shear force at that fac
L_o=(sb/2-cc-3Φ) +8Φ			(additional anchorage length)
L_o=	275	mm	

So,

$$1.3 * \frac{M}{V} + L_o = 2770.778798 > \textbf{Ld} \quad \textbf{OK}$$

BEAM DETAILS

Width (b) =	300	mm
Overall Depth (D) =	400	mm
Effective Clear Cover (d') =	40	mm
Effective Depth (d) =	360	mm
Grade of Concrete (f_{ck}) =	25	N/mm^2
Yield Strength of Steel (f_y) =	500	N/mm^2
Rebar Diameter (Ø) =	20	mm

Beam Name : B12 BML

Length of Beam = 4.9935 m

Secondary beam

Minimum Area Reqd(A_{st},min)

IS 456:2000 26.5.1.1

$$A_{st}, min = \left(\frac{0.85}{f_y}\right) * b * d$$

A_{st}, min= 183.60 mm^2

IS 13920 6.2.1

$$A_{st}, min = 0.24 \frac{\sqrt{f_{ck}}}{f_y} * b * D$$

A_{st}, min= 288.0 mm^2

So adopt maximum of above A_{st},min

A_{st}, min= 288.0 mm^2

Maximum Area Required (A_{st},max)

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

A_{st}, max= 4800 mm^2

Maximum Area Required (A_{sc},max)

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

A_{sc}, max= 4800 mm^2

IS 456:2000 Cl 38

Formula

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

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

a

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 ($M_{u,lim}$)

$$M_{u,lim} = 0.36 * \left(\frac{X_u}{d}\right) * \left\{1 - 0.4\left(\frac{X_u}{d}\right)\right\} * b\, d^2 f_{ck}$$

$M_{u,lim}$ = 130.604 KN-m

Position ***I (top)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 51.6745 KN-m (negative)

Factored Torsion (T_u) = 2.4227 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 3.325 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 54.9998 KN-m

Since M<Mul, design as singly reinforced section

$$e_{sc} = \frac{0.0035}{x_{ul}} (x_{ul} - d')$$

Where,

x_{ul}= 164.3942 mm

d'= 40 mm

We get,

e_{sc}= 0.0026

SP16 (Table A)

Interpolating,

e_{sc}	f_{sc}
0.00226	391.3
0.0026	?
0.00277	413

We,get

f_{sc} = 407.8256 N/mm^2

From IS 456:2000 Cl G-1.1 b)

$$A_{sc} = \frac{M_{e1} - M_{ul}}{f_{sc} * (d - d')}$$

A_{sc}= 0 mm^2

$$M_{ul} = 0.87 * f_y * A_{st1} * (d - \frac{f_y * A_{st1}}{f_{ck} * b})$$

Sloving,

A_{st1}= 1031.416 mm^2

$$A_{st2} = \frac{A_{sc} * f_{sc}}{0.87 * f_y}$$

= 0 mm^2

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

= 1031.416 mm^2

Area of Steel (A_{st}) =	1031.416	mm^2	Top
Percentage of Steel (%) =	0.860	%	
Area of compression steel(A_{sc}) =	515.708	mm^2	Bottom
% of compression steel=	0.430	%	

Position ***I (bottom)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) =	0	KN-m	(positive)
Factored Torsion (T_u) =	2.4227	KN-m	

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1 + \frac{D}{b}}{1.7} \right)$$

M_t = 3.325 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t =$ 3.3253 KN-m

Since M<Mul, design as singly reinforced section

$$Ast= 0.5\frac{f_{ck}}{f_y}(1-\sqrt{1-\frac{4.6M}{f_{ck}bd}}$$

$A_{st}=$ 21.32906 mm^2

Provide Ast min

Ast provided= 288.0 mm^2 Bottom

Area of compression steel= 144.0000 mm^2 Top

Percentage of Steel (%) = 0.240 %

% of compression steel= 0.120 %

Position ***Middle (Top)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 0 KN-m (negative)

Factored Torsion (T_u) = 2.4227 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u\left(\frac{1+\frac{D}{b}}{1.7}\right)$$

$M_t =$ 3.325 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t =$ 3.3253 KN-m

Since M<Mul, design as singly reinforced section

$$A_{st}= 0.5\frac{f_{ck}}{f_y}(1-\sqrt{1-\frac{4.6M_u}{f_{ck}bd^2}})bd$$

Solving,

$A_{st}=$ 21.32906 mm^2

Area of Steel (A_{st}) provoded = 288.000 mm^2 Top

Percentage of Steel (%) = 0.240 %

Area of compression steel(A_{sc}) = 144.000 mm^2 Bottom

% of compression steel= 0.120 %

Position ***Middle (Bottom)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 16.4186 KN-m (positive)

Factored Torsion (T_u) = 1.59 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 2.182 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 18.6010 KN-m

Since M<Mul, design as singly reinforced section

$$A_{st} = 0.5\frac{f_{ck}}{f_y}\left(1-\sqrt{1-\frac{4.6M_u}{f_{ck}\,bd^2}}\right)bd$$

Solving,

A_{st} = 121.5766 mm^2

Area of Steel (A_{st}) =	121.577	mm^2	Bottom
Percentage of Steel (%) =	0.101	%	
Area of compression steel =	60.788	mm^2	
Area of compression steel(A_{sc}) =	60.788	mm^2	Top
% of compression steel=	0.051	%	
%Asc=	0.051		
Position	***J (top)***		
Governing Combination =	Envelope		

From ETABS

Factored Moment (M_u) = 23.3523 KN-m (negative)

Factored Torsion (T_u) = 1.6366 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1+\frac{D}{b}}{1.7}\right)$$

M_t = 2.246 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 25.5986 KN-m

Since M<Mul, design as singly reinforced section

$$e_{sc} = \frac{0.0035}{x_{ul}}(x_{ul} - d')$$

Where,

x_{ul} = 164.3942 mm

d'= 40 mm

We get,

e_{sc}= 0.0026

SP16 (Table A)

Interpolating,

e_{sc}	f_{sc}
0.00226	391.3
0.0026	?
0.00277	413

We,get

f_{sc} = 407.8256 N/mm^2

From IS 456:2000 Cl G-1.1 b)

$$A_{sc} = \frac{M_{e1} - M_{ul}}{f_{sc} * (d - d')}$$

A_{sc}= 0 mm^2

$$M_{ul} = 0.87 * f_y * A_{st1} * (d - \frac{f_y * A_{st1}}{f_{ck} * b})$$

Solving,

A_{st1}= 1031.416 mm^2

$$A_{st2} = \frac{A_{sc} * f_{sc}}{0.87 * f_y}$$

= 0 mm^2

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

= 1031.416 mm^2

Area of Steel (A_{st}) = 1031.416 mm^2 Top

Percentage of Steel (%) = 0.860 %

Area of compression steel(A_{sc}) = 515.708 mm^2 Bottom

% of compression steel= 0.430 %

Position ***J (Bottom)***

Governing Combination = Envelope

From ETABS

Factored Moment (M_u) = 0 KN-m (positive)

Factored Torsion (T_u) = 1.6366 KN-m

IS 456:2000, Cl 41.4.2

Moment due to Torsion (M_t)

$$M_t = T_u \left(\frac{1+\frac{D}{b}}{1.7} \right)$$

M_t = 2.246 KN-m

Therefore, The required Design Moment (M) is

$M = M_u + M_t$ = 2.2463 KN-m

Since M<Mul, design as singly reinforced section

$$A_{st} = 0.5 \frac{f_{ck}}{f_y} \left(1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck}\, bd^2}}\right) bd$$

Sloving,

A_{st} = 14.38979 mm^2

Area of Steel (A_{st}) = 14.390 mm^2 Bottom

Percentage of Steel (%) = 0.012 %

Area of compression steel(A_{sc}) = 7.195 mm^2 Top

Area of Steel (A_{st}) = 14.390 mm^2 Bottom

Percentage of Steel (%) = 0.012 %

Area of compression steel(A_{sc}) = 7.195 mm^2 Top

% of compression steel= 0.006 %

Area of compression steel = 7.195 mm^2 Top

Longitudinal Detailing

Position	Manually Calculated Area of Steel mm^2	Bars	Area Provided (mm^2)
I (top)	1031.416	4-20 Φ	1256.637061
I (bottom)	515.708	2-20 Φ	628.3185307
Mid (top)	288.000	2 - 20 Φ	628.3185307
Mid (bot)	144.000	2-20 Φ	628.3185307
j (top)	1031.416	4-20 Φ	1256.637061
j (bottom)	515.708	2-20 Φ	628.3185307

Check for shear and design of vertical stirrups

Width (b) = 300 mm

Overall Depth (D) = 400 mm

Effective Clear Cover (d') =	40	mm
Effective Depth (d) =	360	mm
Grade of Concrete (f_{ck}) =	25	N/mm^2
Yield Strength of Steel (f_y) =	500	N/mm^2
Stirrup Diameter (Ø) =	10	mm
longitudinal rebar Diameter ($Ø_l$) =	20	mm
At Ends		
Vui=	42.5195	kN
Vuj=	42.5195	kN

(Clause 40.1,40.2.3,26.5.1.6, 40.3.1 and 40.4,Table-19 and 20,IS:456-2000)

Max design shear force at end i (V_{dui}) =	42.5195	*kN*
Maximum designed shear at end j (Vduj)=	42.5195	*kN*

Torsional moment at end i (T_{ui})= 2.423 kN

Equivalent factored shear force at end I (V_{ei})

$$V_{ei} = V_{dui} + 1.6\frac{T_u}{b}$$

V_e= 55.441 KN

Equivalent nominal shear stress of concrete (τ_{ue})

$$\tau_{ue} = \frac{V_e}{b*d}$$

τ_{ue}= 0.513 N/mm^2

Percentage of tensile steel of end(A_{st})= 1.164 %

Now,

Design shear strength of M25 concrete (τ_c)

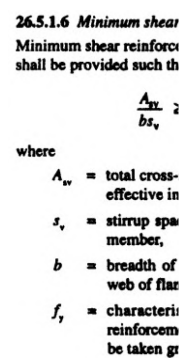
Interpolating,

$\%A_{st,prov}$	τ_c
1	0.64
1.164	?
1.25	0.7

we get,

τ_c= 0.679 N/mm^2

Max shear stregth of M25 concrete ($\tau_{c,max}$)= 3.1 N/mm^2


26.5.1.6 ***Minimum shear***

Minimum shear reinforc
shall be provided such th

$\frac{A_{sv}}{bs_v}$ ≥

where

A_{sv} = total cross-
effective in

s_v = stirrup spa
member,

b = breadth of
web of flan

f_y = characteri
reinforcem
be taken gr

Since,

$\tau_{ue} < \tau_c$

Hence,nominal shear reinforcement is to be designed

According to clause 26.5.1.6,IS456:2000

Take 2-legged 10 mm Φ vertical stirrups

Area of vertical stirrups (A_{sv})= 157.08 mm^2

Spacing of Stirrups (S_v)

(Clause 26.5.1.6 IS:456-2000)

$$A_{sv,min} \geq \frac{0.4*b*S_v}{0.87*f_y}$$

S_v = 569.42 mm

where,

S_v over the distance of 2*d= 720 mm should be

(Clause 6.3.5,IS:13920-1993) $S_v \leq d/4$ 90 mm

$S_v \leq 8*\varphi_l$ 160 mm

$S_v \geq$ 100 mm

Adopt **S_v = 120 mm**

Minimum area of vertical stirrups($A_{sv,min}$)

$$A_{sv,min} \geq \frac{0.4*b*S_v}{0.87*f_y}$$

$A_{sv,min}$ = 33.103 mm^2 < 157.08 sq mm

Hence, ok

At mid span

Governing Equation Envelope

Designed shear force at the mid span= 2.443 kN

Torsional moment at mid span (T_u)= 1.5918 kNm

Equivalent factored shear force at mid (Vmid)

$$V_{\text{mid equvalent}} = \text{Vmid} + 1.6\frac{T_u}{b}$$

Vue at mid= 10.9326 kN

$$\tau_{ue} = \frac{V_e}{b*d}$$

τ_{ue} = 0.101228

rcentage of tensile steel of end mid (Ast mid)= 0.582 %

Design shear strength of M25 concrete (τc)

Interpolating,

$\%A_{st,prov}$	τ_c

0.5	0.49
0.582	
0.75	0.57

τ_c= **0.516** **N/mm^2**

Max shear stregth of M25 concrete ($\tau_{c,max}$)= 3.1 N/mm^2

τue<τc

Hence, nominal shear reinforcement is to be designed

Take 2-legged 10 mm Φ vertical stirrups

Area of vertical stirrups (A_{sv})= 157.08 mm^2

Spacing of Stirrups (S_v)

(Clause 26.5.1.6 IS:456-2000)

$$A_{sv,min} \geq \frac{0.4*b*S_v}{0.87*f_y}$$

S_v = 569.42 mm

(Clause 6.3.5,IS:13920-1993) $S_v \leq d/2$ **180**

Adopt Sv =150mm

Hence, 10 mm Φ two legged vertical stirrups @ 150 mr adopted at mid span.

At j end

Maximum designed shear at end j (Vduj)= 42.52 kN

Torsional moment at mid span ($T_{u)}$= 2.423 kNm

Equivalent factored shear force at end I (V_{ei})

$$V_{ei} = V_{dui} + 1.6\frac{T_u}{b}$$

V_e= 55.442 KN

Equivalent nominal shear stress of concrete (τ_{ue})

$$\tau_{ue} = \frac{V_e}{b*d}$$

τ_{ue}= 0.513 N/mm^2

Percentage of tensile steel of end j (A_{stj})= 1.164 %

Now,

Design shear strength of M25 concrete (τ_c)

Interpolating,

$\%A_{st,prov}$	τ_c
1	0.64
1.164	?
1.25	0.7

we get,

τ_c= 0.679 N/mm^2

Max shear stregth of M25 concrete ($\tau_{c,max}$)= 3.1 N/mm^2

Since,

τue<τc

τue<tcmax

Hence,nominal shear reinforcement is to be designed

Take 2-legged 10 mm Φ vertical stirrups

Area of vertical stirrups (A_{sv})= 157.08 mm^2

Spacing of Stirrups (S_v)

(Clause 26.5.6.1 (a) IS:456-2000)

$$S_v = \frac{0.87 * f_y * A_{sv} * d}{V_{us}}$$

S_v = 569.42 mm

Also

S_v over the distance of 2*d= 720 mm shoul

(Clause 6.3.5,IS:13920-1993)	$S_v \le d/4$	128	mm
	$S_v \le 8*\varphi_l$	144	mm
	$S_v \ge$	100	mm
Adopt	**S_v =**	120	mm

Provision of shear reinforcement

At I end 10 mm Φ two legged vertical stirrups @ 120 mm C/C is provic distance of 720 mm from I end

At mid span 10 mm Φ two legged vertical stirrups @ 150mm C/C is provided

At j end 10 mm Φ two legged vertical stirrups @ 120 mm C/C is provided of 720 mm from the j end

Check for Deflection

IS 456-2000 cl.23.2.1

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

clear span=	4993.5	mm
width of support=	700	mm
1/12 of clear span=	416.13	mm

Since,

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

effective length(L_x)=	4993.5	mm
α=	26	
β=	1	span less than 10 m
δ=	1	not a flanged section

For γ

A_{sc} provided=	628.3185307	mm^2
% A_{sc} provided=	0.31	%

IS 456-2000 cl.23.2.1 fig 5

So,

γ= 2

For λ

$$fs = 0.58\, f_y \frac{Area\ of\ Steel\ \text{Re}quired}{Area\ of\ Steel\ \text{Pr}ovided}$$

A_{st} required=	144.000	mm^2
A_{st} provided=	628.3185307	mm^2

So,

fs=	66.463	N/mm^2
%st=	0.308	%

IS 456-2000 cl.23.2.1 fig 4

λ= 1.4

So,

αβγδλ= 72.8

$\frac{L_x}{d}$= 9.791 ≤ αβγδλ **(OK)**

u

Check for Development Length:

IS 456-2000 cl.26.2.1

$$Ld = \frac{\Phi \sigma_s}{4 \; x \; \tau_{bd}} = 970.98 \text{ mm} \quad \text{for tension}$$

$$Ld = \frac{\Phi \sigma_s}{4 \; x \; \tau_{bd}} = 776.7857 \text{ mm} \quad \text{for compression}$$

φ=	20	mm	(nominal diameter of bar)
σ_s=0.87*f_y=	435	N/mm^2	(stress in bars)
τ_{bd}=	1.4*1.6	N/mm^2	(design bond stress for tension)
τ_{bd}=	1.4*1.6*1.25	N/mm^2	(design bond stress for compressic

Also,

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

Where,

$$M = 0.87 * f_y * A_{stprvd} * (d - \frac{f_y * A_{stprvd}}{f_{ck} * b})$$

A_{st} provided=	1256.637061	mm^2	
M_1=	244438708.8	N-mm	(MOR offered by tension steel prov
V=	199334.5	N	(maximum shear force at that face
L_o=(sb/2-cc-3Φ) +8Φ			(additional anchorage length)
L_o=	275	mm	

So,

$$1.3 * \frac{M}{V} + L_o = 1869.156162 > \textbf{Ld} \quad \textbf{OK}$$

STAIRCASE DESIGN

Grade of concrete (f_{ck})	= 25 MPa
Yield strength of the steel (f_y)	= 500 MPa
Number of risers	= 9
Number of treads	= 10
Tread	= 280 mm
Riser	= 150 mm
Width of staircase slab	= 1.37 m

Let us assume that the width of the landing slab is 1.37m.

Effective Span (l) = 1370+(9*280) mm

=3890mm

(**From IS 456 Cl. 33.1**)

Density of concrete = 25 KN/m

Let us assume that the total depth of the waist slab is 175mm

Clear Cover(c/c) = 15 mm

Dia. Of rebar (Φ) = 12 mm

So, the effective depth of the slab = 175-12/2-15 = 154mm

CALCULATION

With R= 150 mm and T= 300mm, the inclined length of each step=($(150)^2+(280)^2)^{1/2}$=317.64mm

<u>Design of Going and landing slab B</u>

Step 1 : Effective span and depth of slab

The depth of both the landing and waist slab is assumed to be 175mm and effective depth as computed is 154mm.

Step 2: Calculation of loads

i) Loads on going (on projected plan area)

a) Self weight of waist slab = 25*.175*317.64/280 = 4.963 kN/m

b) Self weight of steps= 25*0.5*.15 = 1.875 kN/m

c) Floor finishes = 1.5 kN/m

d) Live load = 3 kN/m

Total = 11.338kN/m

Total Factored Loads = 1.5*11.338 = 17.007 kN/m

ii) Loads on landing slab

a) Self weight of landing slab=25*.175 = 4.375kN/m

b) Floor finishes = 1.5kN/m

c) Live load = 3kN/m

Total Factored load = 1.5*8.875 = 13.3125 kN/m^2

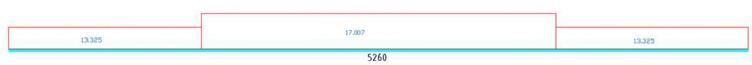


Fig: Loads are drawn

Step 3: Bending moment and shear force

$V_{max} = \frac{13.325*2*1.37+1\ \ .007*2.52}{2} = 39.68407kN$

M_{max} = 39.64*(1.37+2.52/2)-13.325*1.37*(1.37/2+2.52/2)-(17.007*2.52/2*.5*2.52/2)=55.3623 kNm

Step 4 : Checking of depth of slab

From the maximum moment, we get $d = \sqrt{\frac{M}{0.133 * fck * b}} = \sqrt{\frac{55.3623*10^6}{0.133*25*1000}}$

$= 129.037 < 154$ mm (ok)

Step 5: Determination of areas of steel reinforcement

From Annex-G,1.1(b)

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

$\therefore Ast$ =941.56mm^2

$\therefore Ast(min) = 0.12\%\ of\ bD$ =210mm^2

Adopt $\phi = 12\ mm$

Spacing = $\frac{1000}{941.56/(\frac{\pi}{4}*12^2)}$=120.1166$mm\ \leq 3d = 462mm$

$\leq$ 300mm

Provide 12 mm dia bar @90mm c/c.

$\therefore Ast(provided) = \frac{1000}{90*(\frac{\pi}{4}*12^2)}$ =1256.64mm^2

Distribution steel : The same value of distribution steel is provided for both the slab

Which is 0.12*175*1000/100=210mm^2

Provide 8 mm dia bars @ 160mm c/c spacing.

Step 6: Checking of deflection

l/d=3890/154=25.259

$\alpha = 20$

$\beta = 1$

$\lambda = 1$

$\delta = 1$

For γ

$100\frac{Ast(provided)}{bd} =$ 0.71%

$f_s = 0.58 * f_y * \frac{A_{st,req}}{A_{st,provided}}$ = 0.58*500*941.56/1256.865=217.285Mpa

$\gamma = 1.3$

$\alpha\beta\gamma\lambda\delta = 26$

$\therefore \frac{l}{d} \leq \alpha\beta\gamma\lambda\delta, ok.$

Step 7: Check for shear

$\tau_v = \frac{39.69*1000}{154*1000}$=0.258 N/mm^2

For the waist slab of depth 175 mm, K= 1.25 and $\tau_c = 1.25 * .29N/mm^2$= .3625 N/mm^2 > than τ_v (ok)

Hence, safe in shear

Step 8: Checking of development length

L$_d$ =$\phi * \sigma_s/(4\tau_{bd})$=582.59 mm

DESIGN OF BASEMENT WALL

INTRODUCTION

Basement wall is constructed to retain the earth and to prevent moisture from seeping into the building. Since the basement wall is supported by the mat foundation, the stability is ensured and the design of the basement wall is limited to the safe design of vertical stem.

Basement walls are exterior walls of underground structures (tunnels and other earth sheltered buildings), or retaining walls, which must resist lateral earth pressure as well as additional pressure due to other type of loading. Basement walls carry lateral earth pressure generally as vertical slabs supported by floor framing at the basement level and upper floor level. The axial forces in the floor structures are, in turn, either resisted by shear walls or balanced by the lateral earth pressure coming from the opposite side of the building.

The design is based on the assumption that the backfilling of soil is done after the construction of ground floor. This allows for the economy in design considering the wall to be supported by both the mat foundation as a cantilever action.

Known data:

Concrete Grade = M30

Steel Grade = Fe500

1. Design Constants

Floor to floor height, Basement height (h) = 3 m

Unit weight of soil, γ = 19 kN/m^3

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

Surcharge produced due to vehicular movement, W_s = 10 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) = 2.1 m

2. Moment calculation

Coefficient of Earth Pressure, $K_a = \dfrac{1-\text{Sin}\,\theta}{1+\text{Sin}\,\theta} = \dfrac{1-\text{Sin}\,30^\circ}{1+\text{Sin}\,30^\circ} = 0.333$

Lateral load due to soil pressure, $P_a = \dfrac{K_a * \gamma * h^2}{2} = \dfrac{0.333*19*2.1*2.1}{2} = 13.964 kN/m$

Lateral load due to surcharge load, $P_s = Ka * W_s * h = 0.333*10*2.1 = 7$ 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.

$M_c = \frac{Pa*h}{3} + \frac{Ps*h}{2} = \frac{13.964*2.1}{3} + \frac{7*2.1}{2} = 17.1248 kNm$/m

Design moment, M = 1.5 Mc =25.6782 kNm/m perimeter=90.5583

3. Approximate design of section

Let effective depth of wall = d

$$BM = 0.36\left(\frac{x_{u,\max}}{d}\right)\left(1-0.42\left(\frac{x_u}{d}\right)\right)f_{ck}bd^2 = 0.133 f_{ck}bd^2$$ (For Fe500) **(IS 456:2000, Annex G)**

or, $25.6782*10^6 = 0.133*30*1000*d^2$

or, d = 80.22 mm

Let clear cover is 40mm and bar size Φ is 16 mm.

Overall depth of wall, D = 80.22+40+8 = 128.22mm

Take D = 150mm

So, d = 150-40-8 = 102 mm

D = 150 mm < 200mm **(IS 456:2000, Cl. 32.5.1)**

So, single curtains of reinforcement need to be provided.

4. Calculation of main steel reinforcement

$$M_u = 0.87 * f_y * A_{st} * d\left(1-\frac{A_{st}f_y}{bdf_{ck}}\right)$$ **(IS 456:2000, Annex G)**

or, $25.6872 *10^6 = 0.87 * 500 * Ast * 102\left(1 - \frac{Ast*500}{1000*102*25}\right)$

or, Ast =665.87 mm^2

Minimum A_{st} = 0.0012*b*D = 0.0012*1000*150 = 180 mm^2 <A_{st} (OK) **(IS 456:2000, Cl. 26.5.2.1)**

Maximum diameter of bar = D/8 = 150/8 =18.75 mm > 16mm (OK)

Providing 16mm Φ bar,

Spacing of bars (S) = $1000*A_b/A_{st} = \frac{\Pi*16^2*1000}{4*665.87} = 301.9537$ mm/m

Providing 16mm Φ bar @ 200mm c/c

Maximum spacing = 3 * wall thickness or 450 = 450 mm **(IS 456:2000, Cl. 32.5.b)**

So, Provided $A_{st} = \frac{\Pi*16^2*1000}{4*200} = 1005.31\ mm^2$

$p_t = \frac{1005.31}{1000*102} * 100 = 0.985\% > 0.4\%$ (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 = 0.102m from the top of mat foundation i.e. 3-0.102 = 2.898 m below the top edge of wall.

Shear force at critical section is

$Vu = 1.5*(Ka*W_s*z + K_a*\gamma*(z^2/2)$

$= 1.5*(0.333*10*2.898+0.333*19*(2.898^2/2))$

= 54.33 Kn

Nominal shear stress, $\tau_v = \frac{Vu}{bd} = \frac{54.33*1000}{1000*102} = 0.5326 N/mm^2$ **(IS 456:2000, Cl. 31.6.2.1,40.3)**

Permissible shear stress, $\tau_c = 0.6358\ N/mm^2$ **(IS 456:2000, Table 19)**

Maximum shear stress, $\tau_{c,max} = 3.1\ N/mm^2$ **(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

= (3-0.55) + 0.102 = 2.552 m

Allowable deflection = L_{eff} /250 = 2055.2/250 = 8.2208 mm

$$Actualdeflection = \frac{P_s l_{eff}^3}{8EI} + \frac{P_a l_{eff}^3}{30EI}$$ **(IS 456:2000, Cl. 23.2.a)**

$$= \frac{7*2055.2^3*12}{8*5000\sqrt{25}*1000*15\ ^3} + \frac{13.964*2055.2^3*12}{30*5000\sqrt{25}*1000*150^3}$$

= 1.655mm

= 1.655 mm < 8.26 mm (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.**

Raft Foundation Design

f_y =	500	N/mm^2
f_{ck} =	25	N/mm^2
b=	1000	mm
Safe bearing capacity of soil=	150	KN/m^2

Column	Grid	vertical load P (KN)	Co-ordinate		P*x	P*y	Position
			x	y			
C11	A2	783.0115	122.8147	23.2879	96165	18235	corner
C12	A3	1890.2365	122.8147	28.9204	232149	54666	Side
C13	**A4**	1981.1674	122.8147	33.9969	243316	67354	Side
C31	A5	965.967	122.8147	38.9089	118635	37585	Corner
C15	B2	1735.6304	117.821	23.2879	204494	40419	Side
C16	B3	6030.45	117.821	28.9204	710514	174403	centre
C17	**B4**	4970.0957	117.821	33.9969	585582	168968	centre
C30	B5	1142.8744	117.821	38.9089	134655	44468	Side
C18	C1	1183.1234	112.655	19.5537	133285	23134	corner
C19	C2	1904.0813	112.655	23.2879	214504	44342	corner
C20	C3	5434.6291	112.655	28.9204	612238	157172	centre
C21	**C4**	4523.4883	112.655	33.9969	509594	153785	centre
C07	C5	1136.7975	112.655	38.9089	128066	44232	corner
C06	C6	1057.4982	112.655	45.257	119132	47859	Side
C01	C7	1108.701	112.655	49.3483	124901	54713	corner
C22	D1	1140.2378	108.409	19.5537	123612	22296	Side
C23	D2	3296.1501	108.409	23.2879	357332	76760	centre
C24	D3	3696.592	108.409	28.9204	400744	106907	centre
C08	**D4**	3384.8802	108.409	33.9969	366951	115075	centre
C09	D5	3645.4917	108.409	38.9089	395204	141842	centre
C05	D6	3165.0522	108.409	45.257	343120	143241	centre
C02	D7	1027.9623	108.409	49.3483	111440	50728	side
C25	E1	1145.6364	104.792	19.5537	120054	22401	corner
C26	E2	994.9365	104.792	23.2879	104261	23170	Side
C27	E3	994.8282	104.792	28.9204	104250	28771	Side
C28	**E4**	955.293	104.792	33.9969	100107	32477	Side
C10	E5	996.5532	104.792	38.9089	104431	38775	Side
C04	E6	1063.0687	104.792	45.257	111401	48111	Side
C03	E7	1130.0484	104.792	49.3483	118420	55766	COrner
	∑	62484.4824			7028557	2037654	
		So,					
			P(Service load) =	41656.322	KN		

Type Of Foundation

f_y = 500 N/mm^2

f_{ck} = 25 N/mm^2

b= 1000 mm

Total area below the center of columns= 392.99008 m^2

Maximum load under a column(P)= 6030.45 KN

safe bearing capacity of soil(σ)= 225 KN/m^2

As per IS 1893: 1 Cl 6.3.5.2, allowable bearing pressure in soil can be increased depending upon type of foundation thus bearing capacity of soil is increased by 50% assuming it will be raft foundation

Area required below the single isolated column(A)= $\frac{P}{\sigma}$

= 26.80 m^2

no of columns(n)= 29

so,

Total area required below for all the isolated column(A*n)= 777.258 m^2

196.4950379

Since, the total size of the footing required is more than 0.5 * total area available accommodate separately, it is preferred to select the Mat Foundation encompassing whole of the floor of the building as a block.

Position	Maxm (P)	Grid
Corner	1904.08	C2
Side	1981.17	A4
Center	6030.45	B3

Coordinates WRT Center of Mass as Origin

Grid	Coordinate		σ	Remark
	X	Y		
A2	10.62	-9.87	155.76	OK
A3	10.62	-4.24	136.09	
A4	10.62	0.84	118.35	
A5	10.62	5.75	101.19	
B2	5.62	-9.87	148.57	
B3	5.62	-4.24	128.89	
B4	5.62	0.84	111.16	
B5	5.62	5.75	94.00	
C1	0.46	-13.60	154.18	
C2	0.46	-9.87	141.13	
C3	0.46	-4.24	121.45	
C4	0.46	0.84	103.72	
C5	0.46	5.75	86.56	
C6	0.46	12.10	64.38	
C7	0.46	16.19	50.08	
D1	-3.79	-13.60	148.06	
D2	-3.79	-9.87	135.01	
D3	-3.79	-4.24	115.34	
D4	-3.79	0.84	97.60	
D5	-3.79	5.75	80.44	
D6	-3.79	12.10	58.26	
D7	-3.79	16.19	43.97	
E1	-7.41	-13.60	142.85	
E2	-7.41	-9.87	129.80	
E3	-7.41	-4.24	110.13	
E4	-7.41	0.84	92.39	
E5	-7.41	5.75	75.23	
E6	-7.41	12.10	53.05	
E7	-7.41	16.19	38.76	

Center of Mass :		
Xm =	112.1997	m
Ym =	33.1559	m
Eccentricity		
e_x =	0.28514475	m
e_y =	-0.5453341	m

Center of Stiffness		
Xs =	112.4848448	m
Ys =	32.61056589	m
Moment of Inertia		
I_{xx} =	6504.509917	m^4
I_{yy} =	8194.935984	m^4

Moment		
M_{xx} =	-22724.672	KN-m
M_{yy} =	11804.4631	KN-m

FROM ETABS		
Sum Mx =	-8.0588	KN-m
Sum My =	-73.6184	KN-m

Where,

$$\sigma = \frac{P}{A} \pm \frac{M_{yy} * x}{I_{yy}} \pm \frac{M_{xx} * y}{I_{xx}}$$

$\frac{P}{A}$ = 105.998406 KN/m^2

$\frac{M_y}{I_{yy}}$ = 1.440458241 KN/m^3

$\frac{M_x}{I_{xx}}$ = -3.493679315 KN/m^3

Now,

Dividing raft foundation into 4 strips along X-axis

The Bending Moment is obtained by using a coefficient of **0.1** and L as center of column distance $M_{max} = \frac{wL^2}{10}$

Maximum moment along X-direction

Beam	Width(m)	L max(m)	q(KN/m^2)	M_{max} (KN-m/m)	Span Moment (KNm/m)		Support moment KNm/m
1	1.867	4.2461	154.176	277.969	231.641		277.969
2	4.683	5.1661	155.764	415.713	346.428		415.713
3	5.355	5.1661	136.086	363.195	302.663		363.195
4	4.994	5.1661	118.351	315.861	263.218		315.861
5	5.630	5.1661	101.190	270.061	225.051		270.061
6	5.220	4.2461	64.377	116.067	96.723		116.067
7	2.04565	4.2461	50.08	90.297	75.24734913		90.29681895

Dividing raft foundation into 4 strips along Y-axis

The Bending Moment is obtained by using a coefficient of **0.1** and L as center of column distance $M_{max} = \frac{wL^2}{10}$

Maximum moment along Y-direction

Beam	Width(m)	L(m)	q(KN/m^2)	M_{max} (KN-m/m)	Span Moment		Support moment
A	2.497	5.6325	155.764	494.164	411.803		494.164
B	5.080	5.6325	148.571	471.343	392.786		471.343
C	4.706	6.3481	154.176	621.304	517.753		621.304
D	3.932	6.3481	148.060	596.657	497.214		596.657
E	1.809	6.3481	142.850	575.661	479.717		575.661

CALCULATION OF DEPTH OF RAFT FOUNDATION

Along X-direction

w=	155.764	KN/m^2
L=	5.1661	m

From, IS 456-2000 Table 12

Max support moment(M_s)

$$M_s = \frac{wL^2}{10} = \quad 415.713 \text{ KN-m/m}$$

Max span moment(M_m)

$$M_m = \frac{wL^2}{12} = \quad 346.428 \text{ KN-m/m}$$

Along Y-direction

w=	155.764	KN/m^2
L=	6.3481	m

Max support moment(M_s)

$$M_s = \frac{wL^2}{10} = \quad 621.304 \text{ KN-m/m}$$

Max span moment(M_m)

$$M_m = \frac{wL^2}{12} = \quad 517.753 \text{ KN-m/m}$$

Depth by momemt consideration

From SP 16 Table D

$$d = \sqrt{\frac{M}{0.133 * f_{ck} * b}} = \quad 432.2711798 \text{ mm}$$

Adopt effective depth (d)= 1040 mm

Overall depth(D)

$$D = d + \frac{\phi}{2} + 50 = \quad 1100 \text{ mm}$$

The depth of the mat foundation will be governed by two way shear at one of the exterior columns. In each case location of critical shear is not obvious, it may be necessary to check all possible locations.

Check two way shear

From, IS 456-2000 Cl. 31.6.3.1

Shear strength of concrete (τc) = $0.25*(f_{ck})^{0.5}$

So,

τ_c= 1.250 N/mm^2

By selecting the critical column among (side , corner and center)
i.e. from the column with highest load ,the depth is checked

Critical column

1. For corner column (A1)

Column load = 1904.08

perimeter (b_o) = 2*(d/2+700)

b_o= 2440 mm

Nominal shear stress(t_v)

From, IS 456-2000 Cl. 31.6.2.1

$\tau_v = \frac{P_u}{b_o d} =$ 0.750 N/mm^2 **<τ_c (OK)**

2. For side column (C20)

Column load = 1981.17

perimeter (b_o) = 2*(.35+d/2)+(.7+d/2) mm

b_o= 2960 mm

Nominal shear stress(t_v)

$\tau_v = \frac{P_u}{b_o d} =$ 0.644 N/mm^2 **<τ_c (OK)**

3. For middle column (C40)

Column load = 6030.45

perimeter (b_o) = 4(d+700) mm

b_o= 6960 mm

Nominal shear stress(t_v)

$\tau_v = \frac{P_u}{b_o d} =$ 0.833 N/mm^2 **<τ_c (OK)**

Reinforcement Calculation

Along X-direction

From IS456:2000, Cl. 32.5.a

At Bottom,

$$Ast = 0.5 * \frac{f_{ck}}{f_y} * bd * \left(1 - \sqrt{1 - \frac{4.6 * M}{f_{ck}\, bd^2}}\right)$$

Where,

M_u= 415.713 KN-m/m

f_y = 500 N/mm^2

d= 1040 mm

f_{ck} = 25 N/mm^2

b= 1000 mm

Solving, we get

A_{st}= **936.222331** mm^2

From, IS 456-2000 cl. 26.5.2.1

But,

A_{st}min= 0.12% * b * D

= **1320** mm^2 > 712.846 mm2 (NOT OK)

Adopt,

A_{st}= 1320 mm^2

Using 20 mm dia bars

Area of individual bar ($A_{b)}$= 314.159 mm^2

Spacing= 237.9992424 mm

adopt spacing($S_{v)}$= 200 mm

From IS456:2000, Cl. 32.5.a

At Top,

$$Ast = 0.5 * \frac{f_{ck}}{f_y} * bd * \left(1 - \sqrt{1 - \frac{4.6 * M}{f_{ck}\, bd^2}}\right)$$

Where,

M_u= 346.428 KN-m/m

f_y = 500 N/mm^2

d= 1040 mm

f_{ck} = 25 N/mm^2

b= 1000 mm

Solving, we get

A_{st}= **777.7718563** mm^2

From, IS 456-2000 cl. 26.5.2.1

But,

A_{st}min= 0.12% * b * D

= **1320** mm^2 > 592.886 mm^2 (NOT OK)

Adopt,

A_{st}= 1320 mm^2

Using 20 mm dia bars

area of individual bar ($A_{b)}$= 314.159 mm^2

Spacing= 237.9992424 mm

adopt spacing(S_v)= 200 mm

Provide 20mm φ bar at 200 mm c/c at bottom and top in x-direction

Along Y-direction

From IS456:2000, Cl. 32.5.a

At Bottom,

$$Ast = 0.5 * \frac{f_{ck}}{f_y} * bd * \left(1 - \sqrt{1 - \frac{4.6 * M}{f_{ck}\, bd^2}}\right)$$

Where,

M_u=	621.304	KN-m/m
f_y =	500	N/mm^2
d=	1040	mm
f_{ck} =	25	N/mm^2
b=	1000	mm

Solving, we get

A_{st}= **1412.400896** mm^2

From, IS 456-2000 cl. 26.5.2.1

But,

A_{st}min= 0.12% * b * D

= **1320** mm^2 <1327.202 mm^2 (OK)

Adopt,

A_{st}= 1412.400896 mm^2

Using 20 mm dia bars

area of individual bar (A_b)=	314.159	mm^2
Spacing=	222.4290574	mm
adopt spacing(S_v)=	200	mm

From IS456:2000, Cl. 32.5.a

At Top,

$$Ast = 0.5 * \frac{f_{ck}}{f_y} * bd * \left(1 - \sqrt{1 - \frac{4.6 * M}{f_{ck}\, bd^2}}\right)$$

Where,

M_u=	517.753	KN-m/m
f_y =	500	N/mm^2
d=	1040	mm
f_{ck} =	25	N/mm^2
b=	1000	mm

solving, we get

A_{st}= **1171.420539** mm^2

From, IS 456-2000 cl. 26.5.2.1

But,

$A_{st}min = 0.12\% * b * D$

$= \textbf{1320}$ $mm^2 > 1101.937\ mm^2$ (NOT OK)

Adopt,

$A_{st} = 1320$ mm^2

Using 20 mm dia bars

area of individual bar ($A_{b)}$=	314.159	mm^2
Spacing=	237.9992424	mm
adopt spacing($S_{v)}$=	200	mm

Provide 20mm φ bar at 200 mm c/c at bottom and top in y-direction

9 Discussion

Various discussions are given below in sub-heads:

9.1 Structural designs with or without seismic considerations

There are all-together 19 combinations among which Combo 1 doesn'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.

9.2 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.

9.3 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

10 Conclusion

During the course of this project different problems were encountered and solutions to these problems were effectively found under the guidance of Supervisor Er**. Subash Bastola.**

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 **"Earthquake Resistant Analysis and Design of Eight Storied Commercial Building"** helped us to understand the effect of earthquake load on structural elements and in structure as-a-whole.

10.1 Role of Cantilever Slab

Cantilever in buildings are used for projection purposes such as in balconies in which one end is supported and next has free end. Cantilever slabs are those transferring loads by cantilever action.

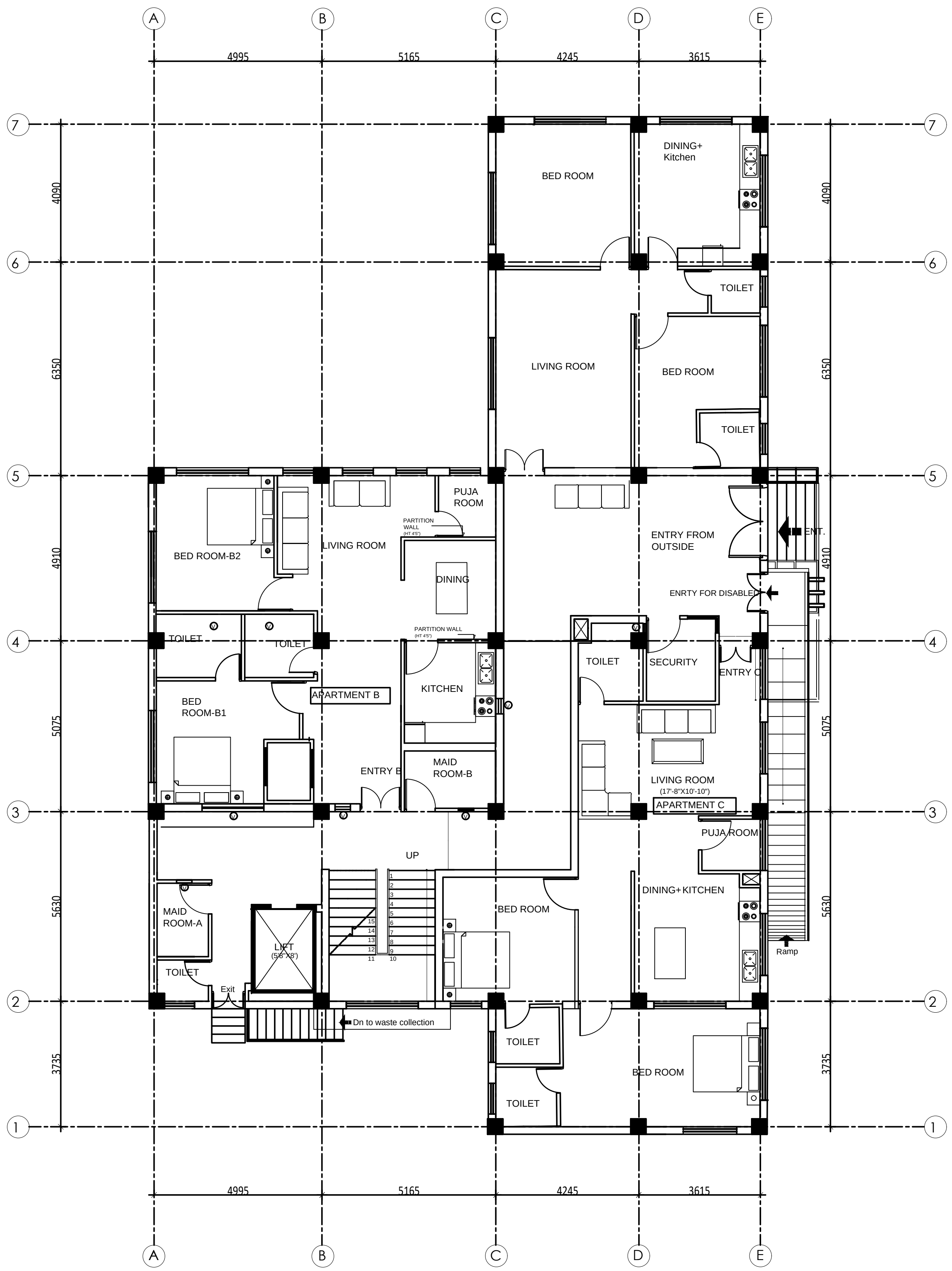
During analysis of our models with cantilever slab and with-out them, it was found out that in models with cantilever slab, rebar percentage was more than that in without them. It also played significant role in increasing eccentricity and hence torsion in buildings. Moreover, difference in percentage of rebar in these models decreased as structural element near stiffness was compared. **From it, we can conclude that use of cantilever projections should be minimized as far as possible.**

10.2 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.

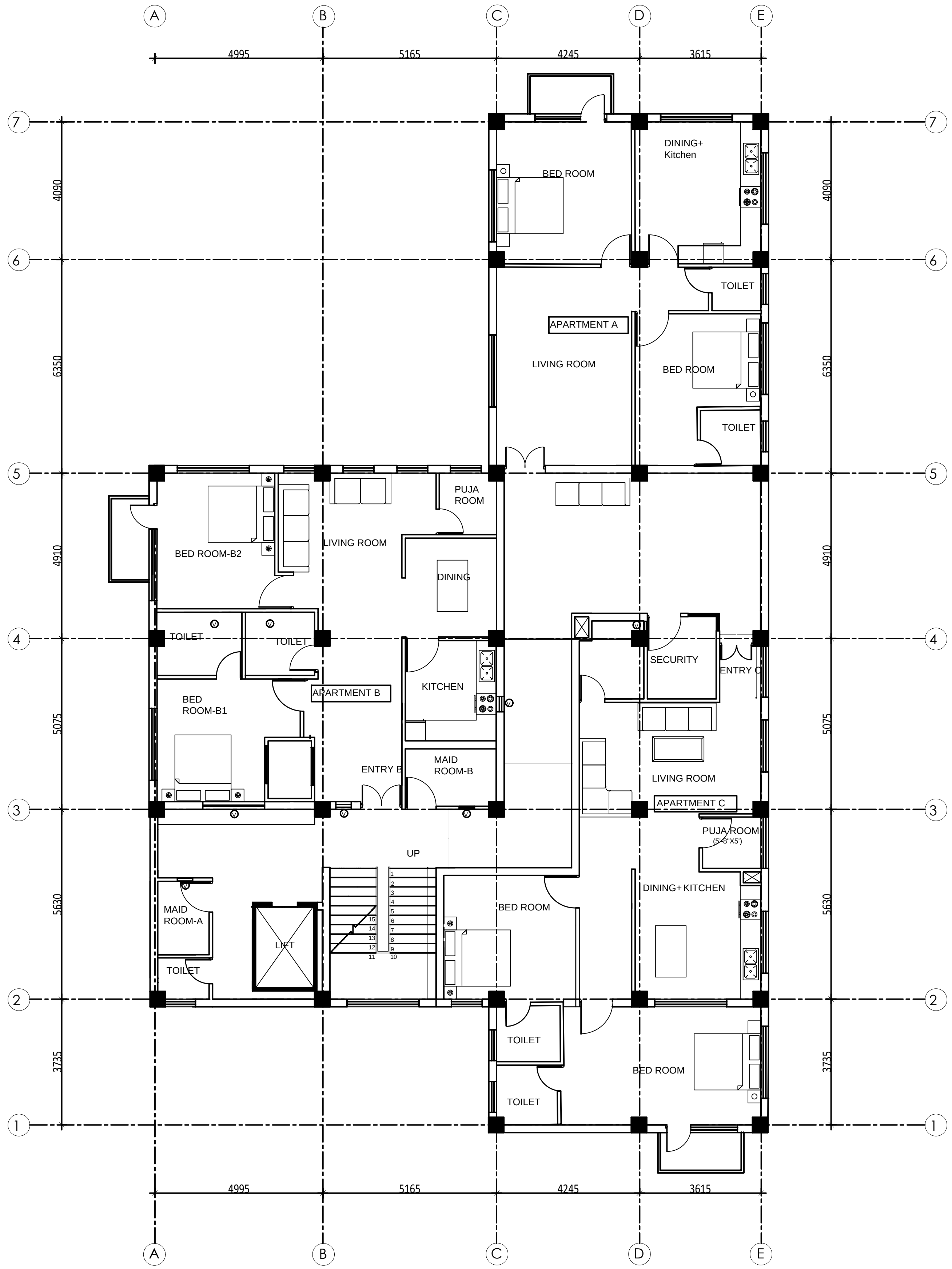
10.3 Percentage of Reinforcement

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.



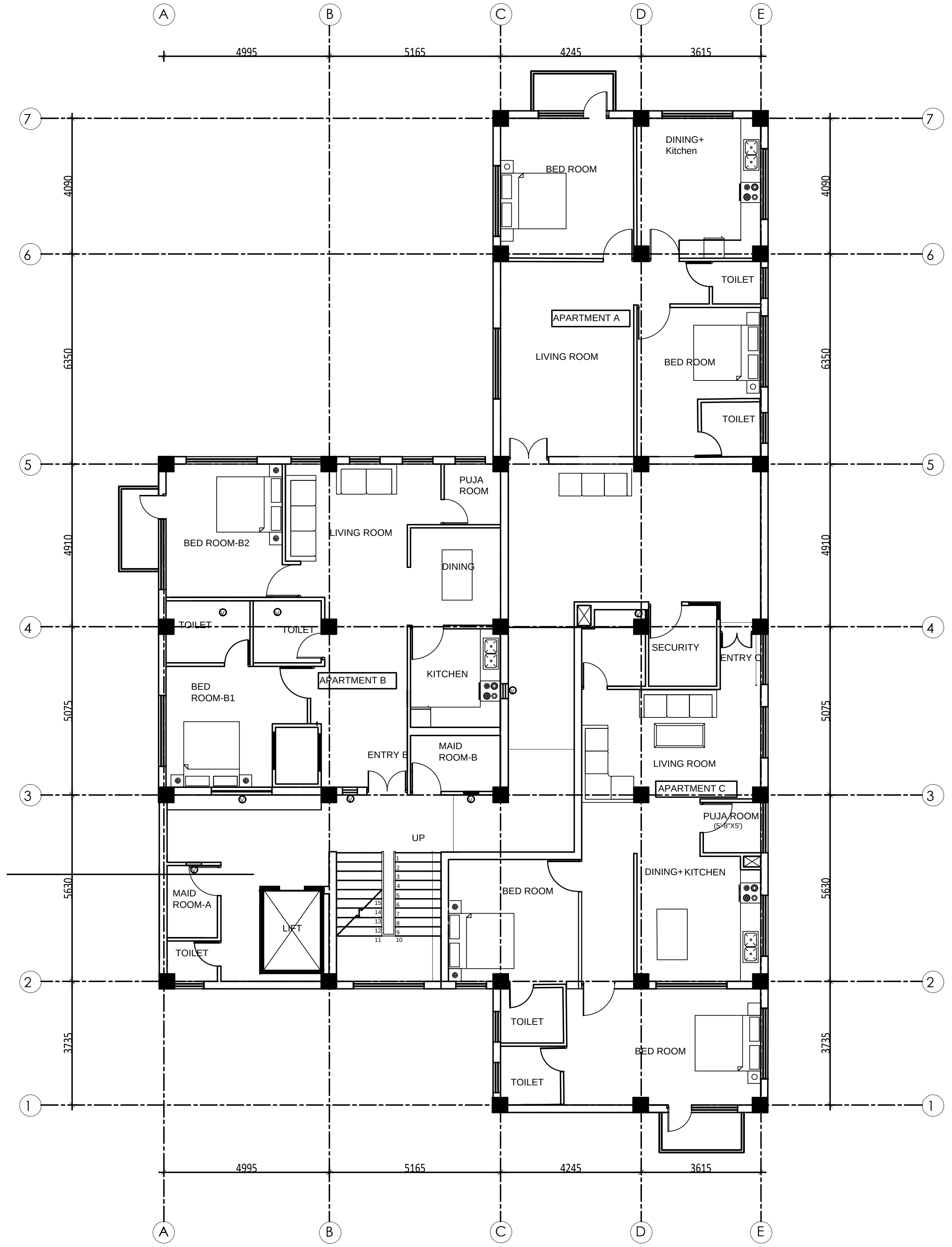
Ground Floor

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE PLAN	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



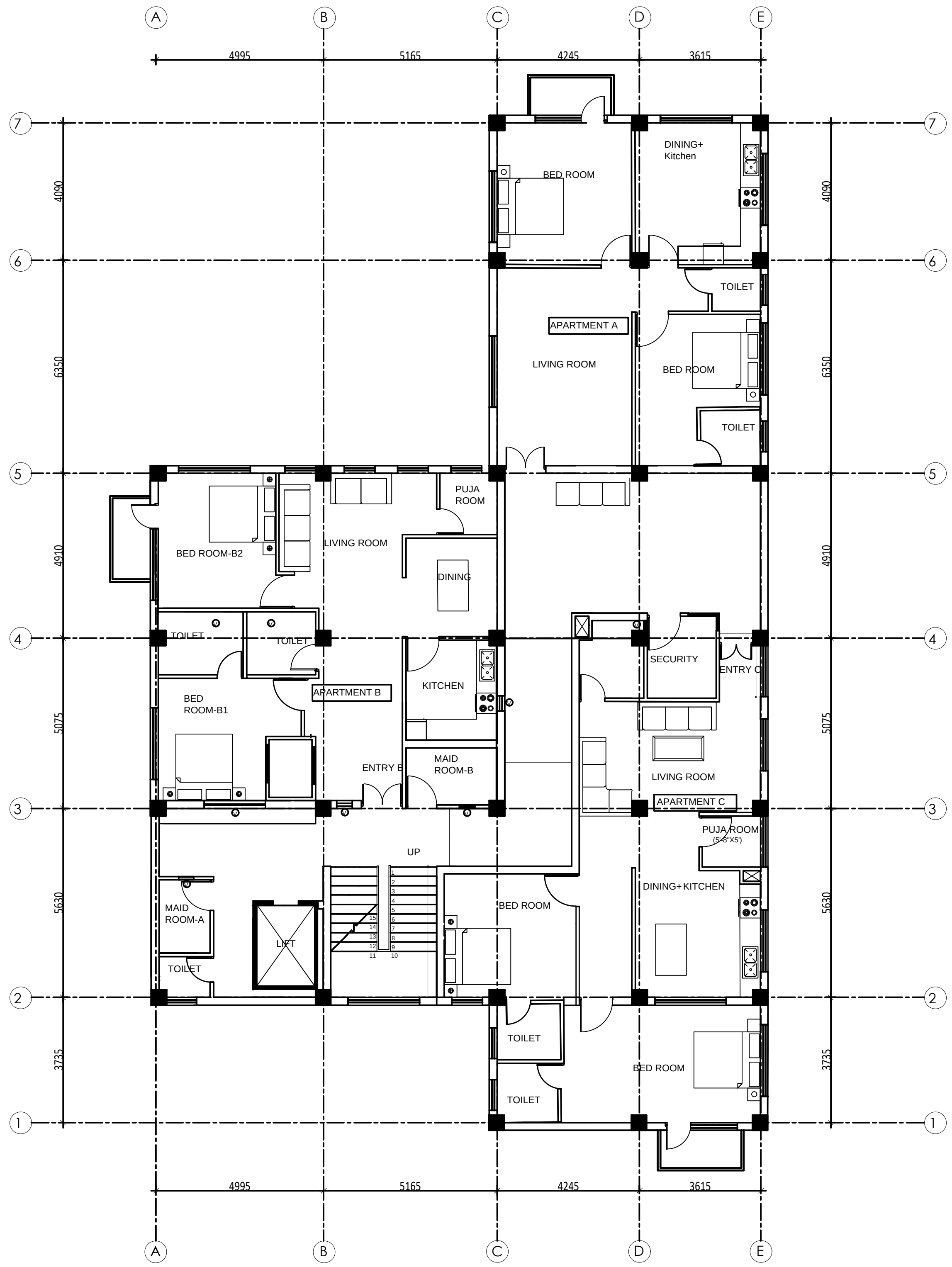
1st Floor

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE PLAN	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



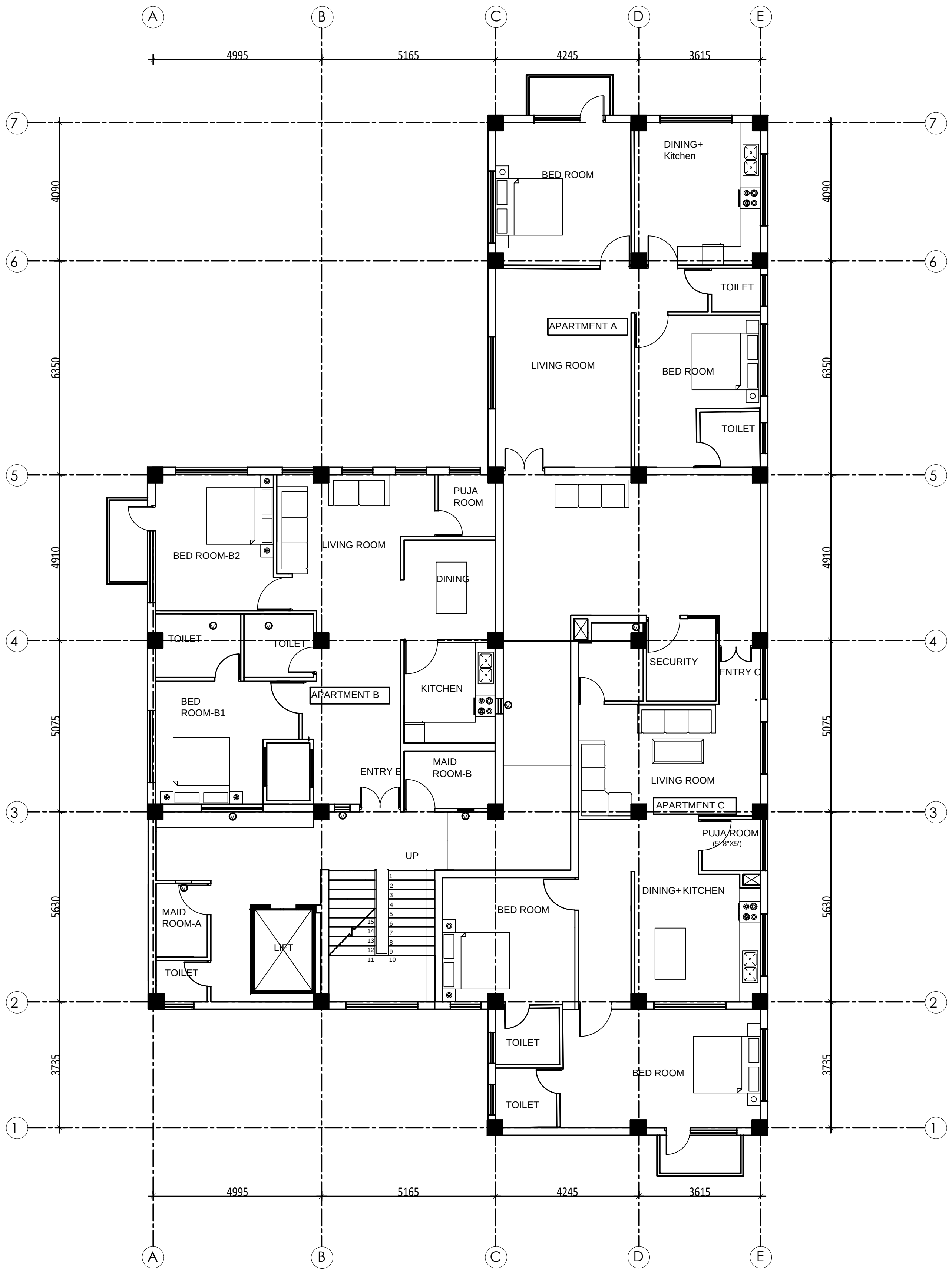
2nd Floor

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE PLAN	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



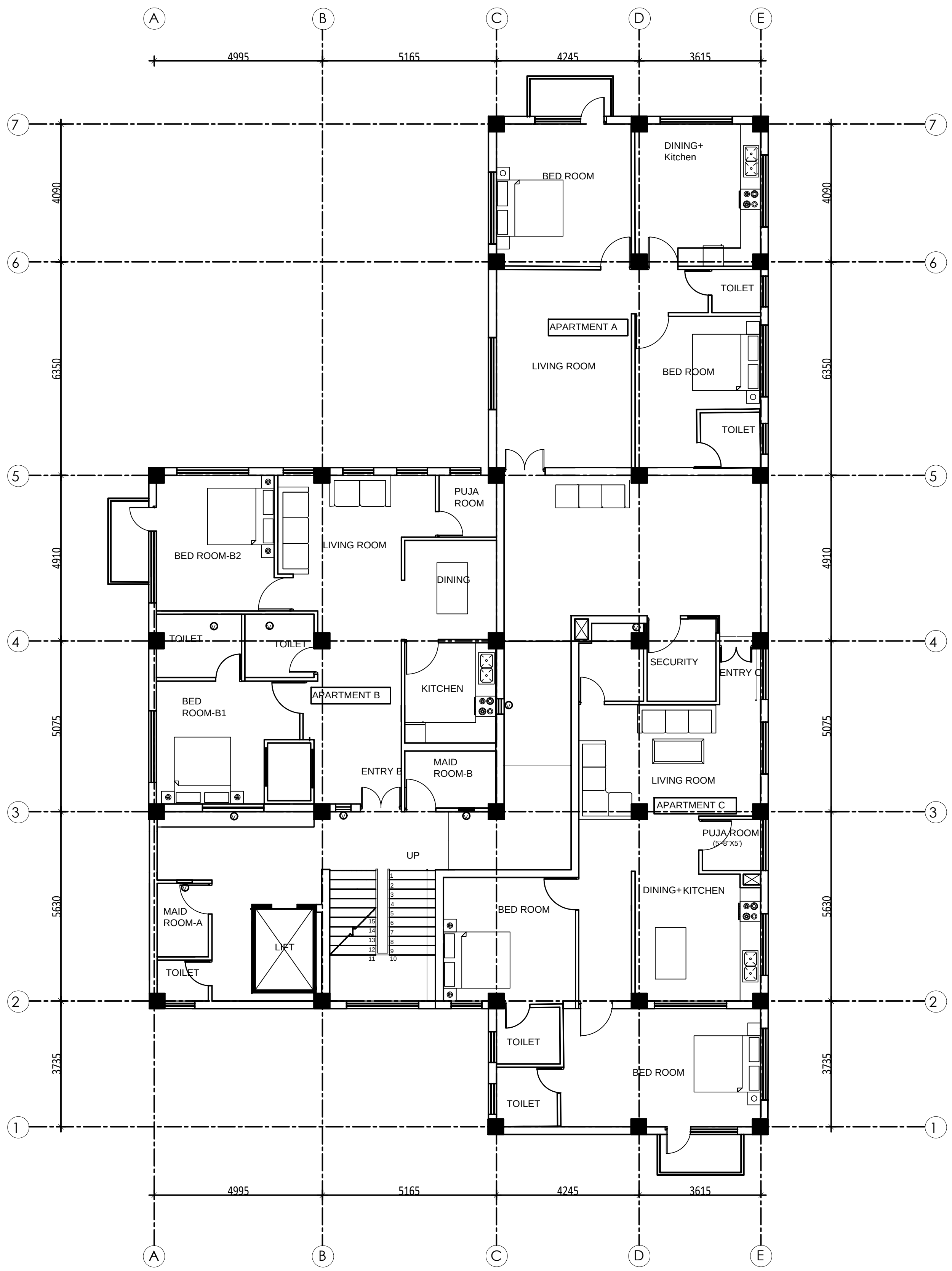
3rd Floor

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE PLAN	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



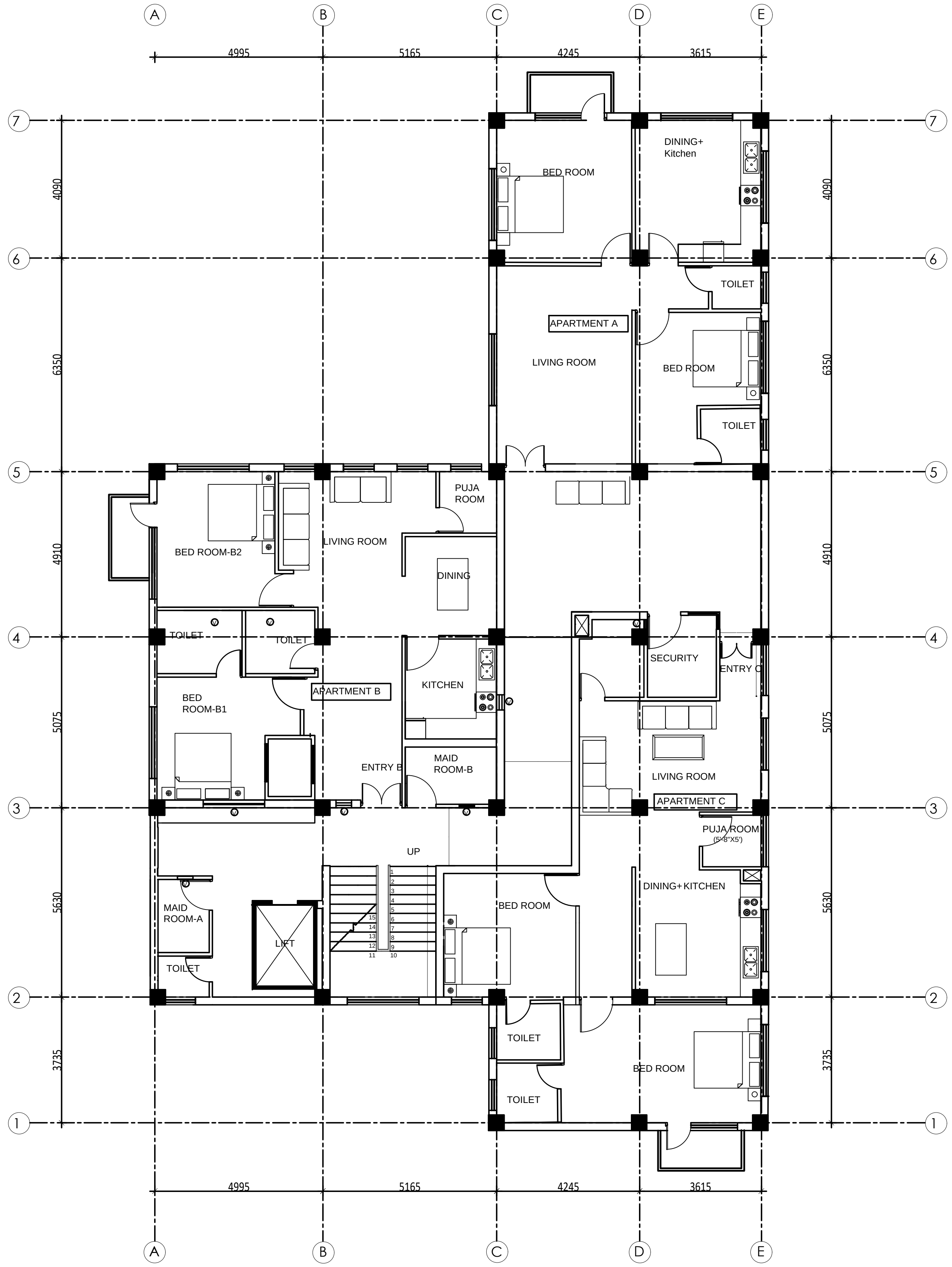
5th Floor

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE PLAN	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



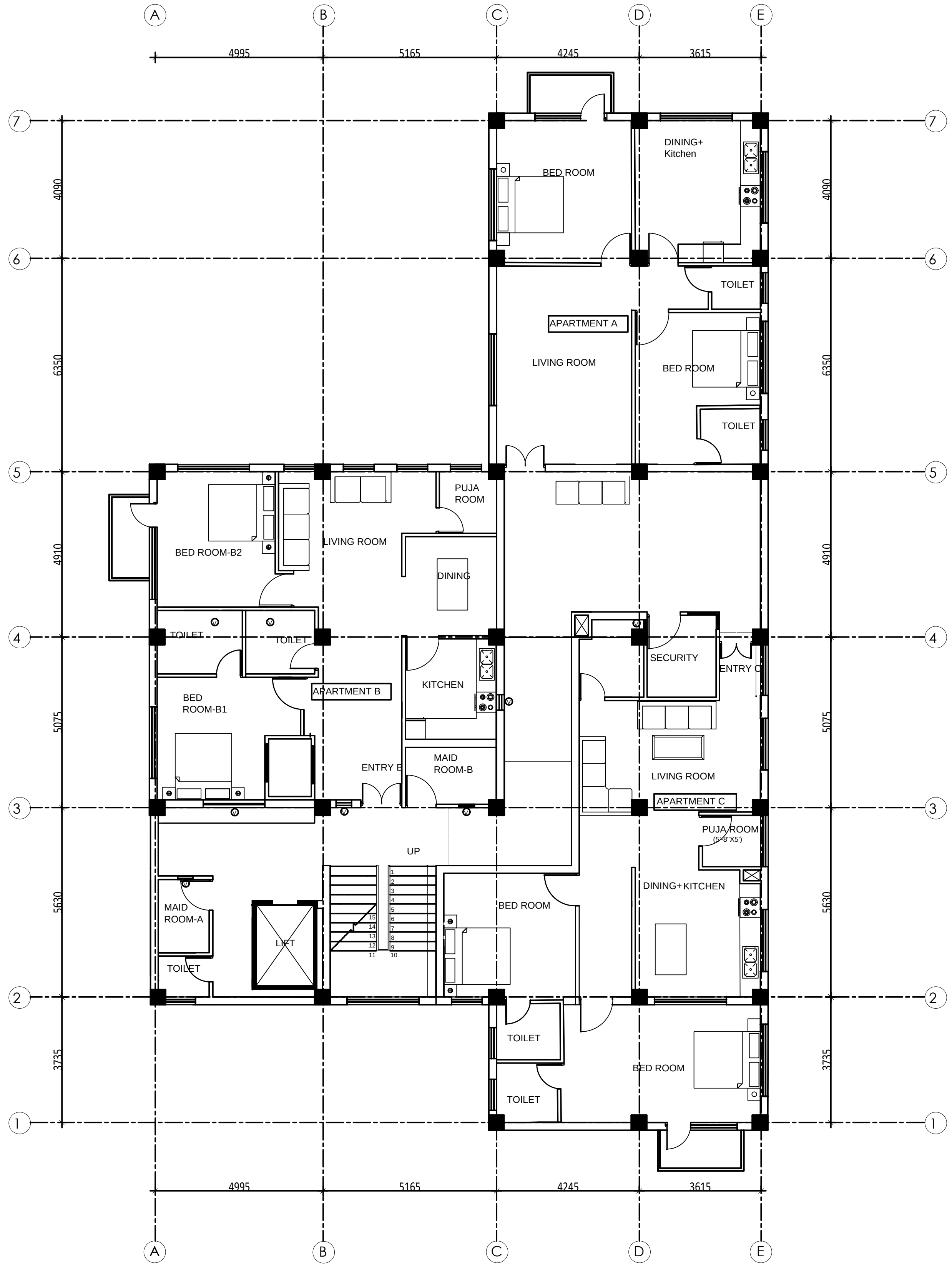
4th Floor

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE PLAN	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



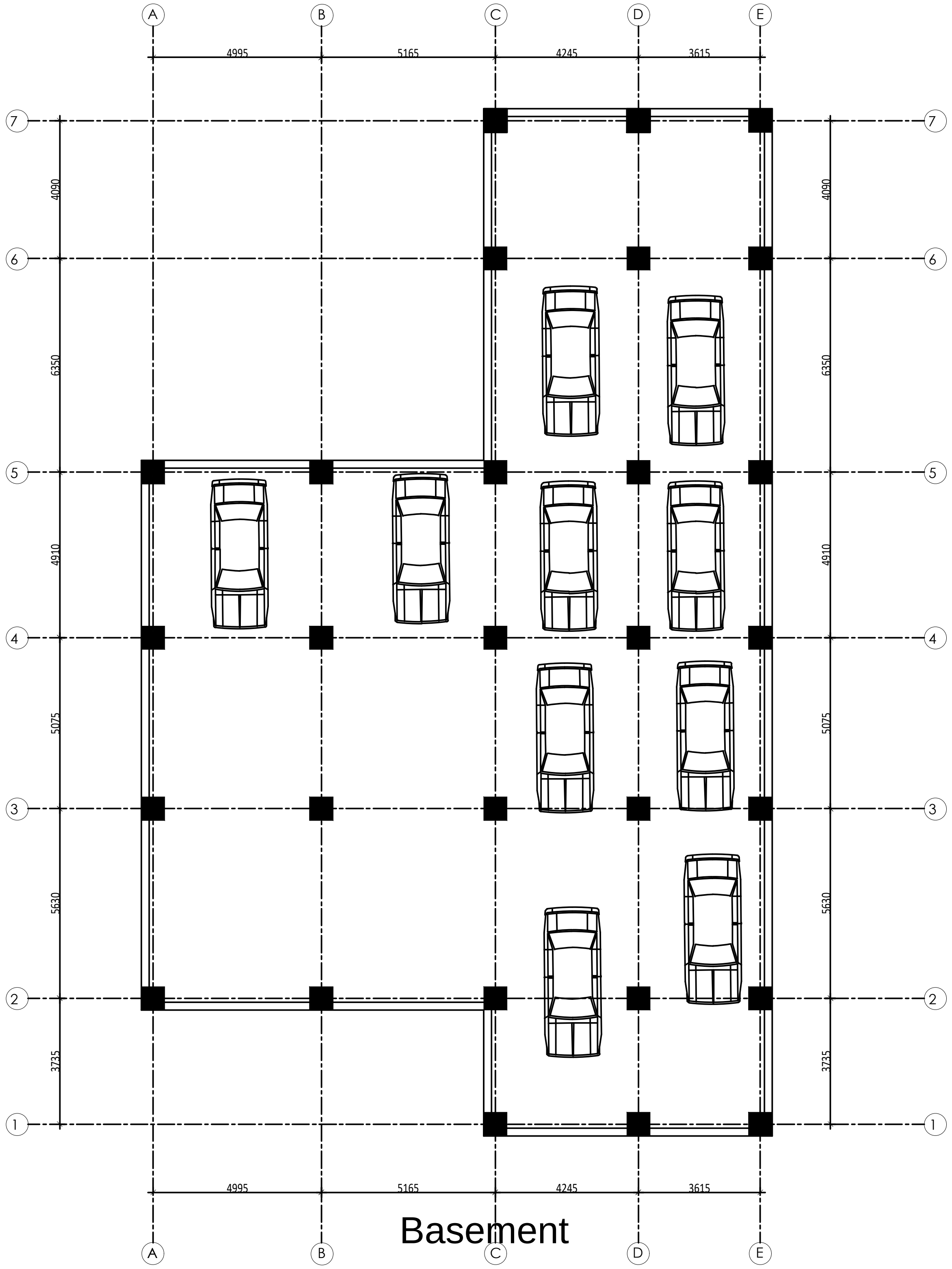
6th Floor

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE PLAN	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



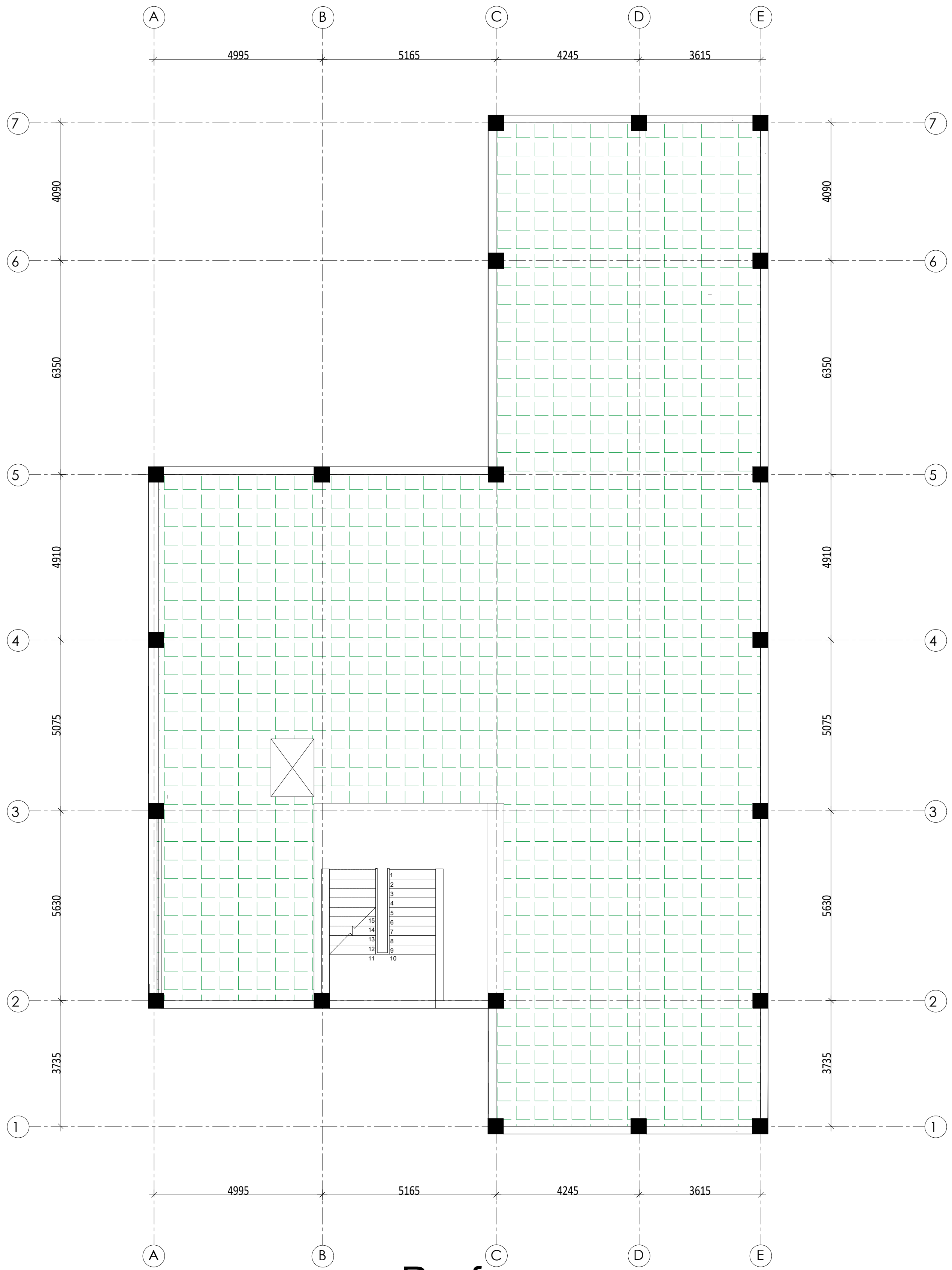
7th Floor

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE PLAN	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



Basement

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE PLAN	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



Roof

TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS

PROJECT TITLE:
EARTHQUAKE RESISTANT DESIGN OF
MULTISTORIED RESIDENTIAL BUILDING

SHEET TITLE
PLAN

GROUP MEMBERS:
AVINASH JHA- 075BCE031
BHUPENDRA DULAL- 075BCE034
BIBEK PANDEYA- 075BCE036
BIJAY NEPAL- 075BCE037
ALISH KHATIWADA- 075BCE012
DIPIKA PANDEY- 075BCE054

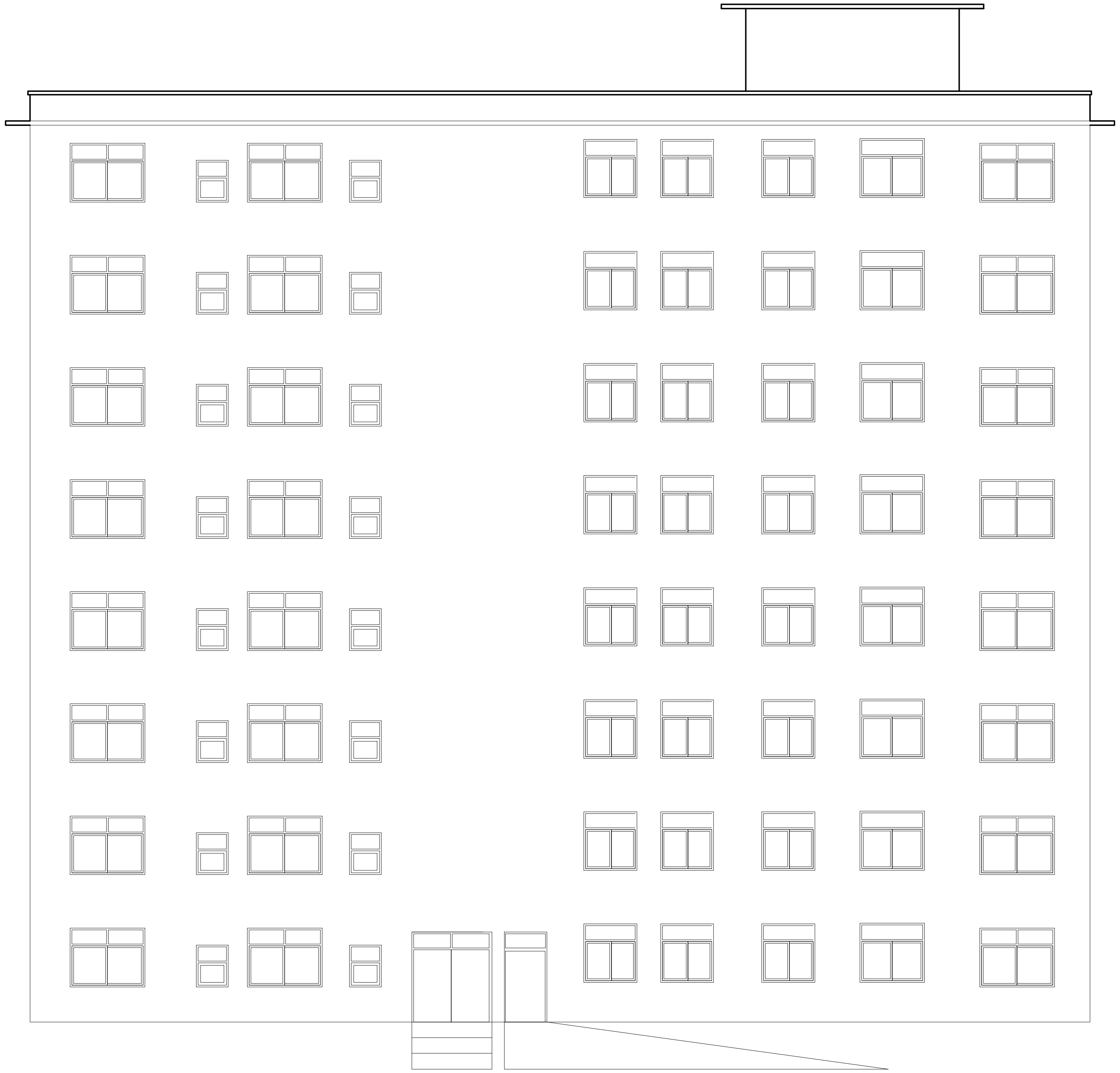
PROJECT SUPERVISOR :
ASST. PROF. SUBASH BASTOLA

Date : 2079 / 12 / 01

SCALE : NOT TO SCALE

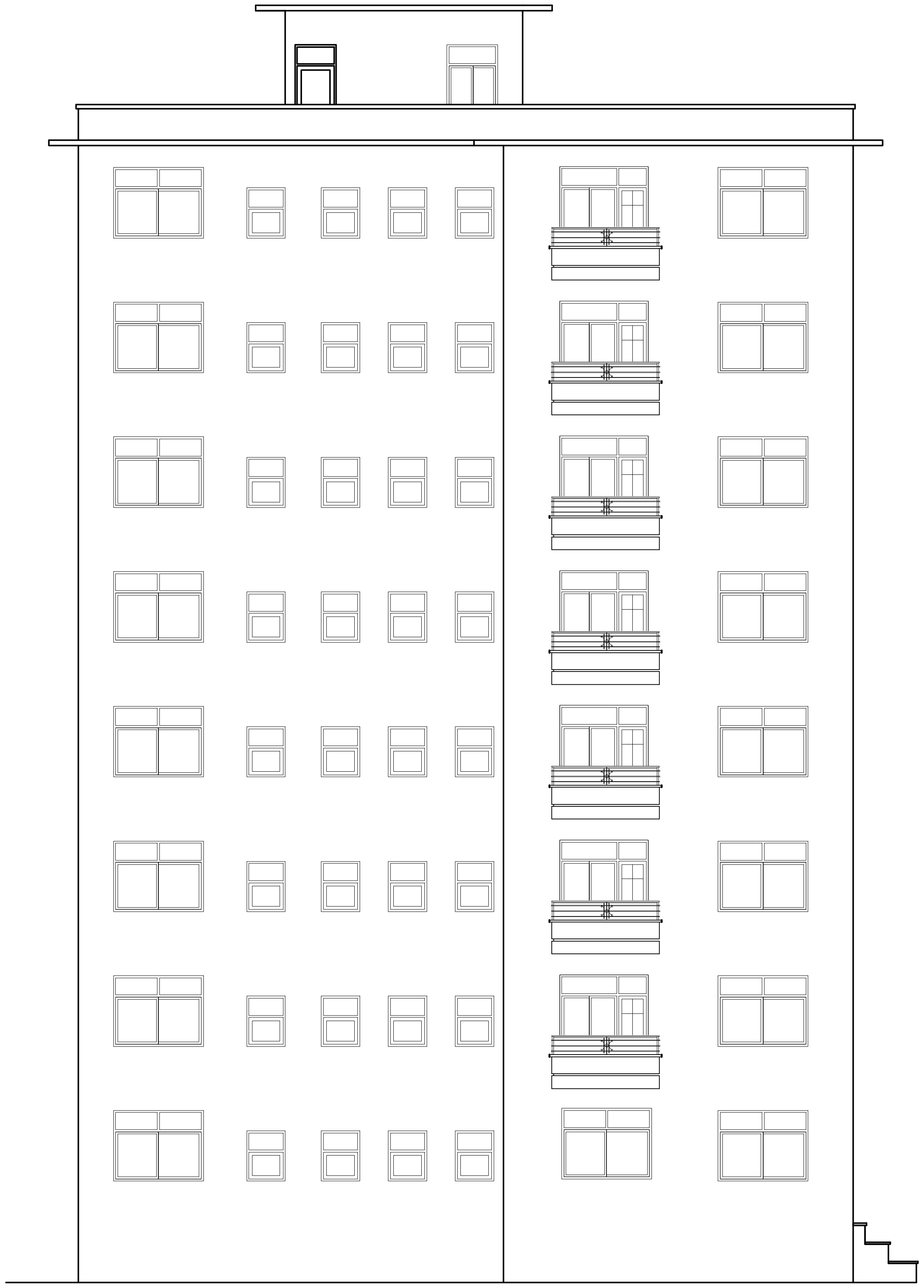
DWG No. :

Sheet No. :



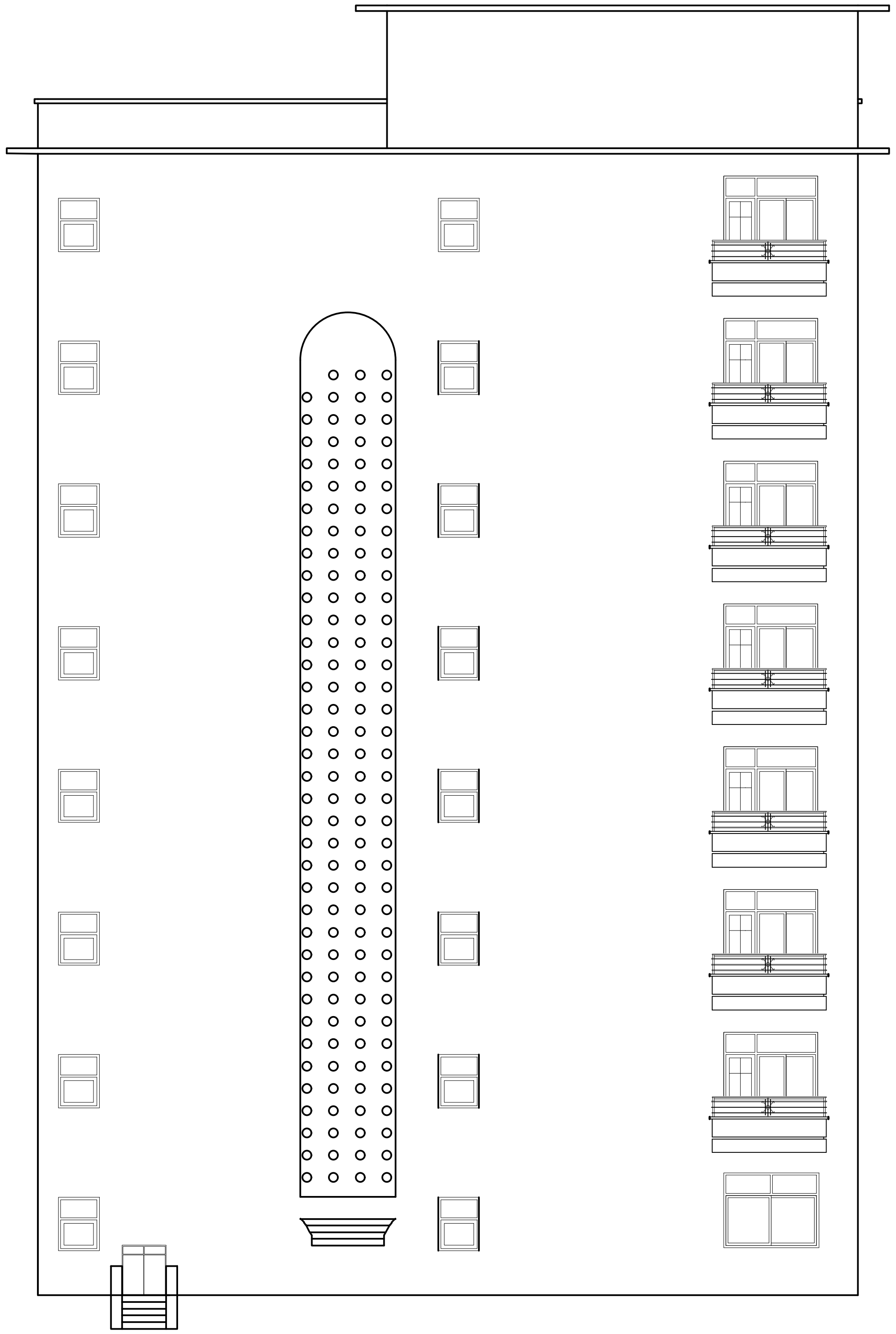
East Elevation

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE ELEVATION	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



North Elevation

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE ELEVATION	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :

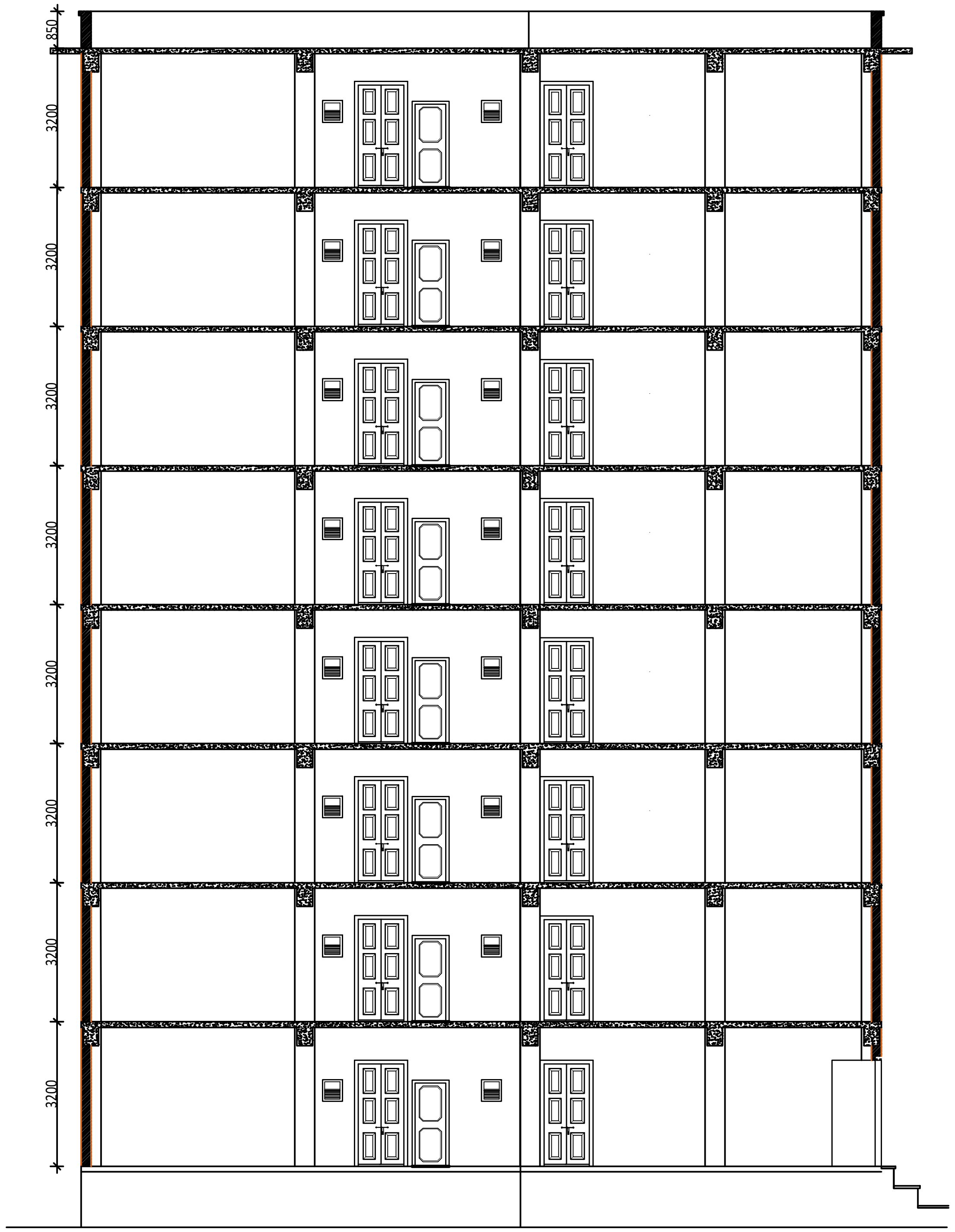


South Elevation

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE ELEVATION	<u>GROUP MEMBERS:</u> AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :

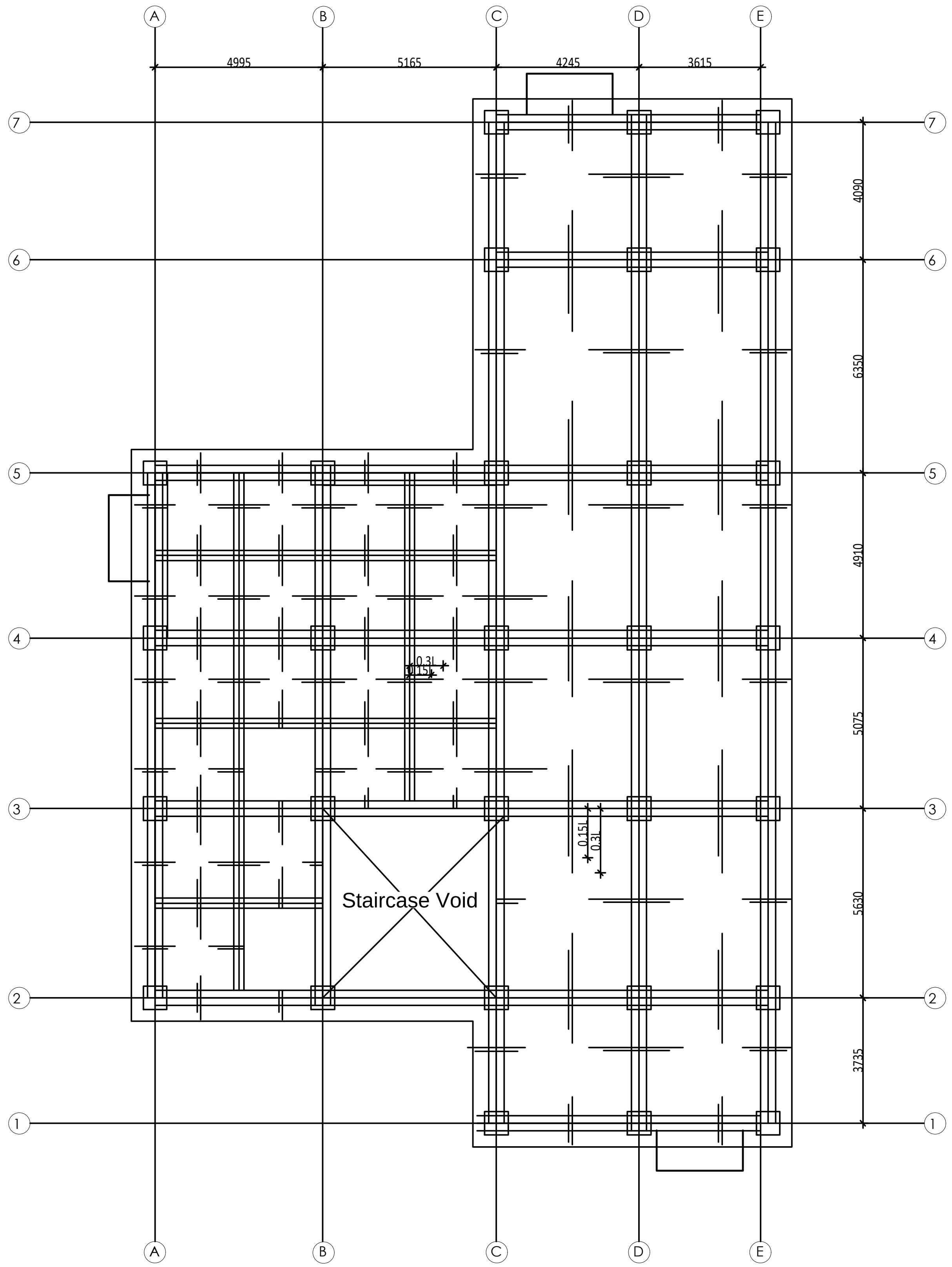
West Elevation

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE ELEVATION	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



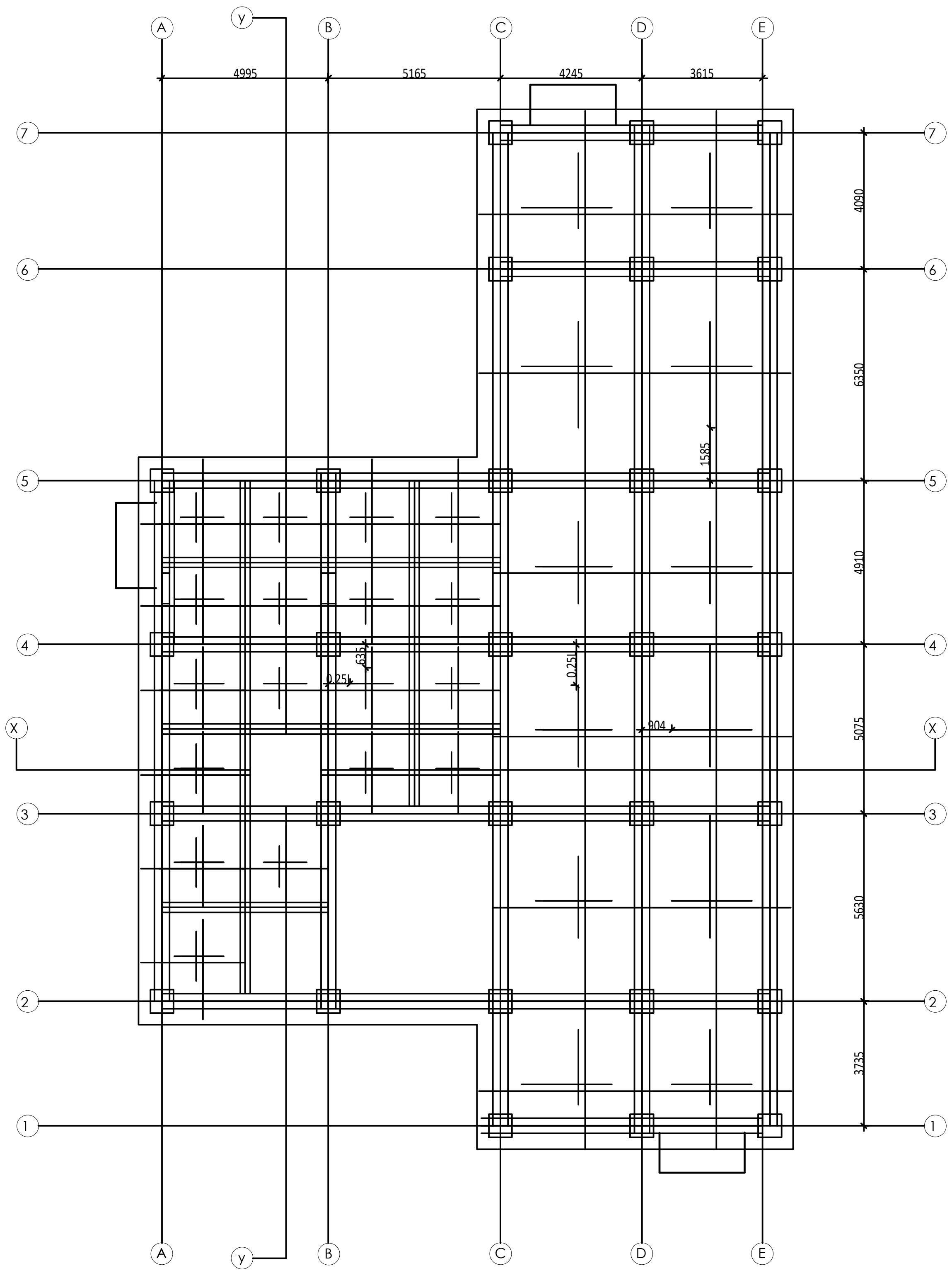
Section at X-X

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE SECTION	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



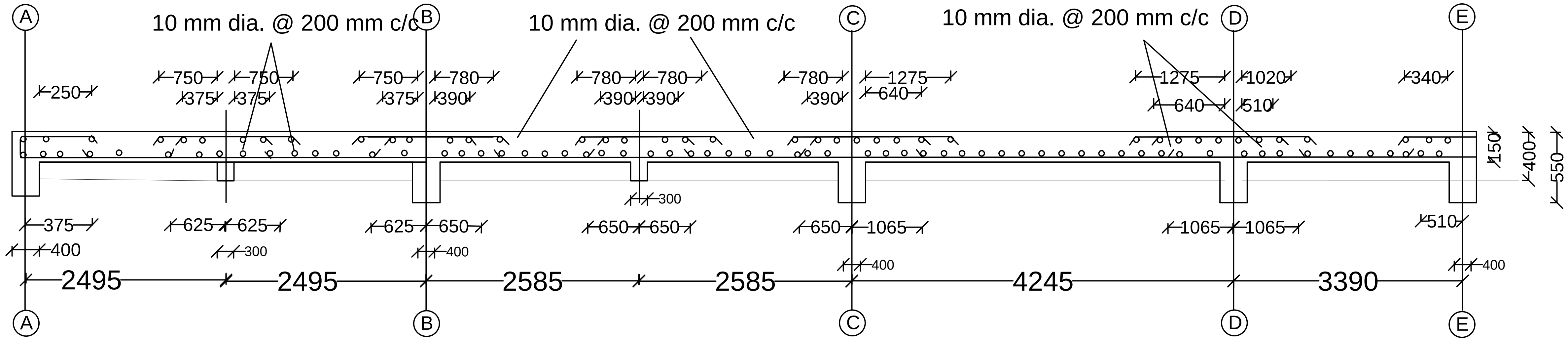
Top reinforcement detailing of slab

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :

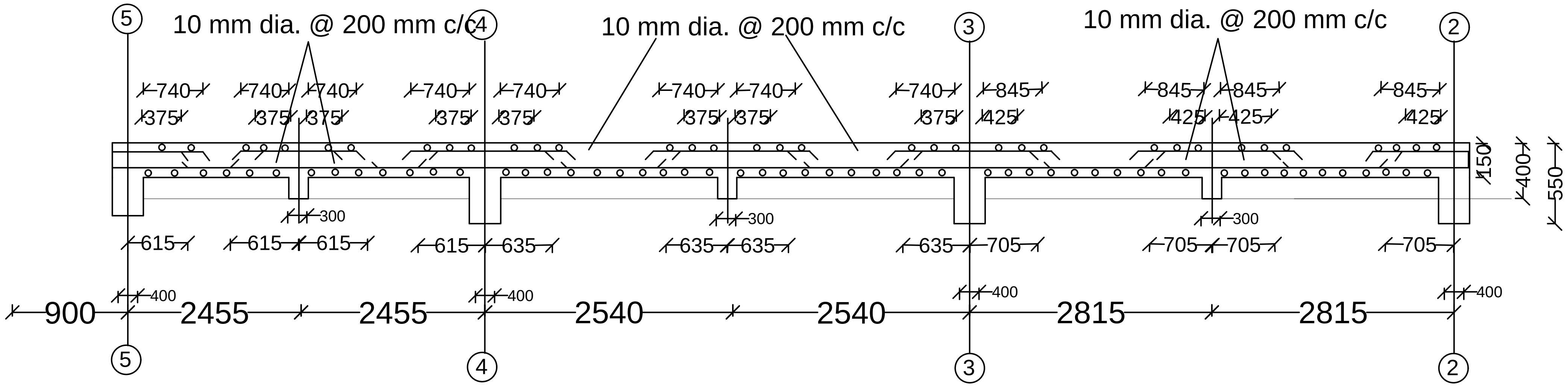


Bottom reinforcement detailing of slab

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE REINFORCEMENT DETAILING OF SLAB (PLAN)	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :

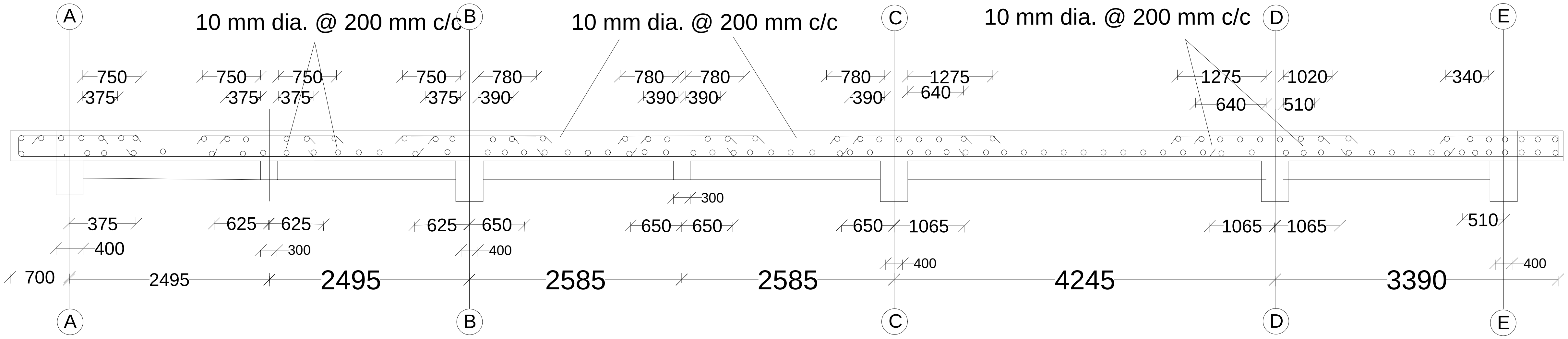


Section Along X-X

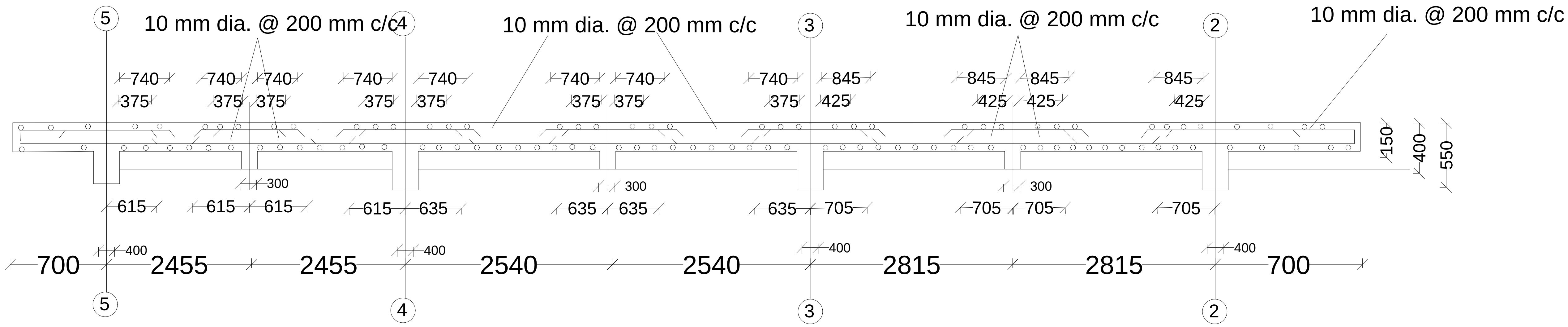


Section Along Y-Y

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE DETAILING OF SLAB SECTION	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :

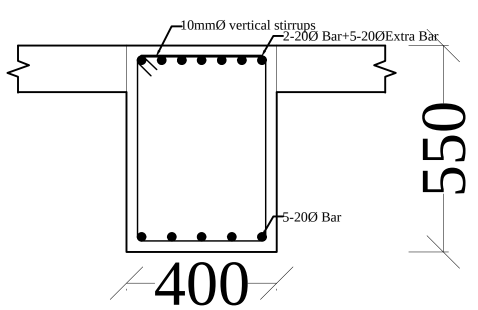


Section Along X-X for 8th floor

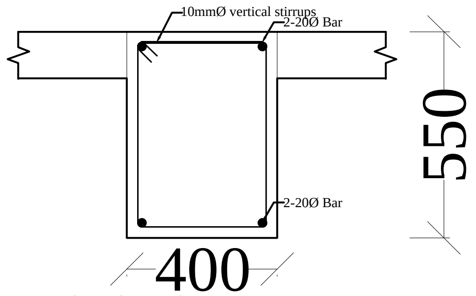


Section Along Y-Y for 8th floor

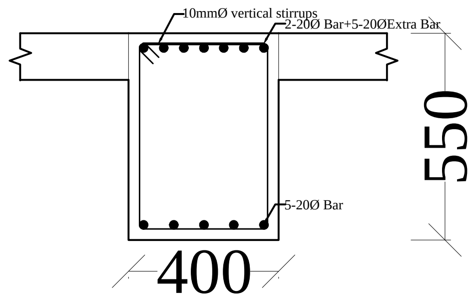
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE DETAILING OF SLAB SECTION	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



SECTION AT A-A

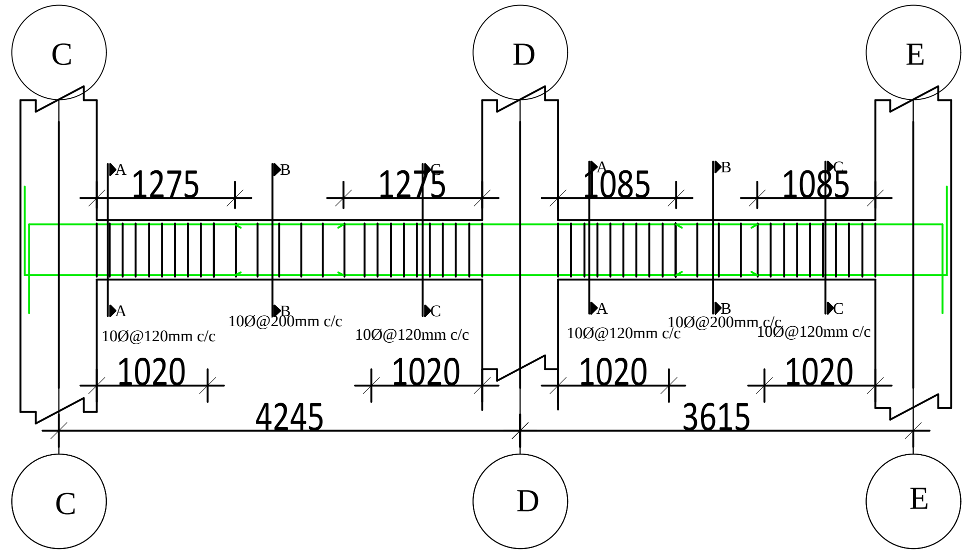


SECTION AT B-B



SECTION AT C-C

Beam cross-section along grid 1-1



Beam longitudinal section along grid 1-1

TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS

PROJECT TITLE:
EARTHQUAKE RESISTANT DESIGN OF
MULTISTORIED RESIDENTIAL BUILDING

SHEET TITLE
BEAM DETAILING

GROUP MEMBERS:
AVINASH JHA- 075BCE031
BHUPENDRA DULAL- 075BCE034
BIBEK PANDEYA- 075BCE036
BIJAY NEPAL- 075BCE037
ALISH KHATIWADA- 075BCE012
DIPIKA PANDEY- 075BCE054

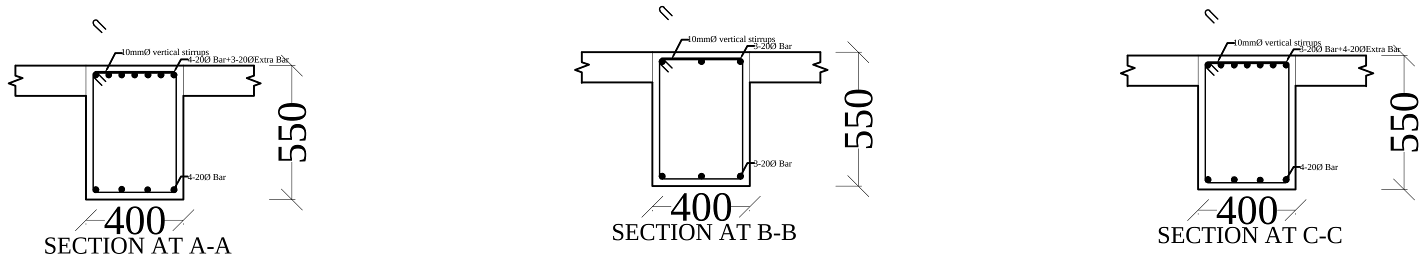
PROJECT SUPERVISOR :
ASST. PROF. SUBASH BASTOLA

Date : 2079 / 12 / 01

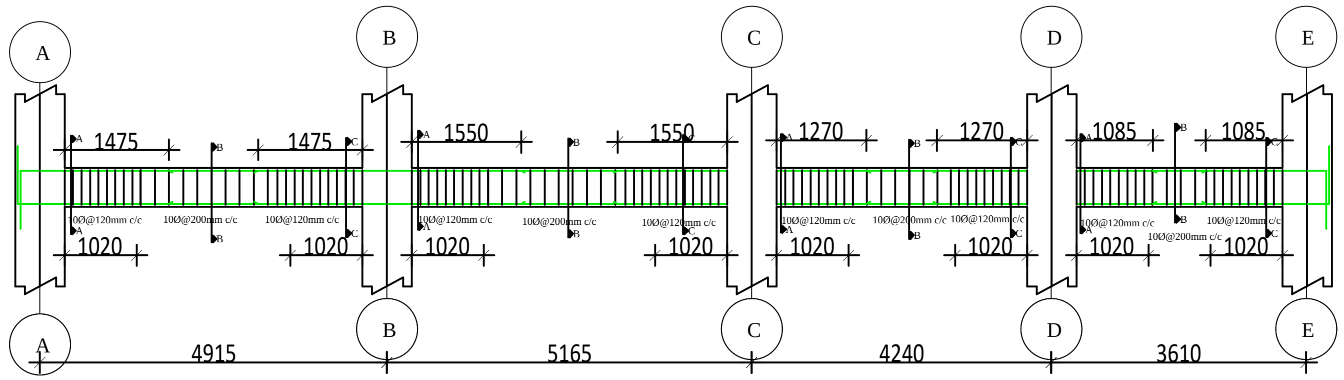
SCALE : NOT TO SCALE

DWG No. :

Sheet No. :

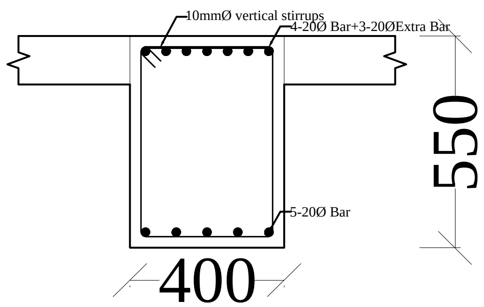


Beam cross-section along grid 2-2

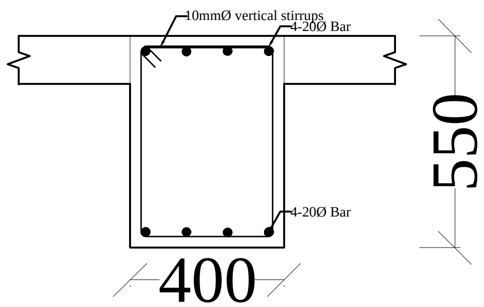


Beam longitudinal section along grid 2-2

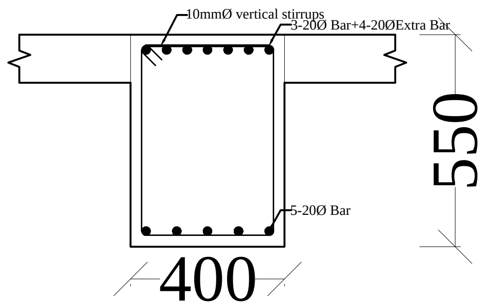
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



SECTION AT A-A

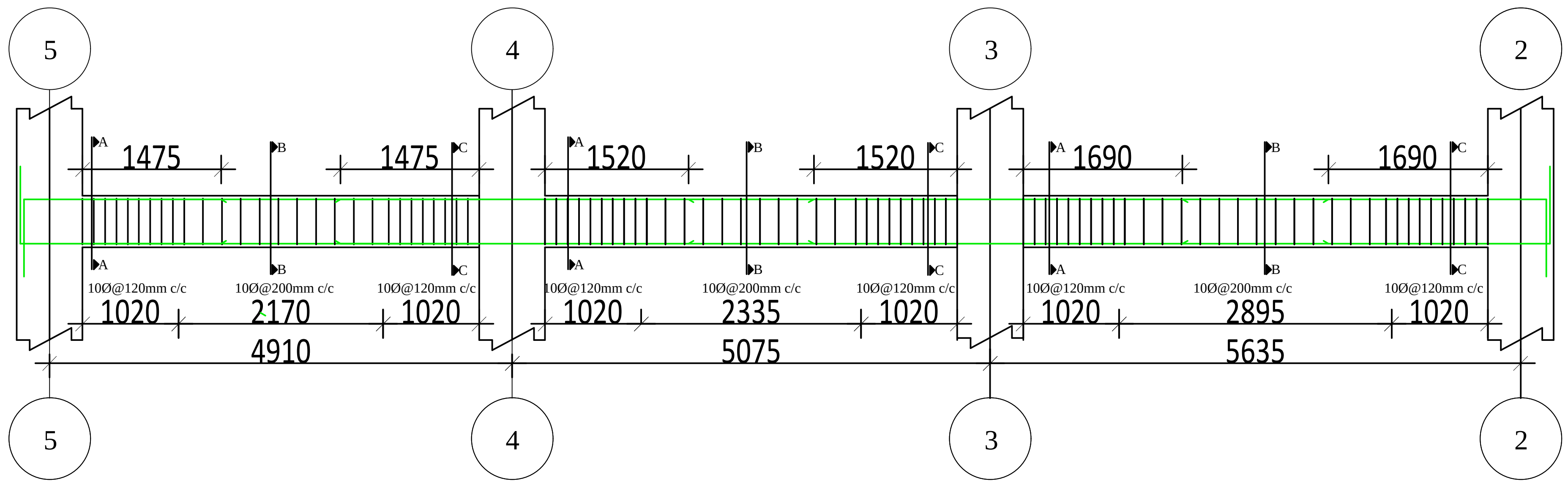


SECTION AT B-B



SECTION AT C-C

Beam cross-section along grid A-A



Beam longitudinal section along grid A-A

TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS

PROJECT TITLE:
EARTHQUAKE RESISTANT DESIGN OF
MULTISTORIED RESIDENTIAL BUILDING

SHEET TITLE
BEAM DETAILING

GROUP MEMBERS:
AVINASH JHA- 075BCE031
BHUPENDRA DULAL- 075BCE034
BIBEK PANDEYA- 075BCE036
BIJAY NEPAL- 075BCE037
ALISH KHATIWADA- 075BCE012
DIPIKA PANDEY- 075BCE054

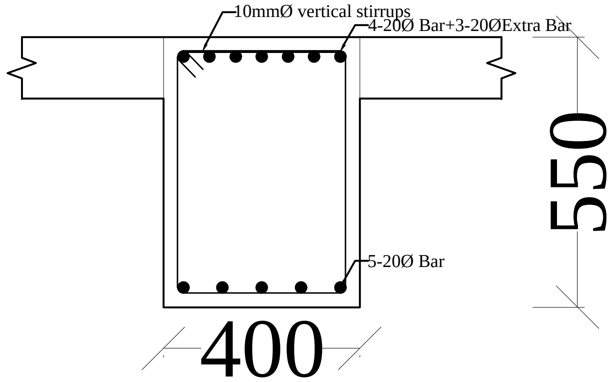
PROJECT SUPERVISOR :
ASST. PROF. SUBASH BASTOLA

Date : 2079 / 12 / 01

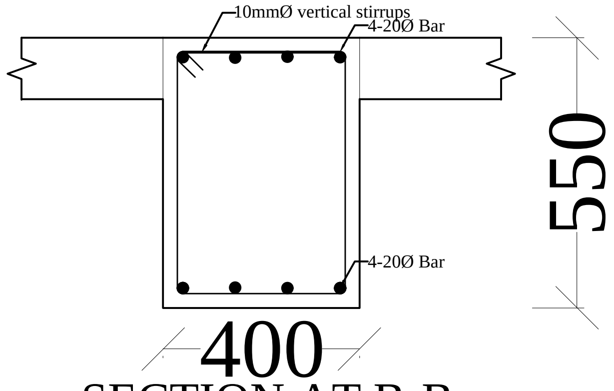
SCALE : NOT TO SCALE

DWG No. :

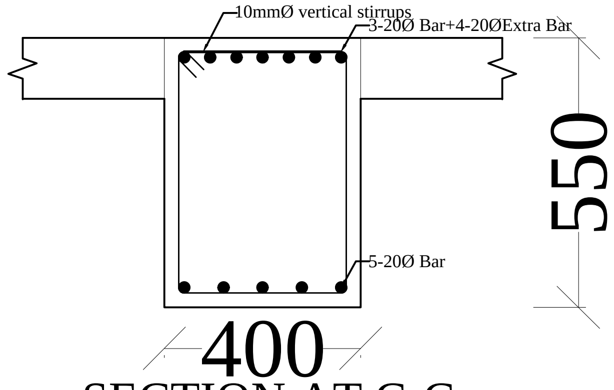
Sheet No. :



SECTION AT A-A



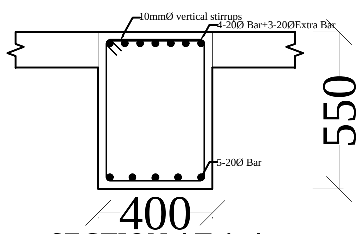
SECTION AT B-B



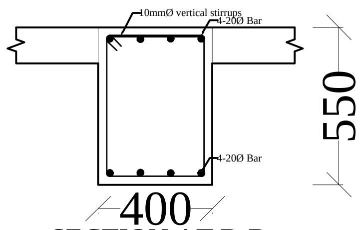
SECTION AT C-C

Beam cross-section along grid A-A 2F

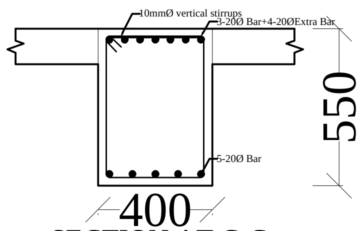
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



SECTION AT A-A

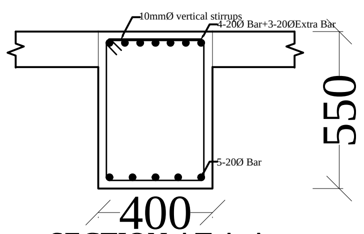


SECTION AT B-B

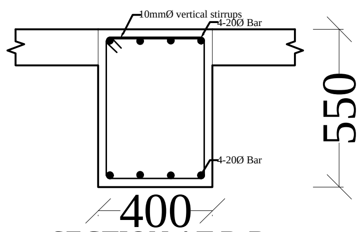


SECTION AT C-C

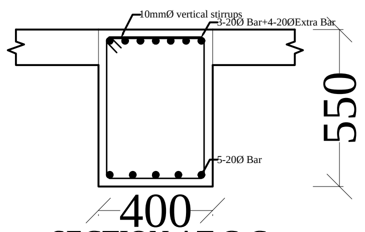
Beam cross-section along grid A-A 3F



SECTION AT A-A

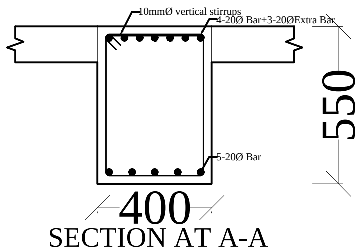


SECTION AT B-B

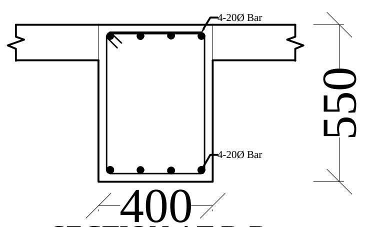


SECTION AT C-C

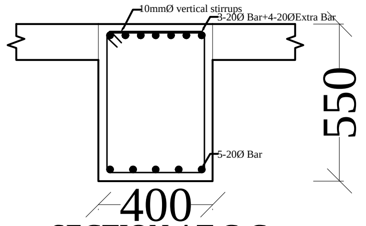
Beam cross-section along grid A-A 4F



SECTION AT A-A



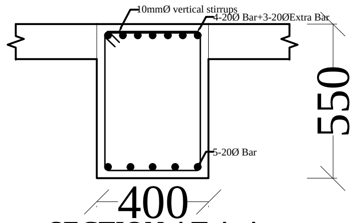
SECTION AT B-B



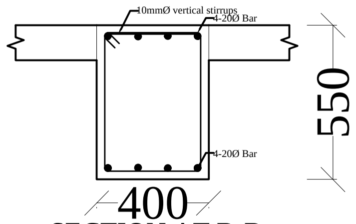
SECTION AT C-C

Beam cross-section along grid A-A 5F

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



SECTION AT A-A



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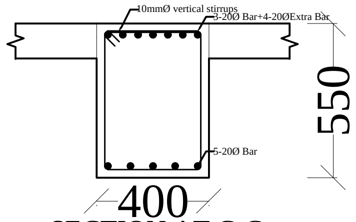
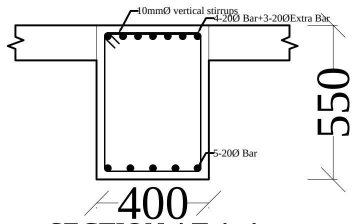
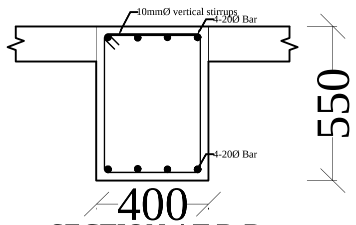


SECTION AT C-C

Beam cross-section along grid A-A 6F



SECTION AT A-A



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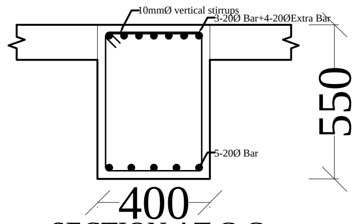
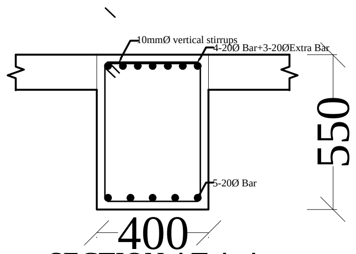


SECTION AT C-C

Beam cross-section along grid A-A 7F



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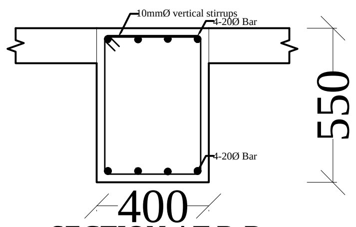

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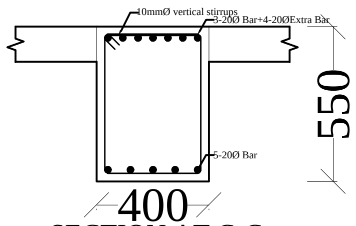

SECTION AT C-C

Beam cross-section along grid A-A 8F

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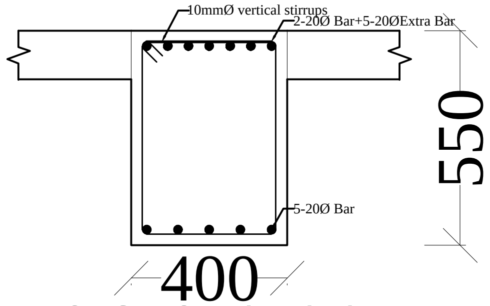
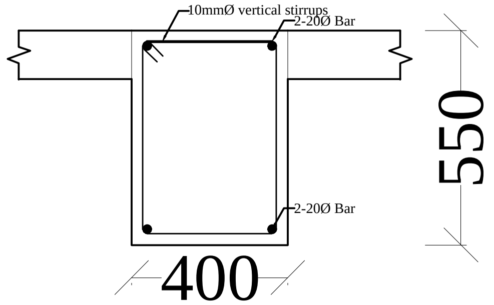


SECTION AT A-A



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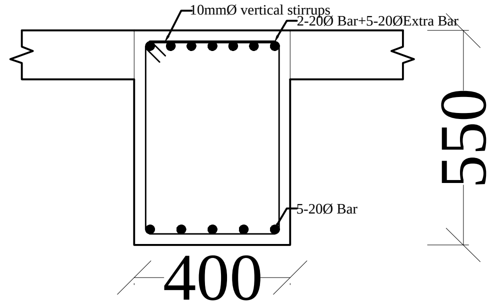
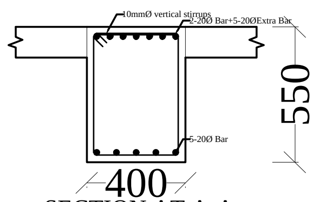


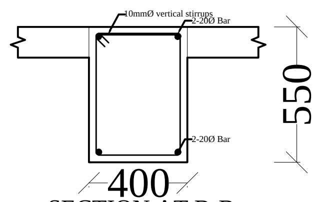
SECTION AT C-C

Beam cross-section along grid 1-1 2F

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



SECTION AT A-A



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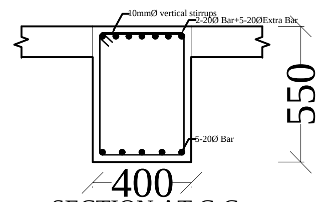
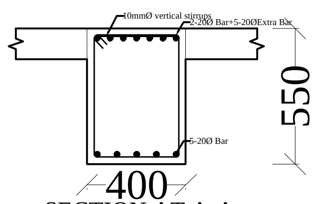
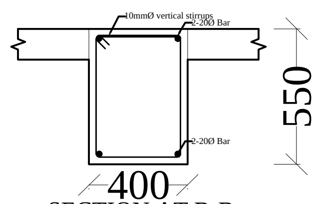


SECTION AT C-C

Beam cross-section along grid 1-1 3F



SECTION AT A-A



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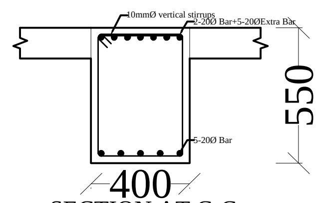
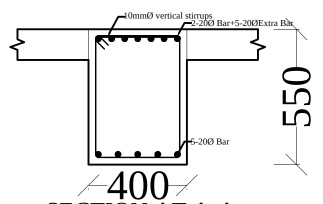


SECTION AT C-C

Beam cross-section along grid 1-1 4F



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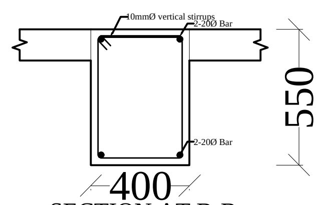

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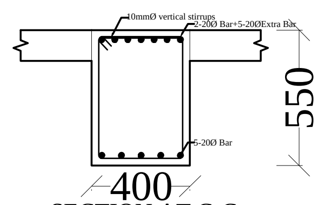

SECTION AT C-C

Beam cross-section along grid 1-1 5F

TRIBHUVAN UNIVERSITY
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SHEET TITLE
BEAM DETAILING

GROUP MEMBERS:
AVINASH JHA- 075BCE031
BHUPENDRA DULAL- 075BCE034
BIBEK PANDEYA- 075BCE036
BIJAY NEPAL- 075BCE037
ALISH KHATIWADA- 075BCE012
DIPIKA PANDEY- 075BCE054

PROJECT SUPERVISOR :
ASST. PROF. SUBASH BASTOLA

Date : 2079 / 12 / 01

SCALE : NOT TO SCALE

DWG No. :

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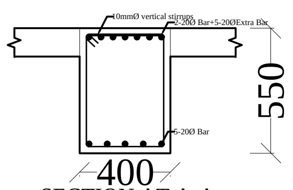

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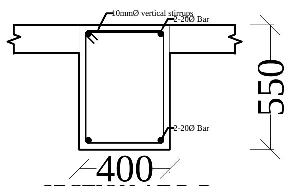

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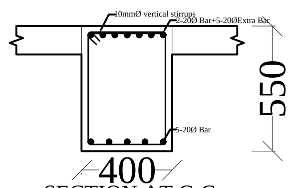
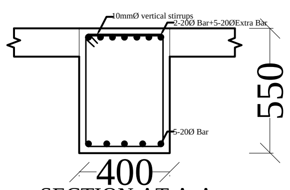
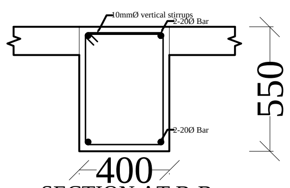


SECTION AT C-C

Beam cross-section along grid 1-1 Basement



SECTION AT A-A



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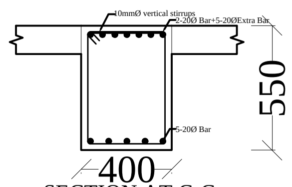
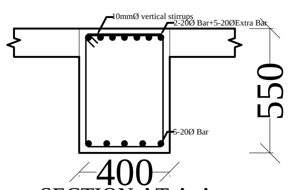
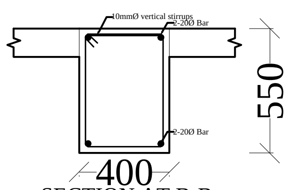


SECTION AT C-C

Beam cross-section along grid 1-1 GF



SECTION AT A-A



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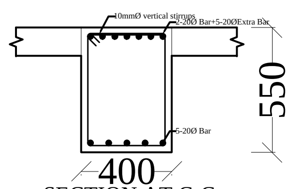

SECTION AT C-C

Beam cross-section along grid 1-1 1F

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SHEET TITLE
BEAM DETAILING

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AVINASH JHA- 075BCE031
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BIJAY NEPAL- 075BCE037
ALISH KHATIWADA- 075BCE012
DIPIKA PANDEY- 075BCE054

PROJECT SUPERVISOR :
ASST. PROF. SUBASH BASTOLA

Date : 2079 / 12 / 01

SCALE : NOT TO SCALE

DWG No. :

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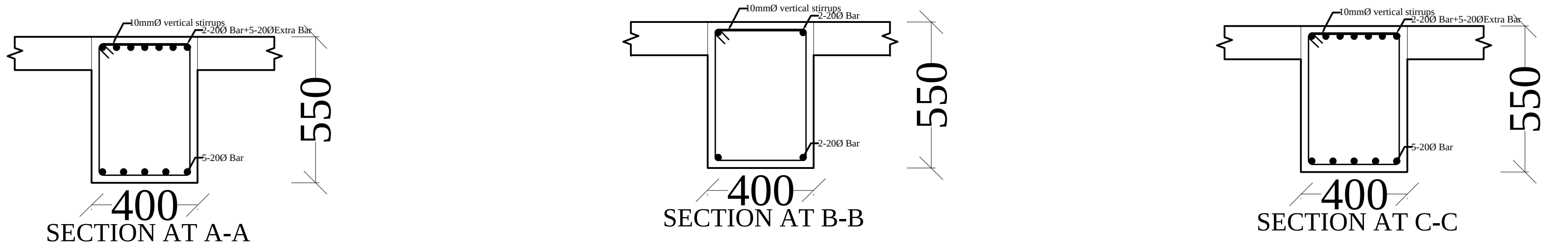
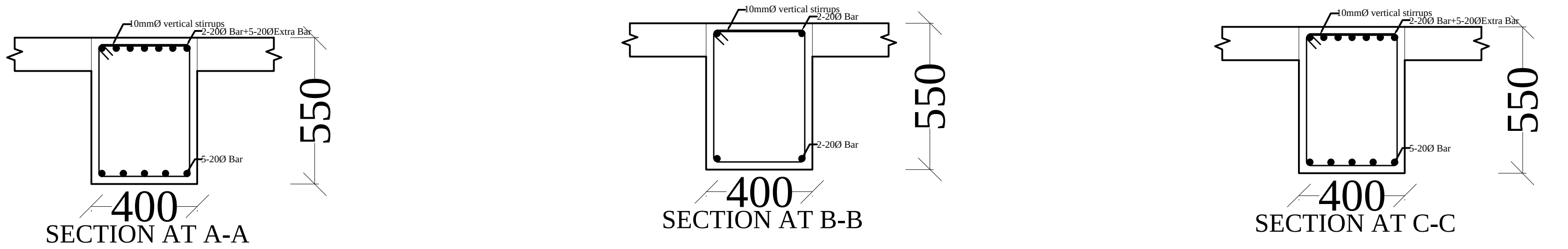
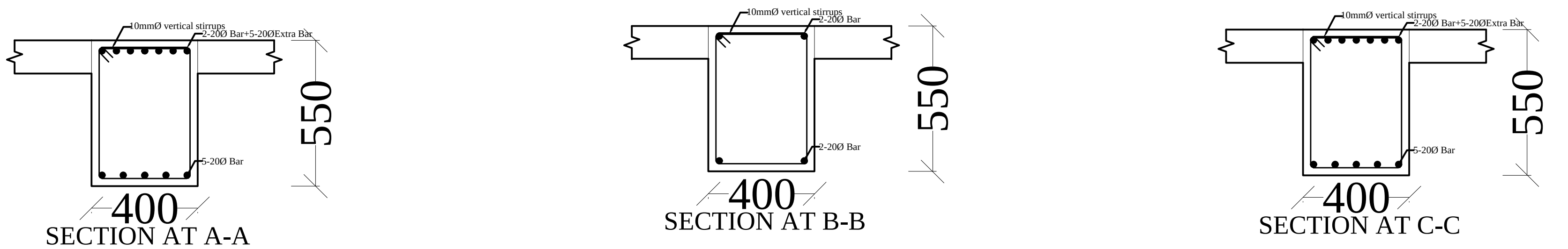


Beam cross-section along grid 1-1 6F

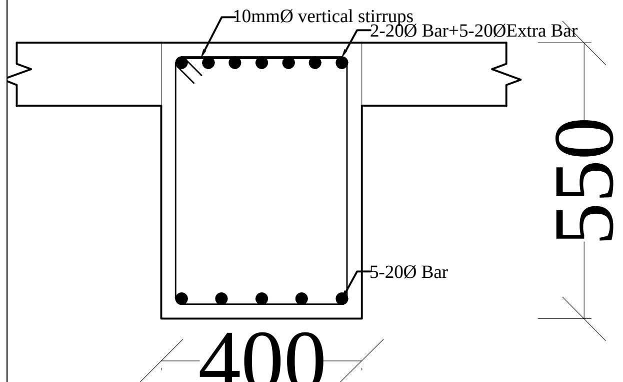


Beam cross-section along grid 1-1 7F

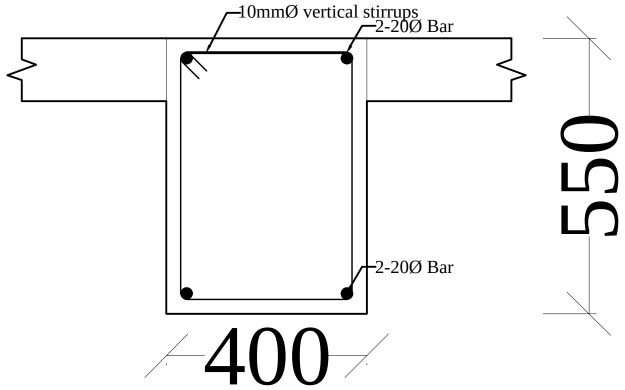


Beam cross-section along grid 1-1 8F

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



SECTION AT A-A



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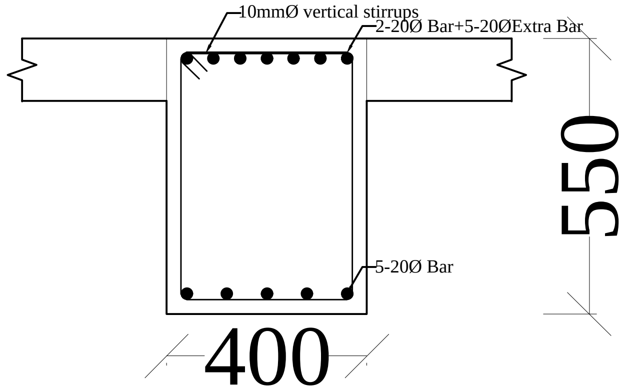
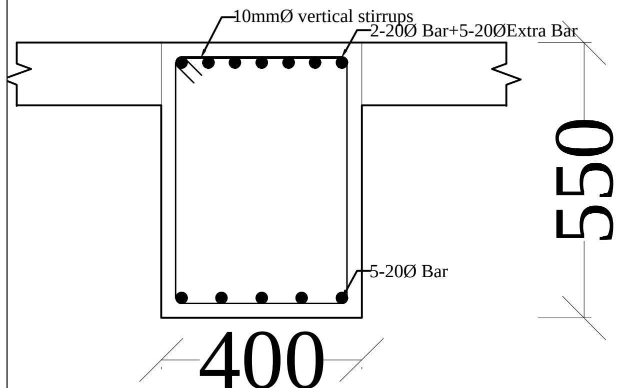


SECTION AT C-C

Beam cross-section along grid 1-1 2F

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



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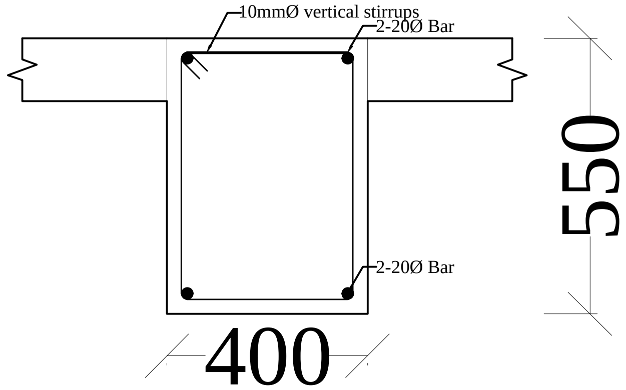

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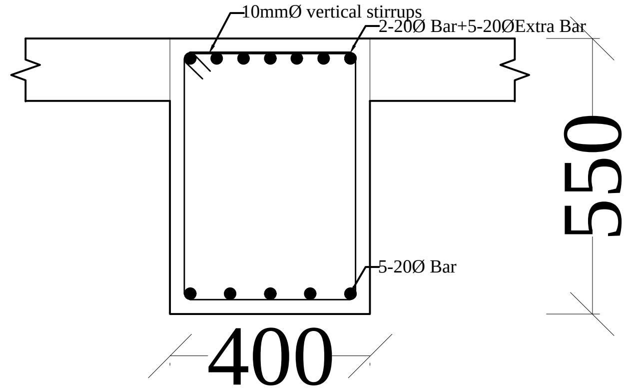

SECTION AT C-C

Beam cross-section along grid 1-1 2F

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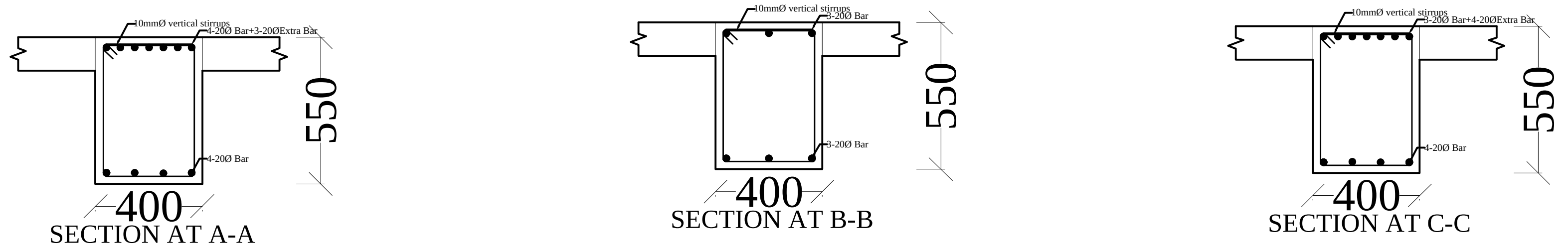
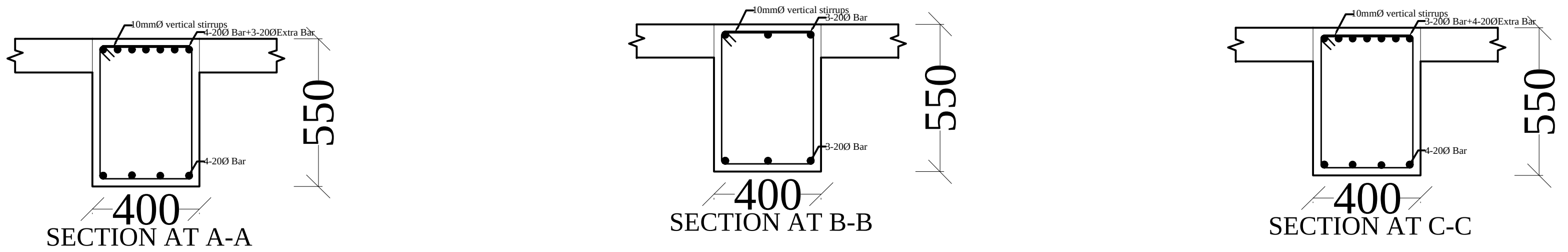
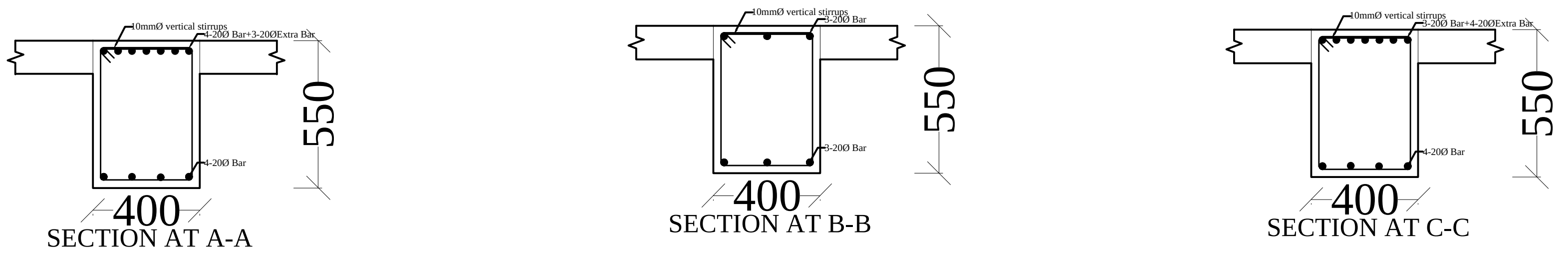


Beam cross-section along grid 2-2 Basement



Beam cross-section along grid 2-2 GF



Beam cross-section along grid 2-2 1F

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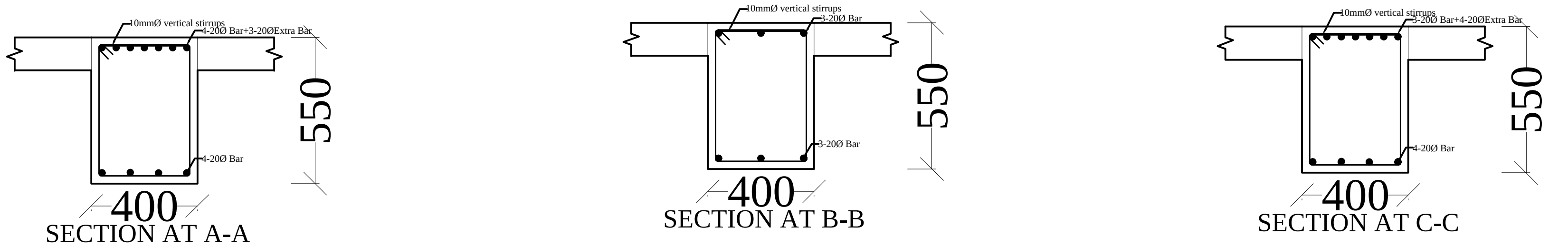
PROJECT SUPERVISOR :
ASST. PROF. SUBASH BASTOLA

Date : 2079 / 12 / 01

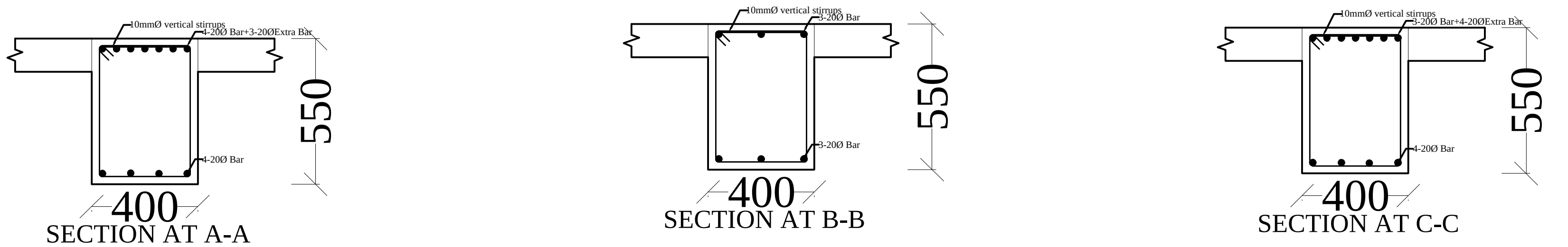
SCALE : NOT TO SCALE

DWG No. :

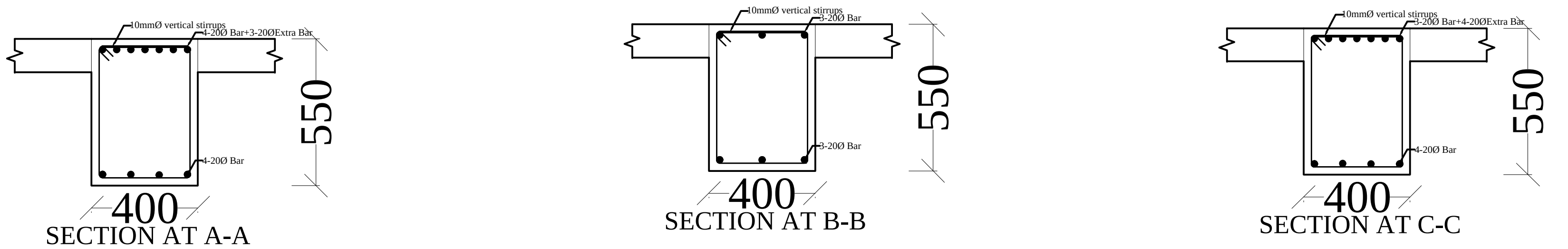
Sheet No. :



Beam cross-section along grid 2-2 3F

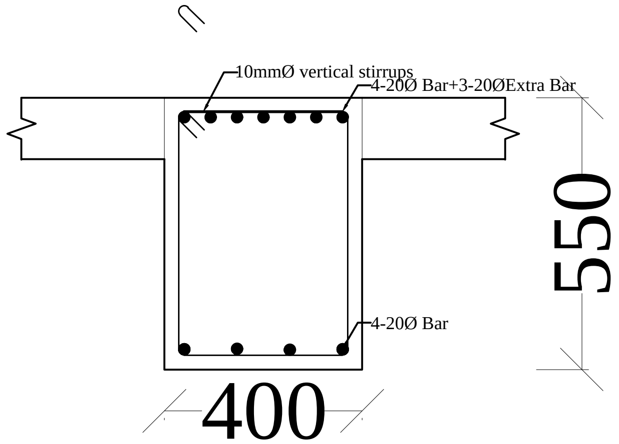


Beam cross-section along grid 2-2 4F

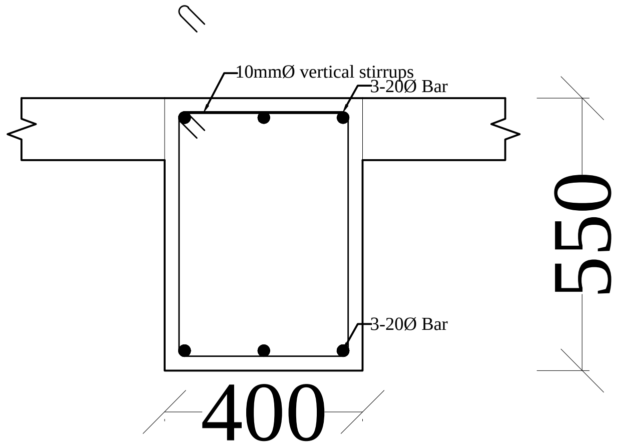


Beam cross-section along grid 2-2 5F

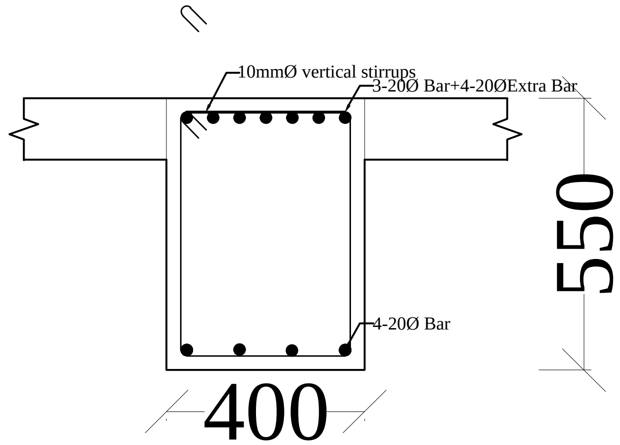
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



SECTION AT A-A



SECTION AT B-B



SECTION AT C-C

Beam cross-section along grid 2-2 2F

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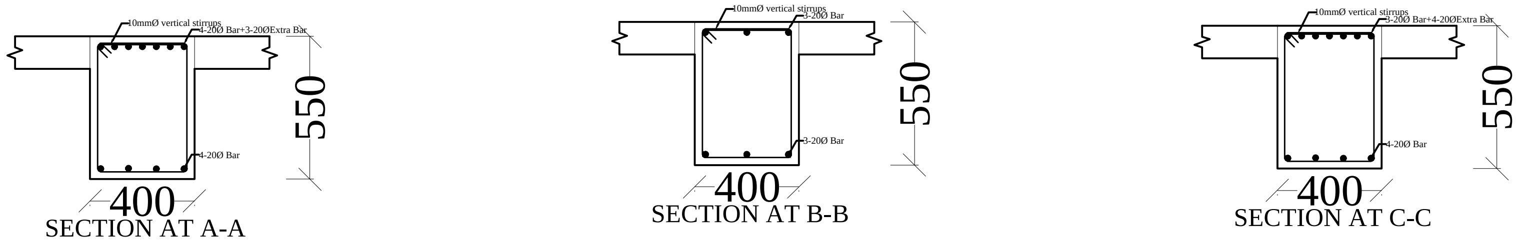
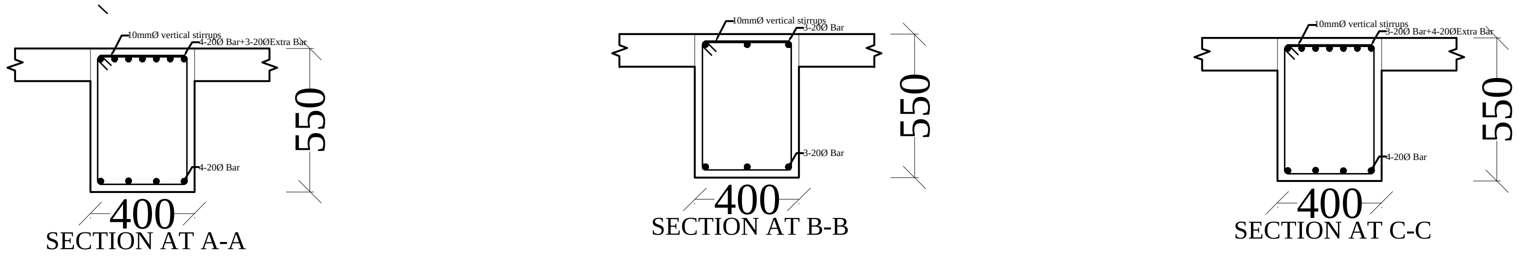
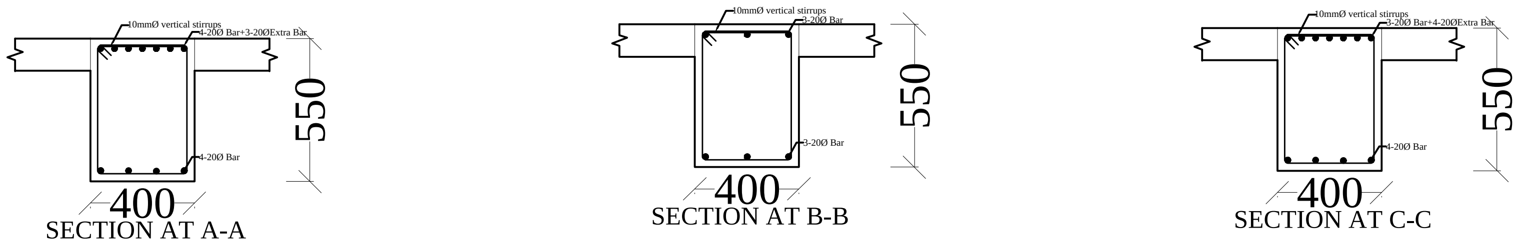


Beam cross-section along grid 2-2 6F

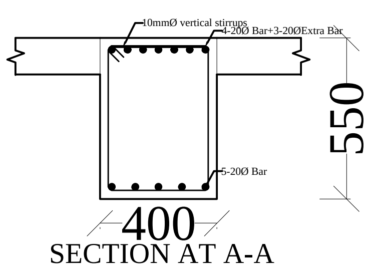


Beam cross-section along grid 2-2 7F

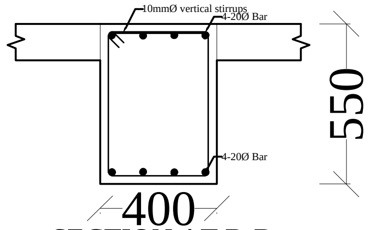


Beam cross-section along grid 2-2 8F

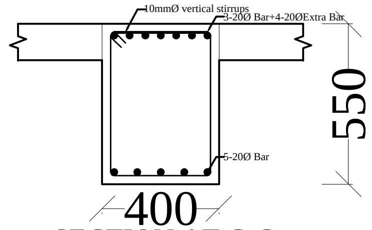
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



SECTION AT A-A

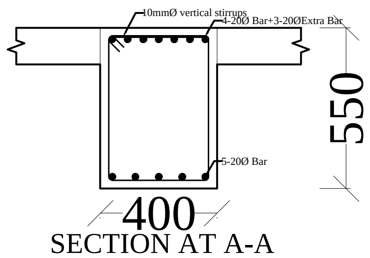


SECTION AT B-B

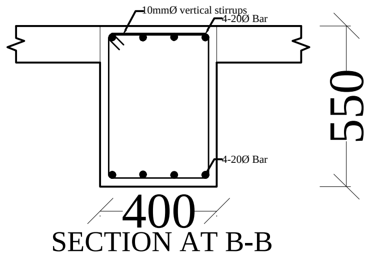


SECTION AT C-C

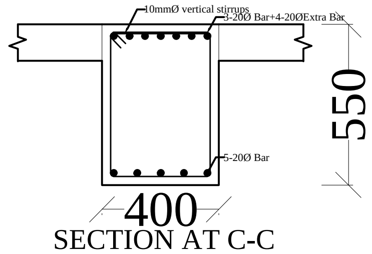
Beam cross-section along grid A-A Basement



SECTION AT A-A

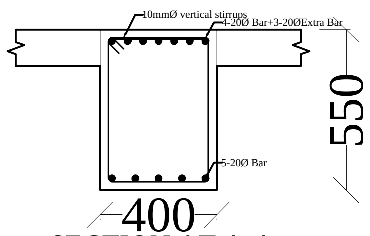


SECTION AT B-B

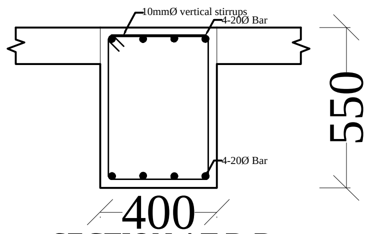


SECTION AT C-C

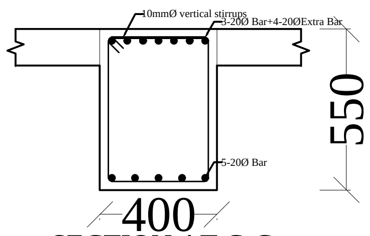
Beam cross-section along grid A-A GF



SECTION AT A-A



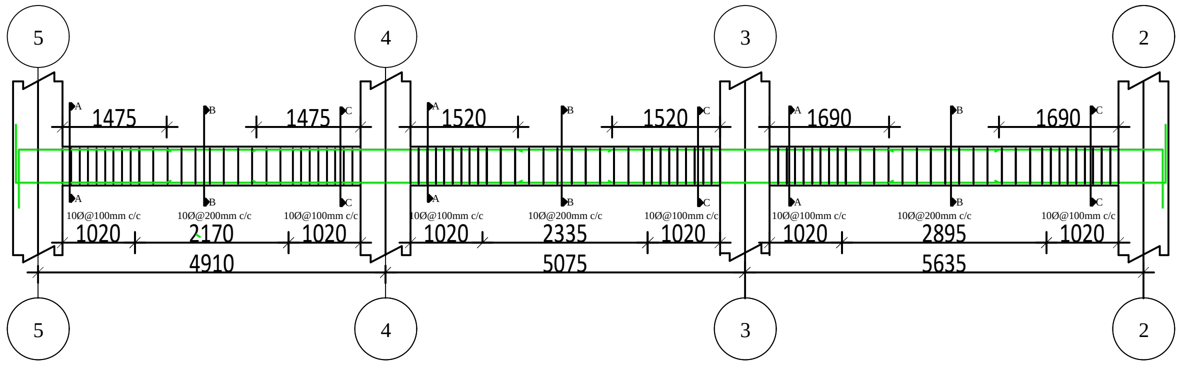
SECTION AT B-B



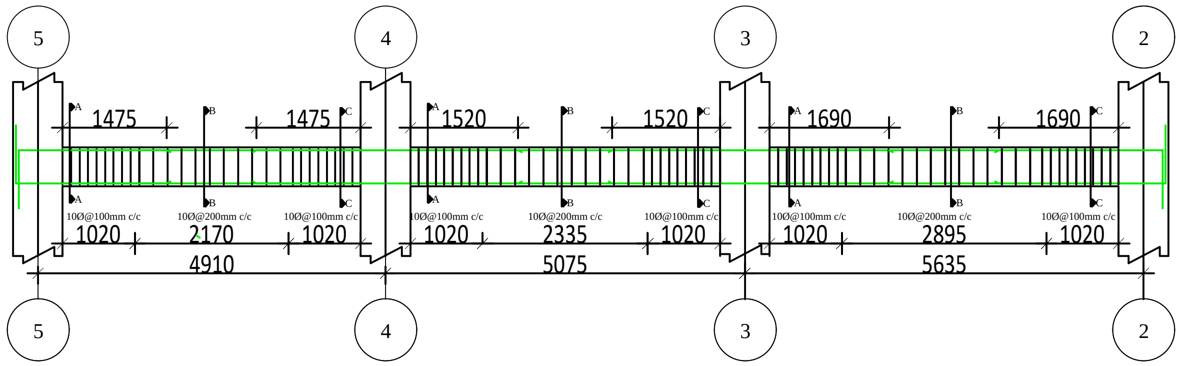
SECTION AT C-C

Beam cross-section along grid A-A 1F

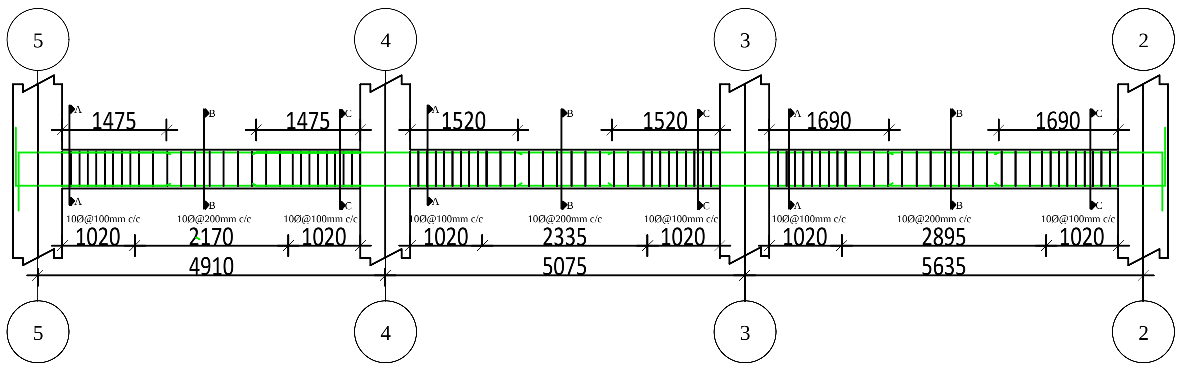
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



Beam longitudinal section along grid A-A 3F

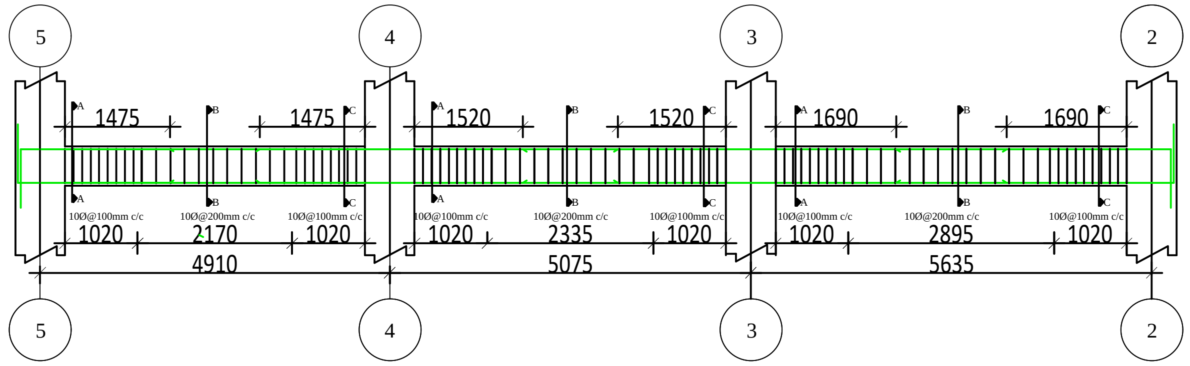


Beam longitudinal section along grid A-A 4F

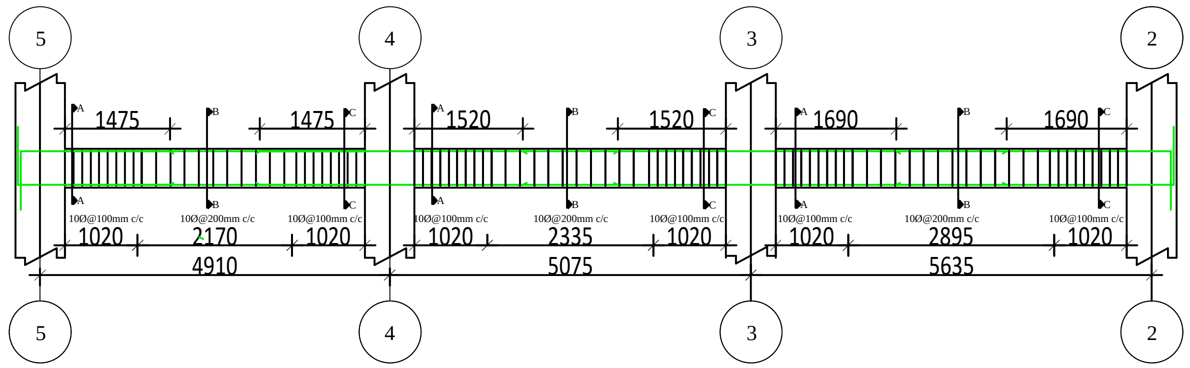


Beam longitudinal section along grid A-A 5F

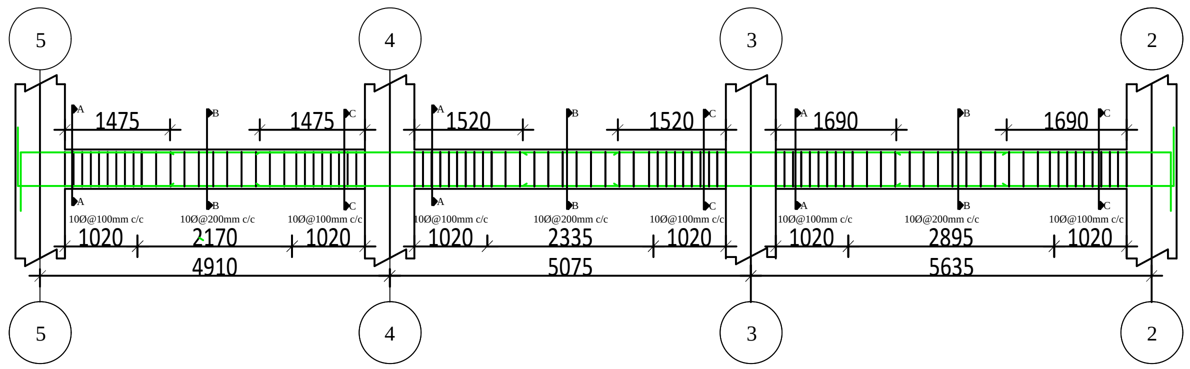
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



Beam longitudinal section along grid A-A 6F

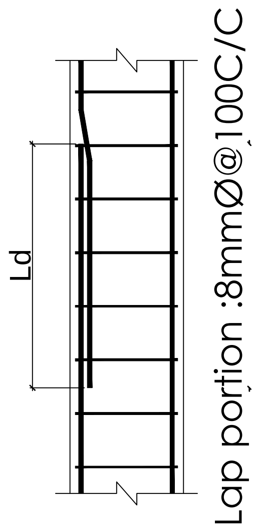


Beam longitudinal section along grid A-A 7F

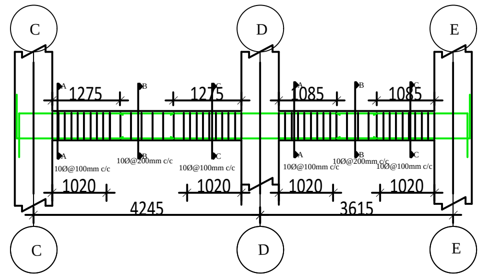


Beam longitudinal section along grid A-A 8F

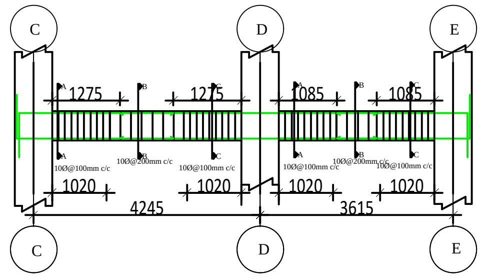
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



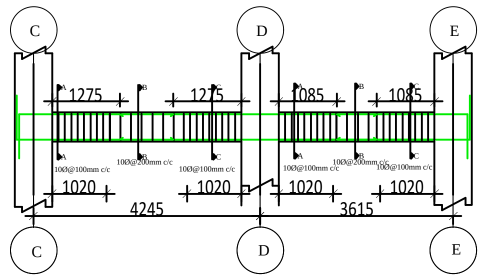
Lap Portion Detail



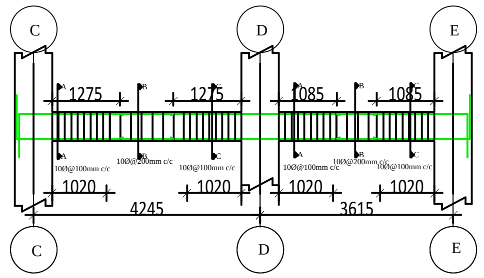
Beam longitudinal section along grid 1-1 Basement



Beam longitudinal section along grid 1-1 GF

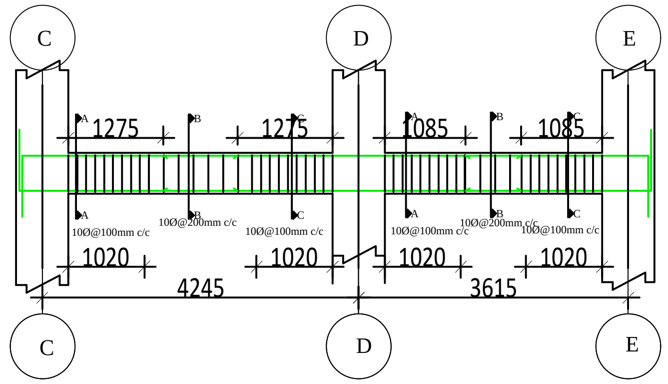


Beam longitudinal section along grid 1-1 1F

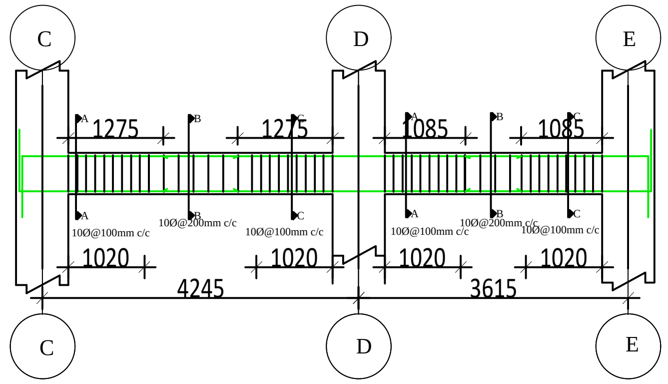


Beam longitudinal section along grid 1-1 2F

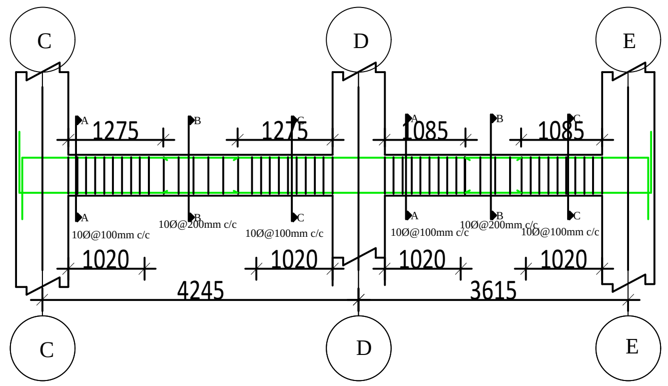
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



Beam longitudinal section along grid 1-1 3F

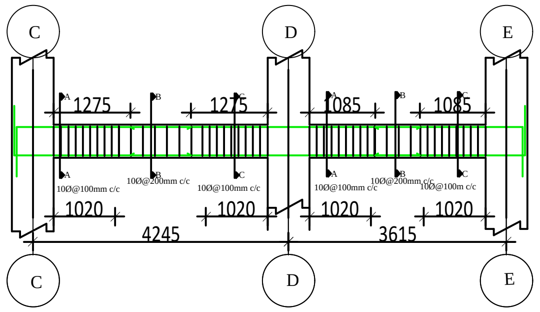


Beam longitudinal section along grid 1-1 4F

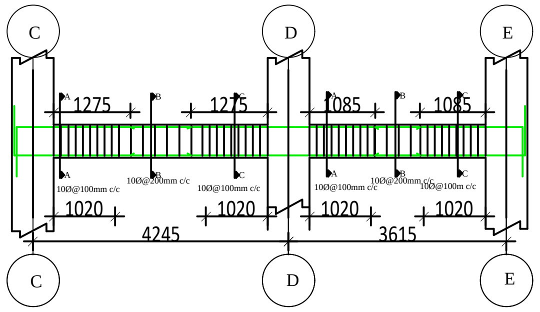


Beam longitudinal section along grid 1-1 5F

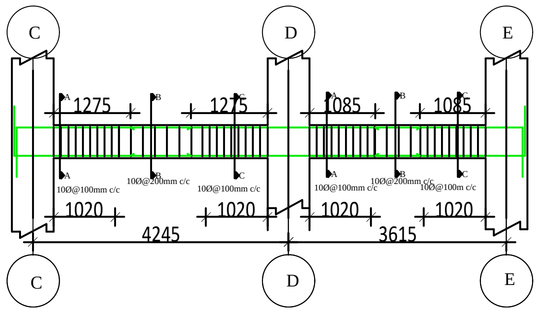
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



Beam longitudinal section along grid 1-1 6F

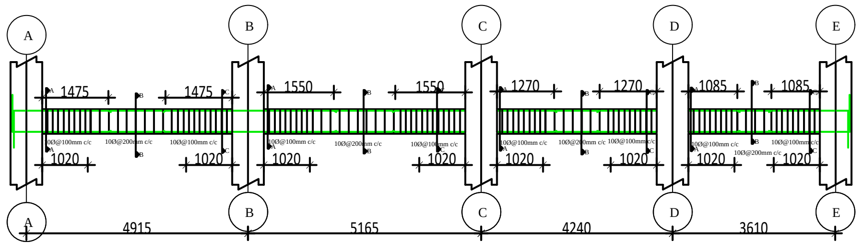


Beam longitudinal section along grid 1-1 7F

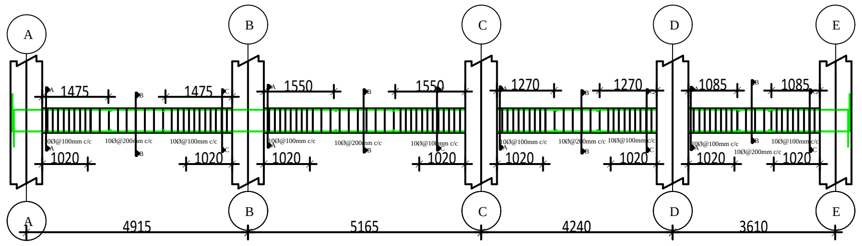


Beam longitudinal section along grid 1-1 8F

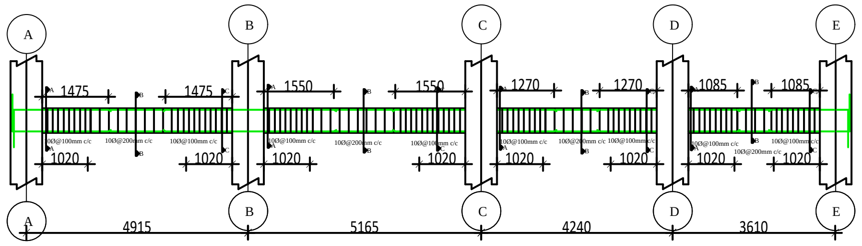
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



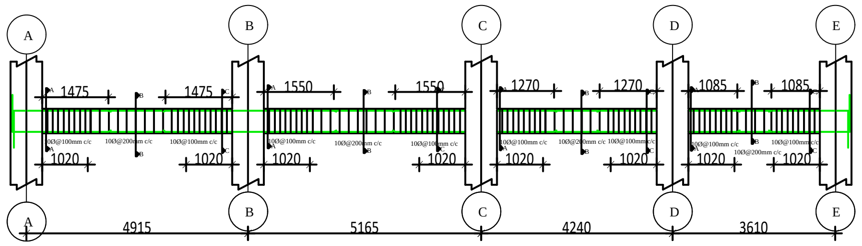
Beam longitudinal section along grid 2-2 Basement



Beam longitudinal section along grid 2-2 GF

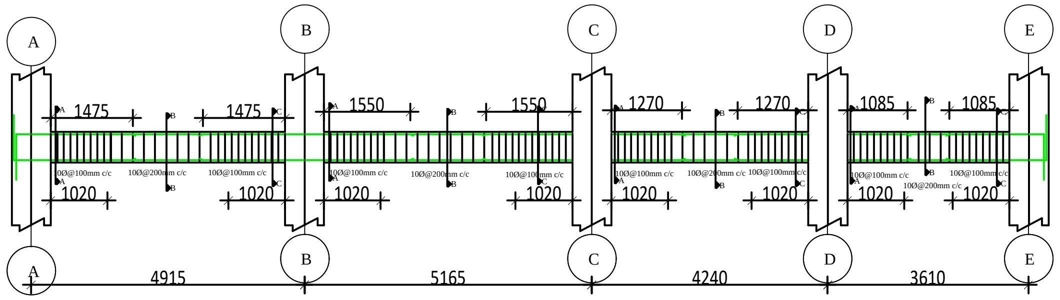


Beam longitudinal section along grid 2-2 1F

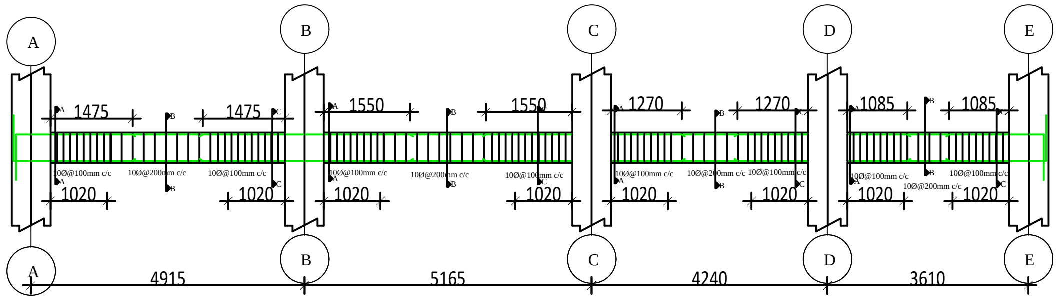


Beam longitudinal section along grid 2-2 2F

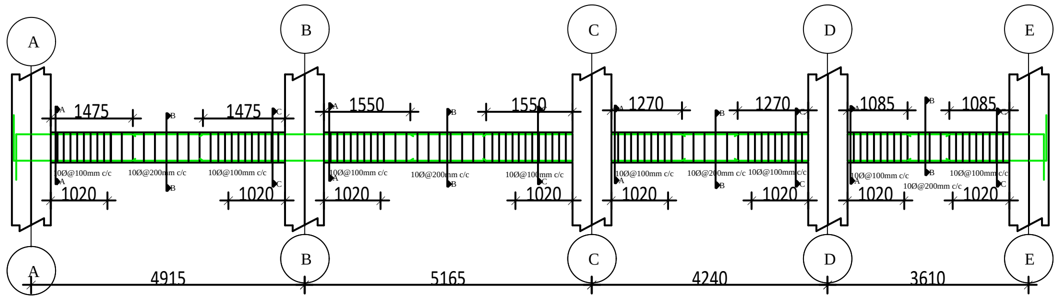
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



Beam longitudinal section along grid 2-2 3F

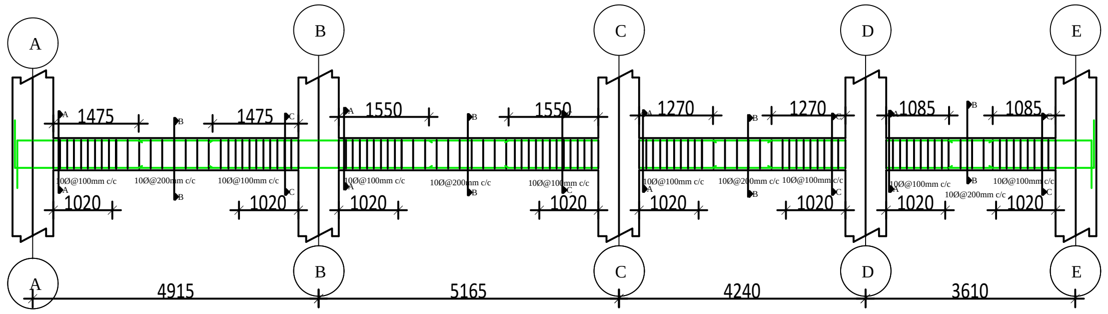


Beam longitudinal section along grid 2-2 4F

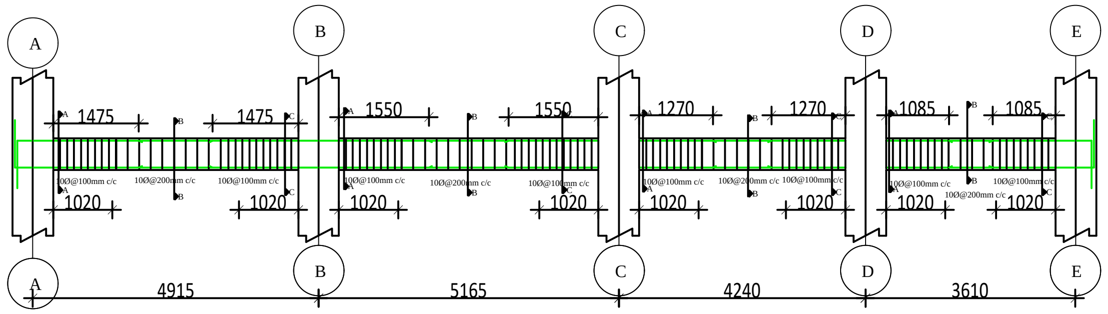


Beam longitudinal section along grid 2-2 5F

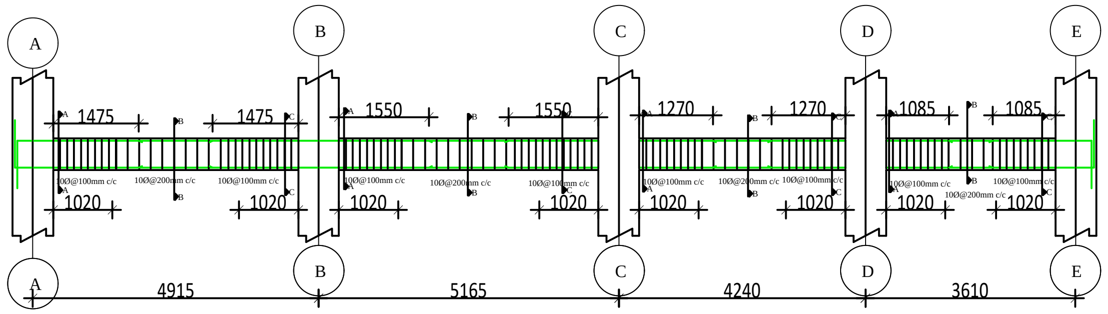
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



Beam longitudinal section along grid 2-2 6F

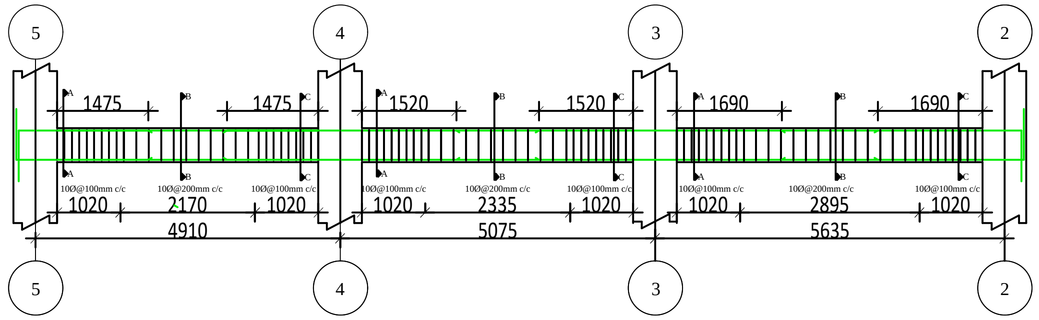


Beam longitudinal section along grid 2-2 7F

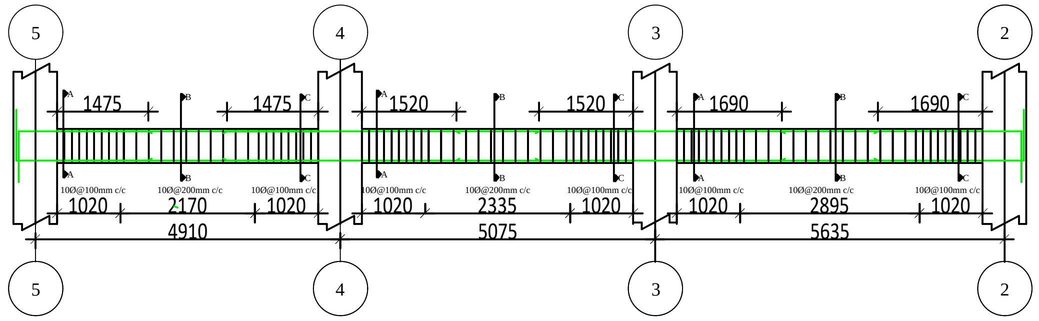


Beam longitudinal section along grid 2-2 8F

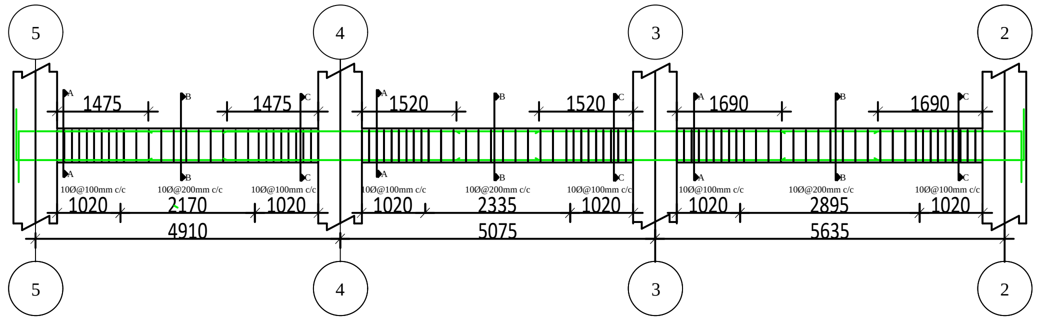
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



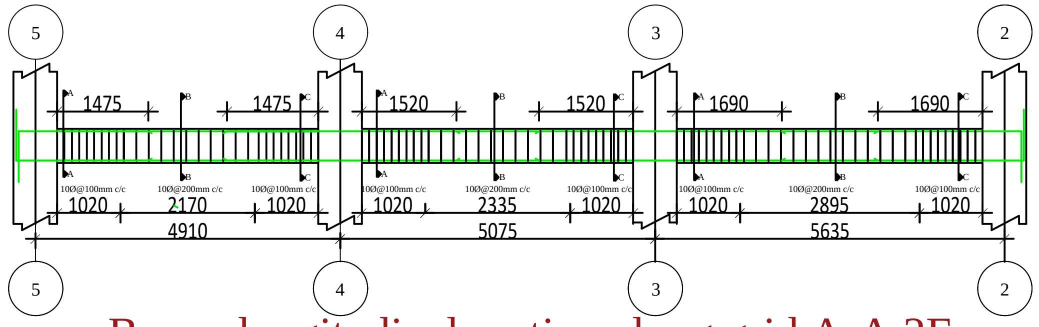
Beam longitudinal section along grid A-A GF



Beam longitudinal section along grid A-A GF

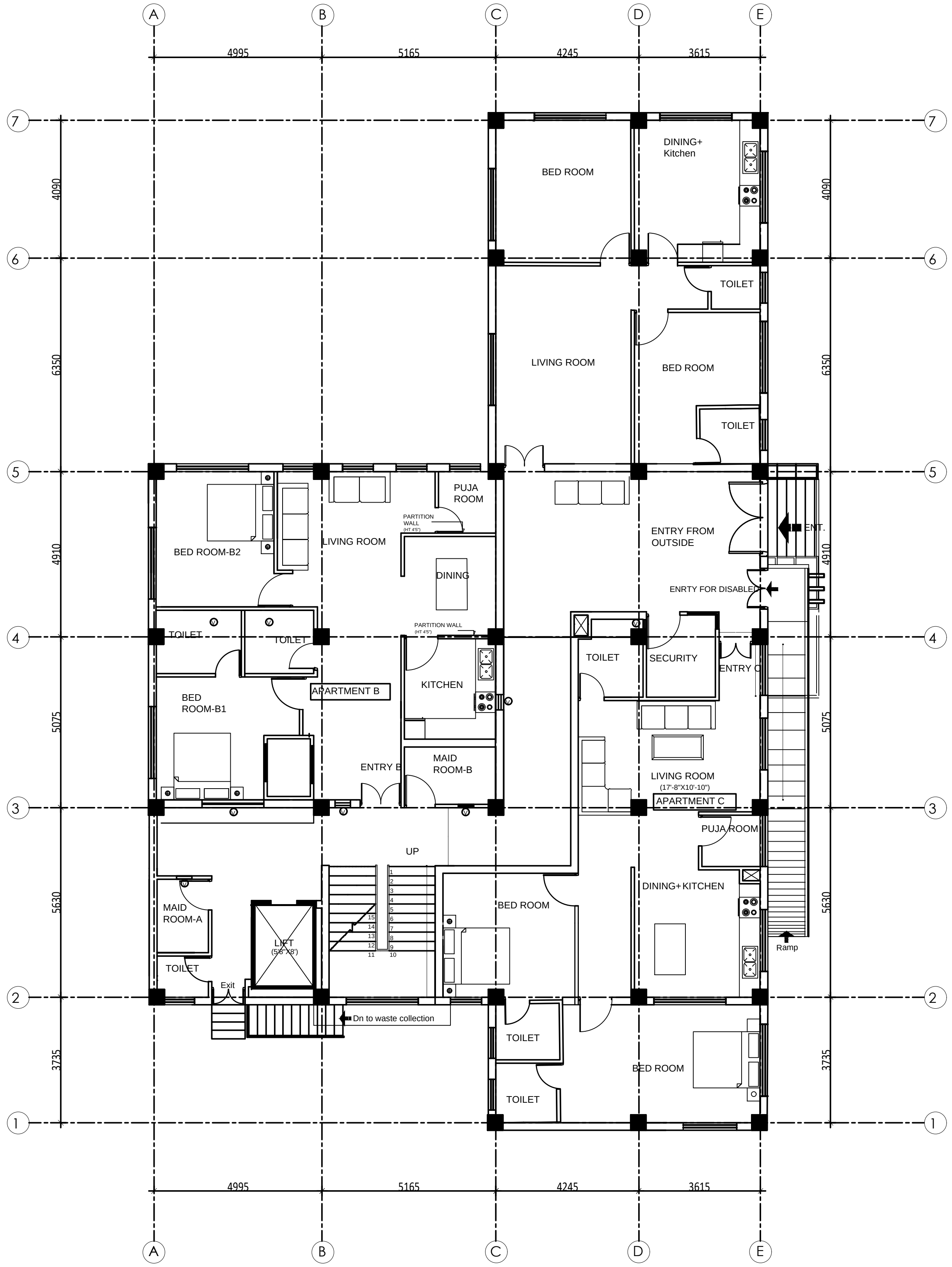


Beam longitudinal section along grid A-A 1F



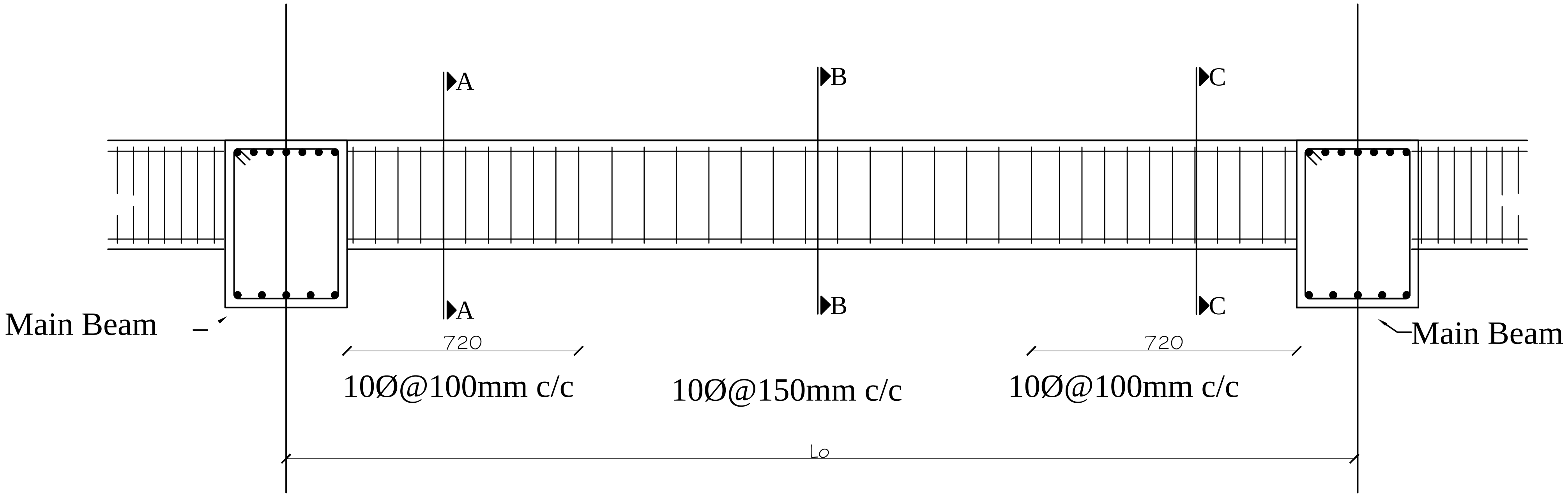
Beam longitudinal section along grid A-A 2F

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :

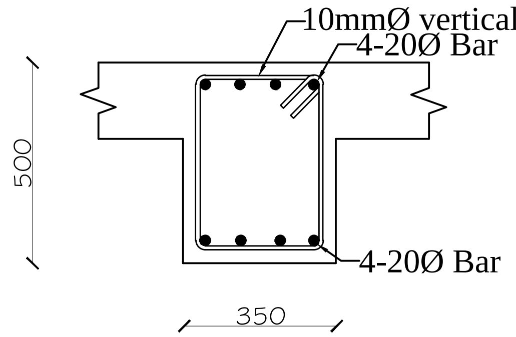


Ground Floor

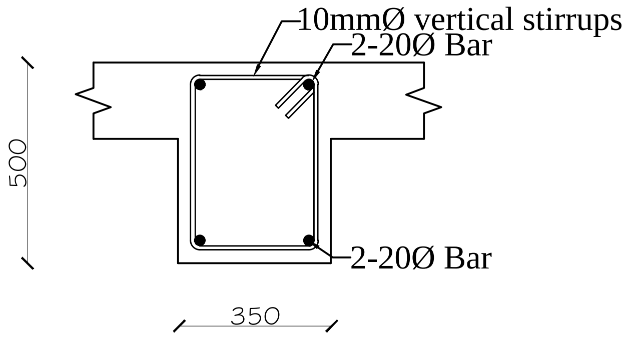
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE PLAN	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



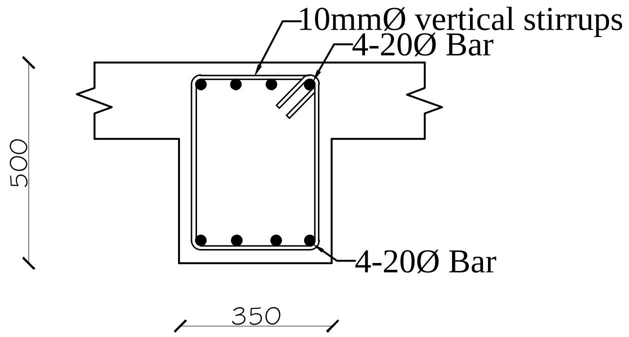
Secondary Beam Longitudinal Section



SECTION AT A-A

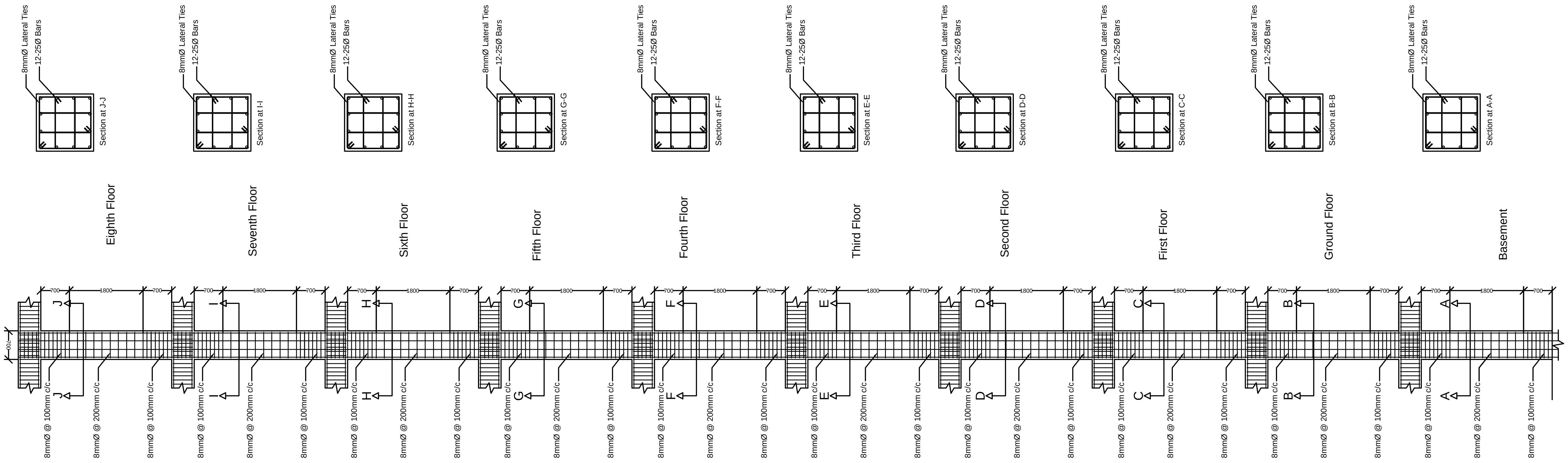


SECTION AT B-B



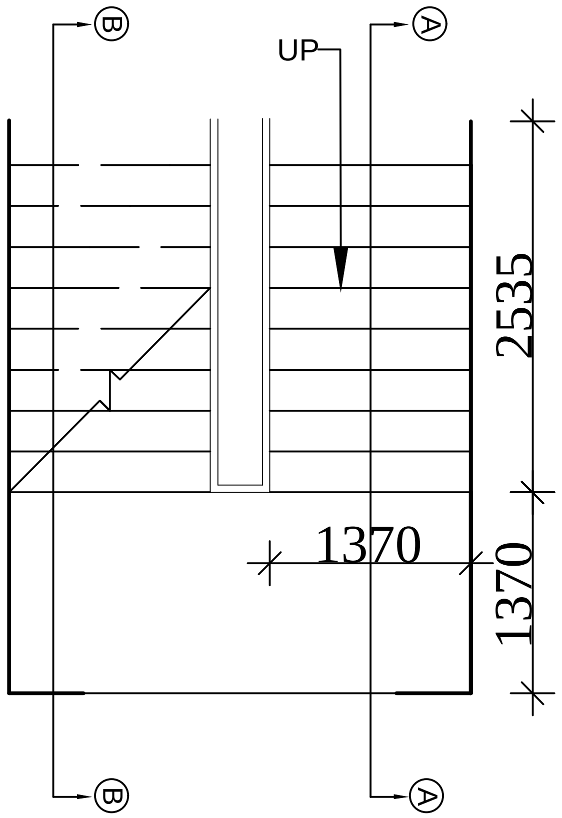
SECTION AT C-C

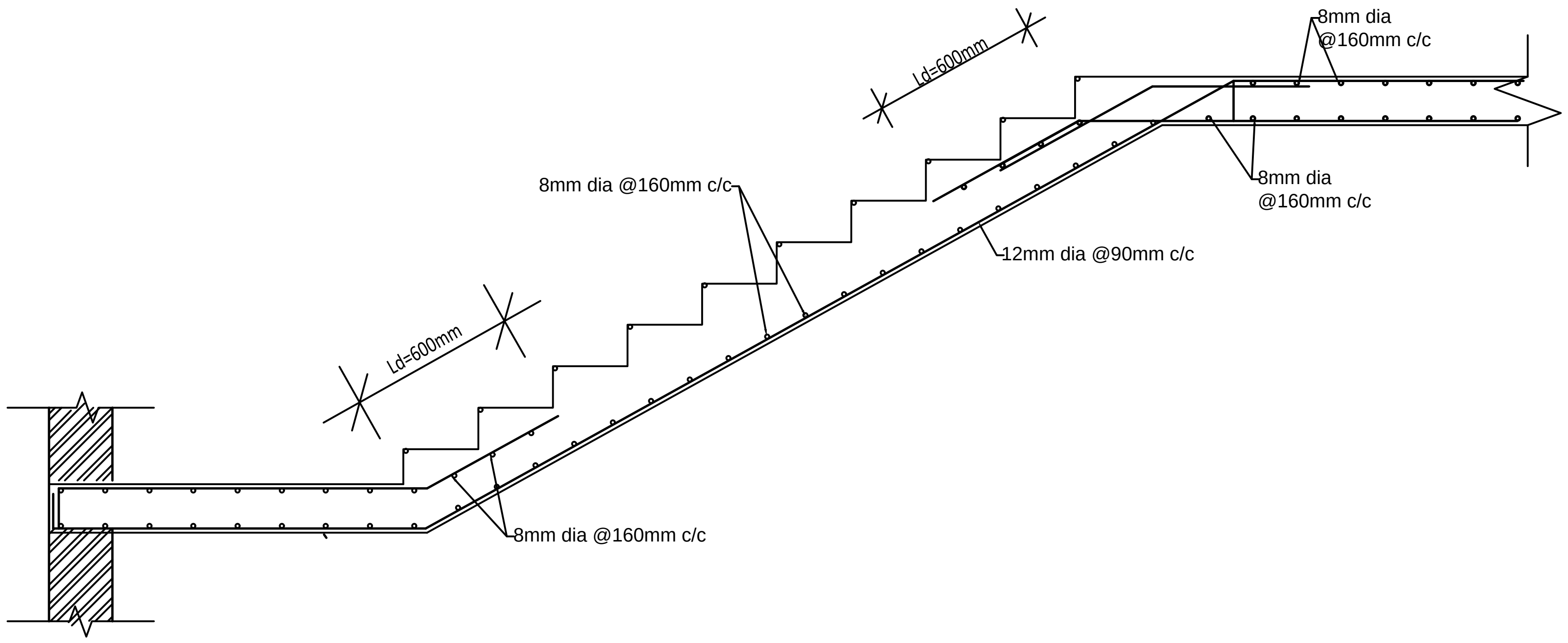
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE BEAM DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



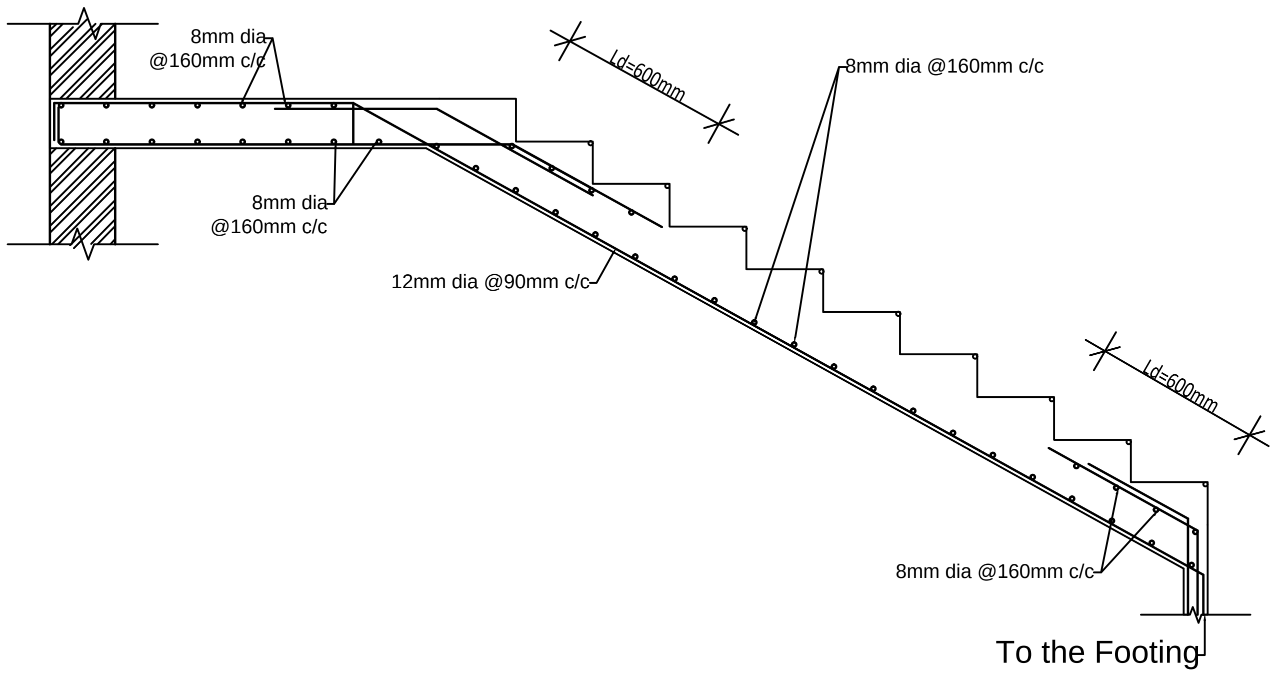
REINFORCEMENT DETAILING OF COLUMN

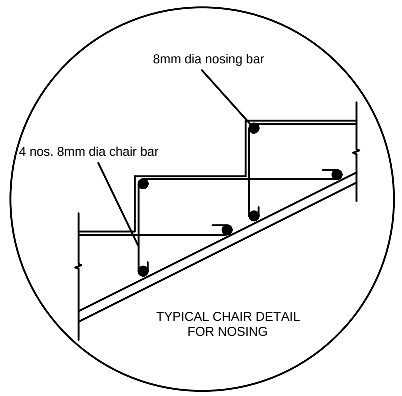
TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE COLUMN DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :





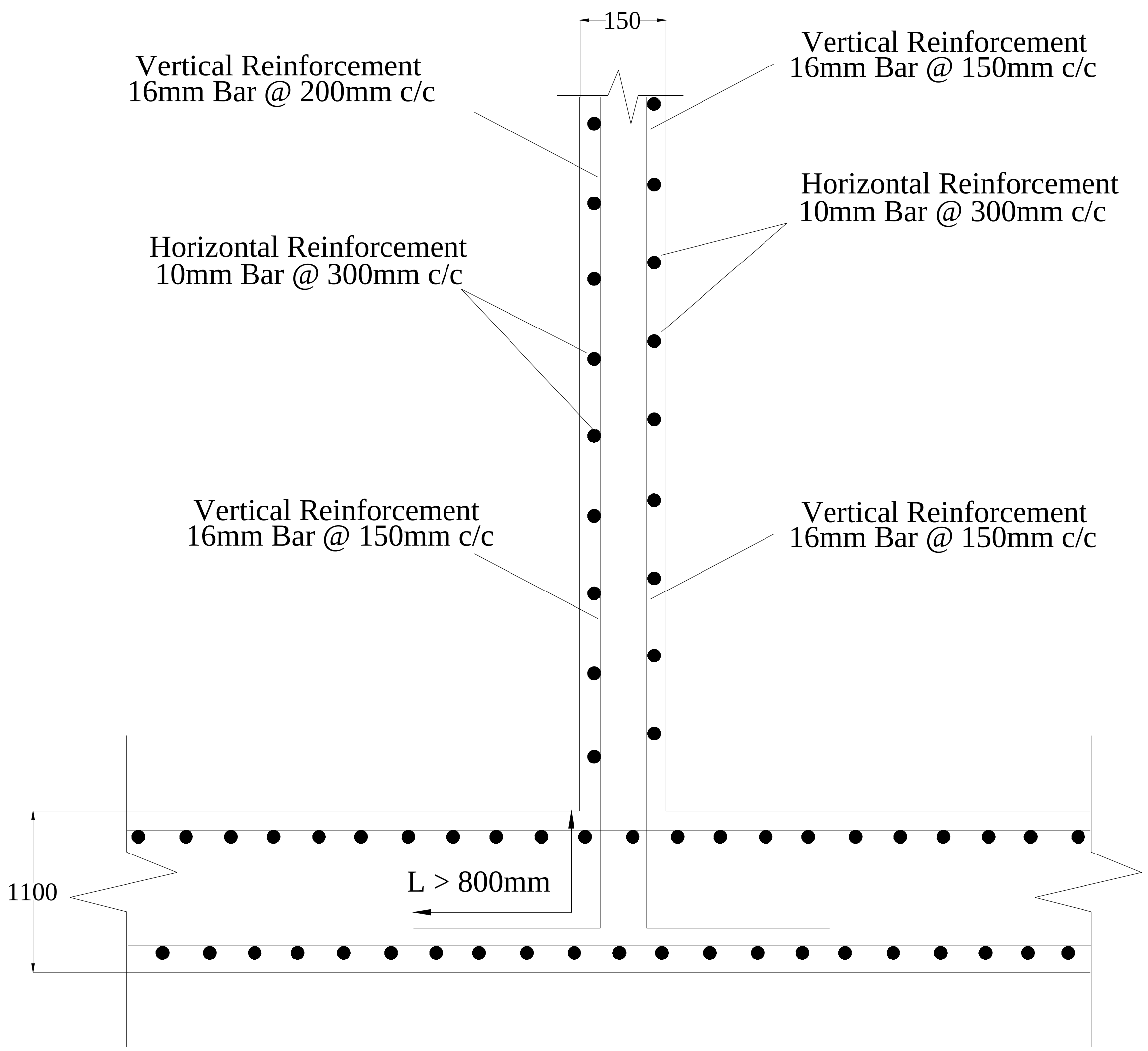
SECTION ALONG B-B AXIS



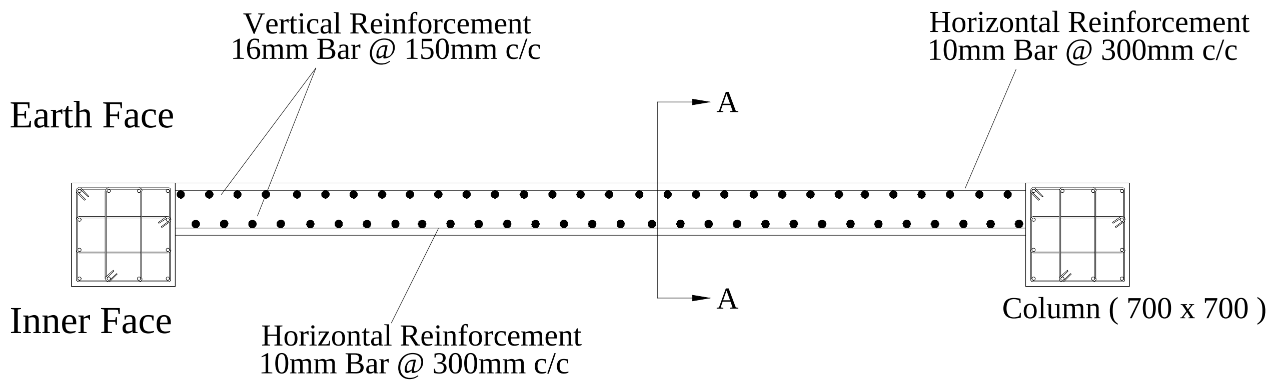


SECTION ALONG A-A AXIS

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE STAIRCASE DETAILING	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



Vertical Section at A-A



Basement Retaining Wall Detailing in Plan

TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS

PROJECT TITLE:
EARTHQUAKE RESISTANT DESIGN OF
MULTISTORIED RESIDENTIAL BUILDING

SHEET TITLE
BASEMENT WALL DETAILING

GROUP MEMBERS:
AVINASH JHA- 075BCE031
BHUPENDRA DULAL- 075BCE034
BIBEK PANDEYA- 075BCE036
BIJAY NEPAL- 075BCE037
ALISH KHATIWADA- 075BCE012
DIPIKA PANDEY- 075BCE054

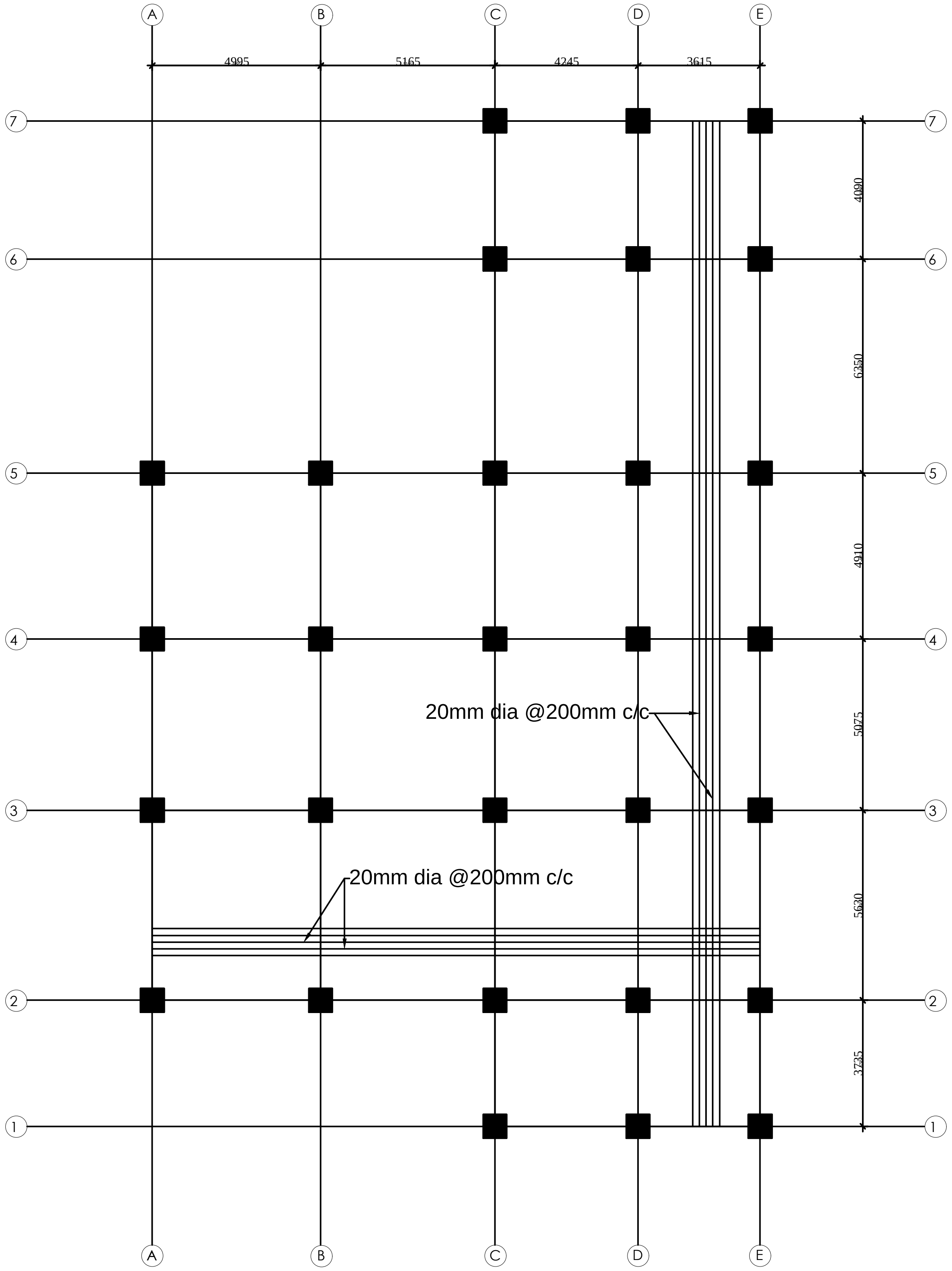
PROJECT SUPERVISOR :
ASST. PROF. SUBASH BASTOLA

Date : 2079 / 12 / 01

SCALE : NOT TO SCALE

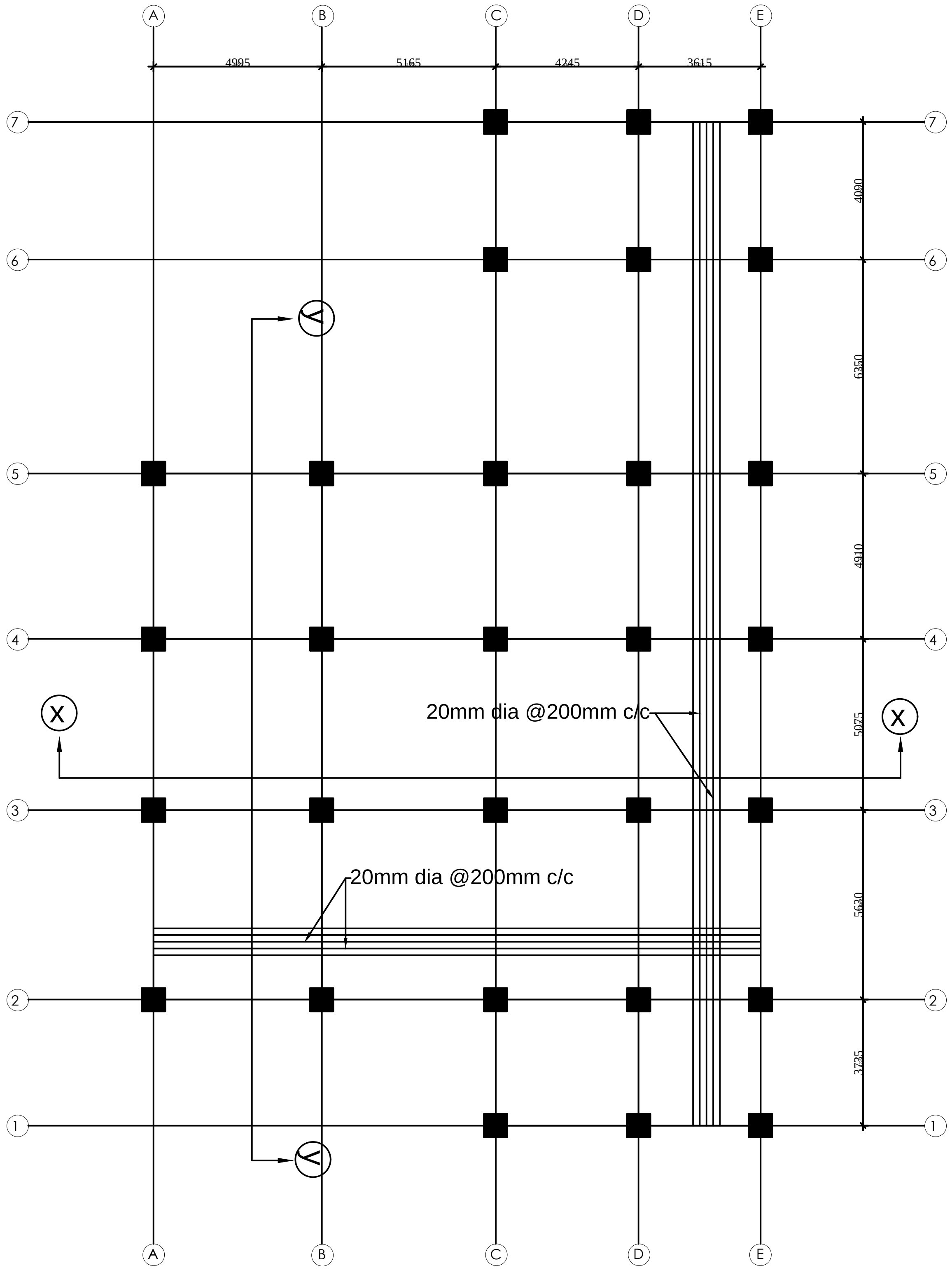
DWG No. :

Sheet No. :



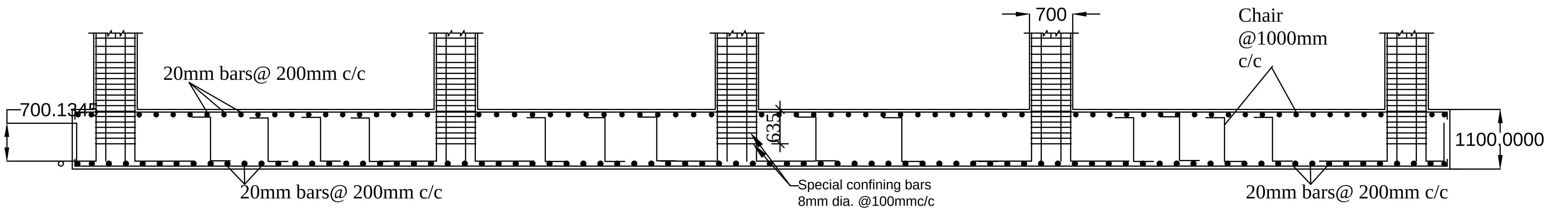
Top reinforcement of footing

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE FOUNDATION DETAILING(PLAN)	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :

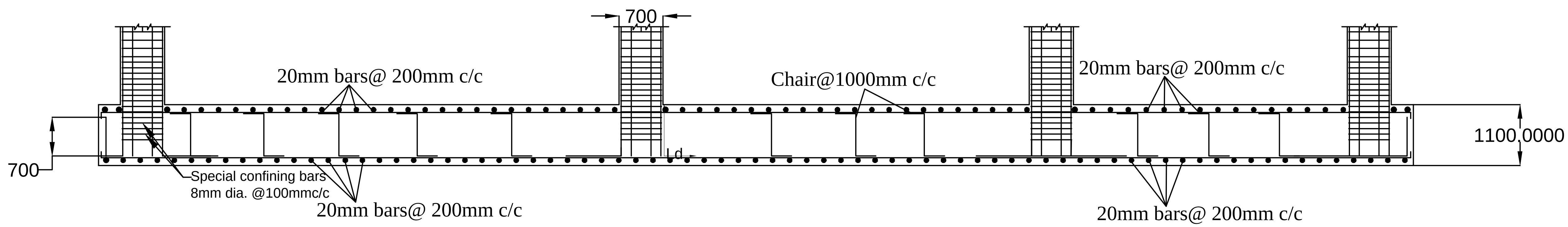


Bottom reinforcement of footing

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE FOUNDATION DETAILING(PLAN)	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :



SECTION ALONG X-X AXIS



SECTION ALONG Y-Y AXIS

TRIBHUVAN UNIVERSITY INSTITUTE OF ENGINEERING PULCHOWK CAMPUS	PROJECT TITLE: EARTHQUAKE RESISTANT DESIGN OF MULTISTORIED RESIDENTIAL BUILDING	SHEET TITLE FOUNDATION DETAILING(SECTION)	GROUP MEMBERS: AVINASH JHA- 075BCE031 BHUPENDRA DULAL- 075BCE034 BIBEK PANDEYA- 075BCE036 BIJAY NEPAL- 075BCE037 ALISH KHATIWADA- 075BCE012 DIPIKA PANDEY- 075BCE054	PROJECT SUPERVISOR : ASST. PROF. SUBASH BASTOLA Date : 2079 / 12 / 01	SCALE : NOT TO SCALE DWG No. : Sheet No. :

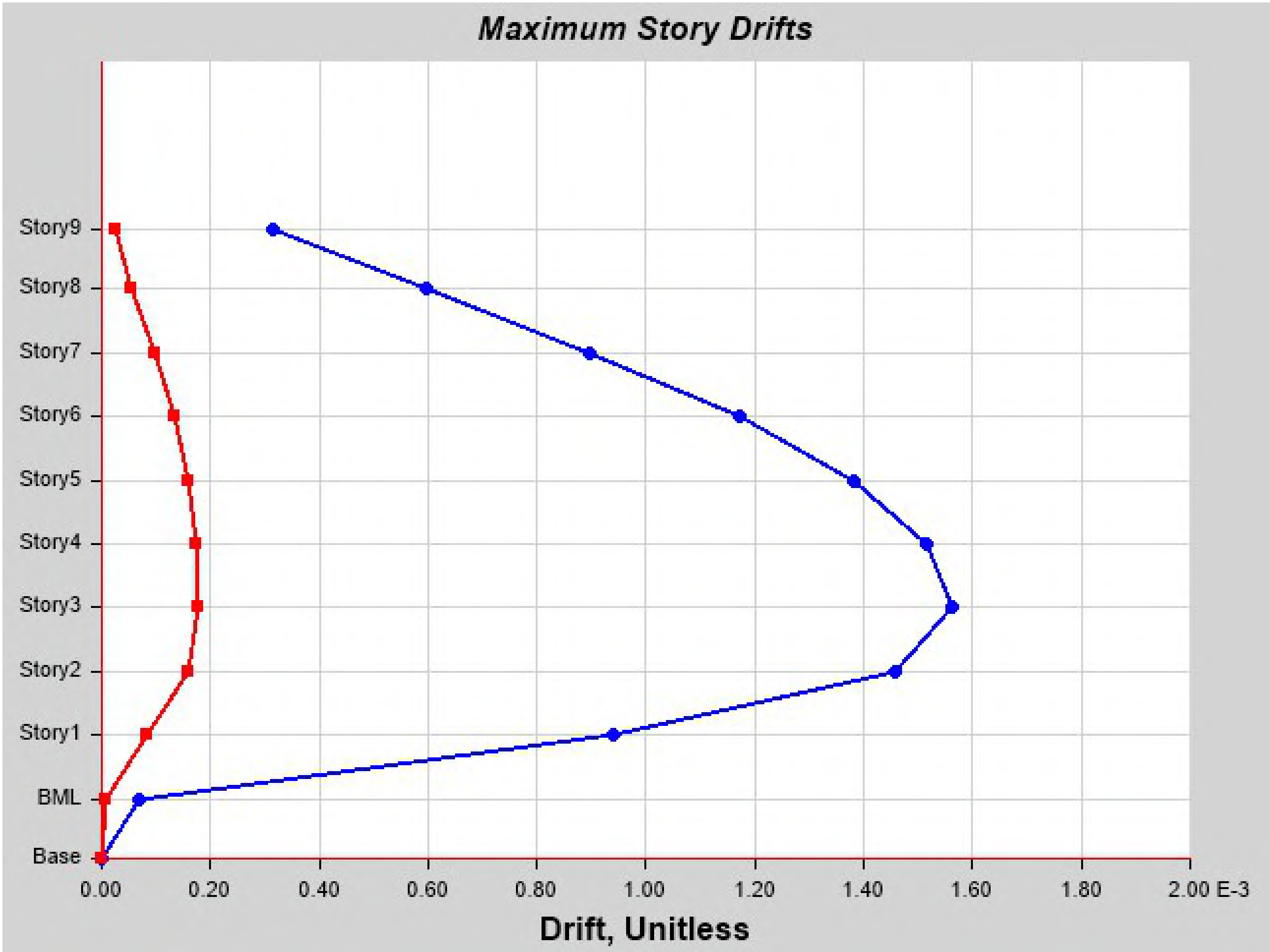


Fig.Maximum Story drft in EQx direction

Fig.Maximum Story drft in EQy direction

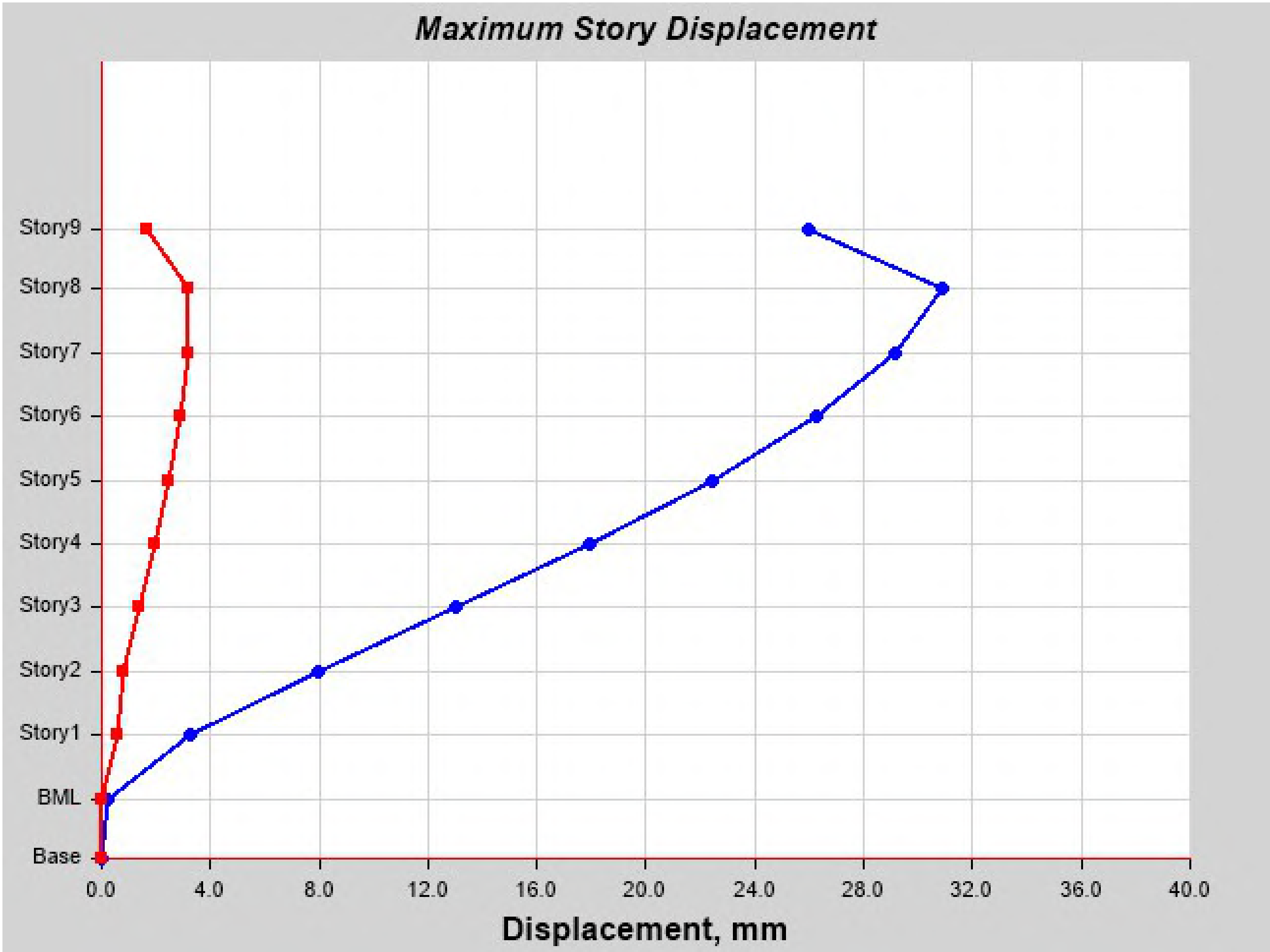


Fig.Maximum Story displacement in EQx direction

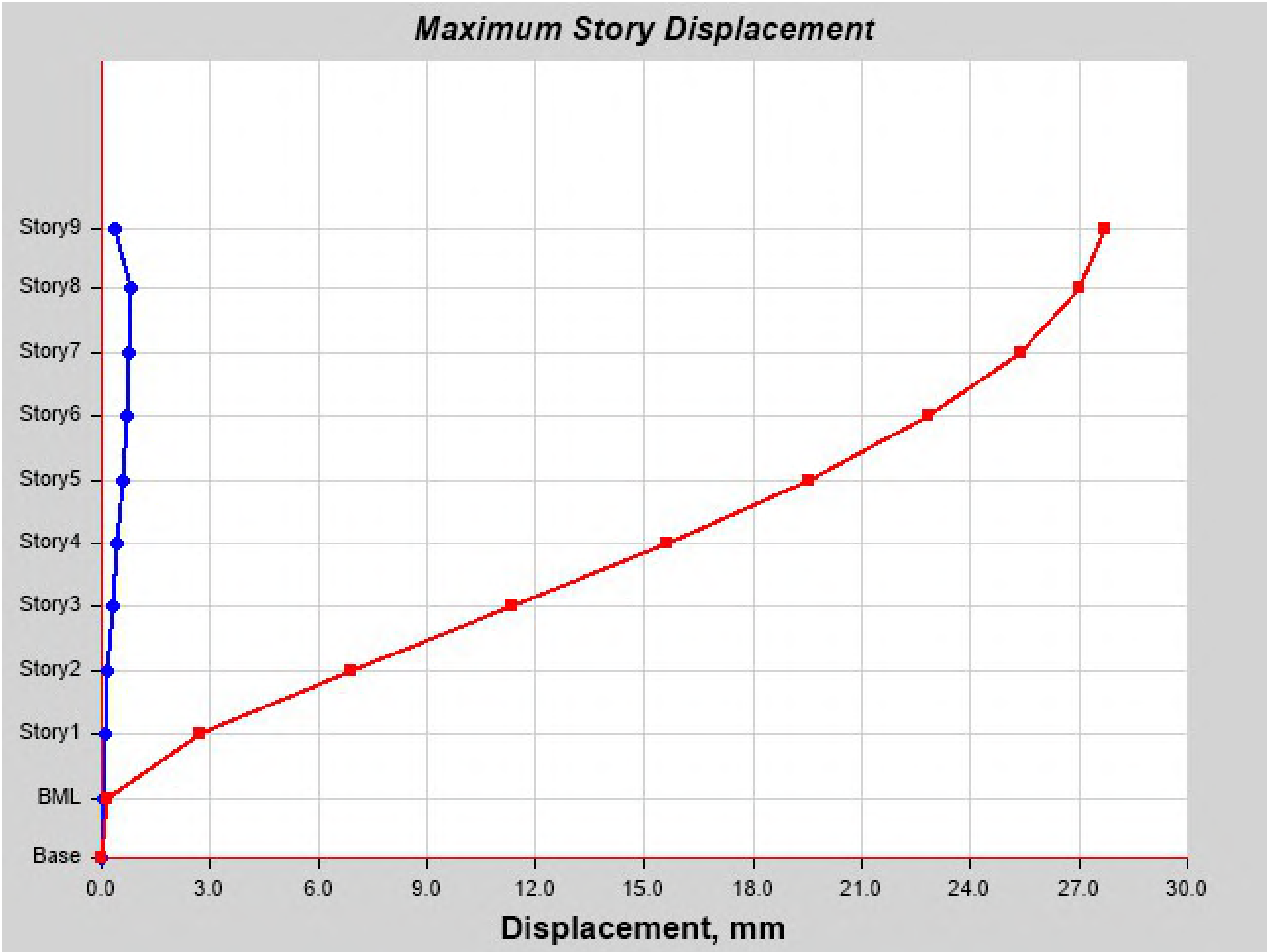
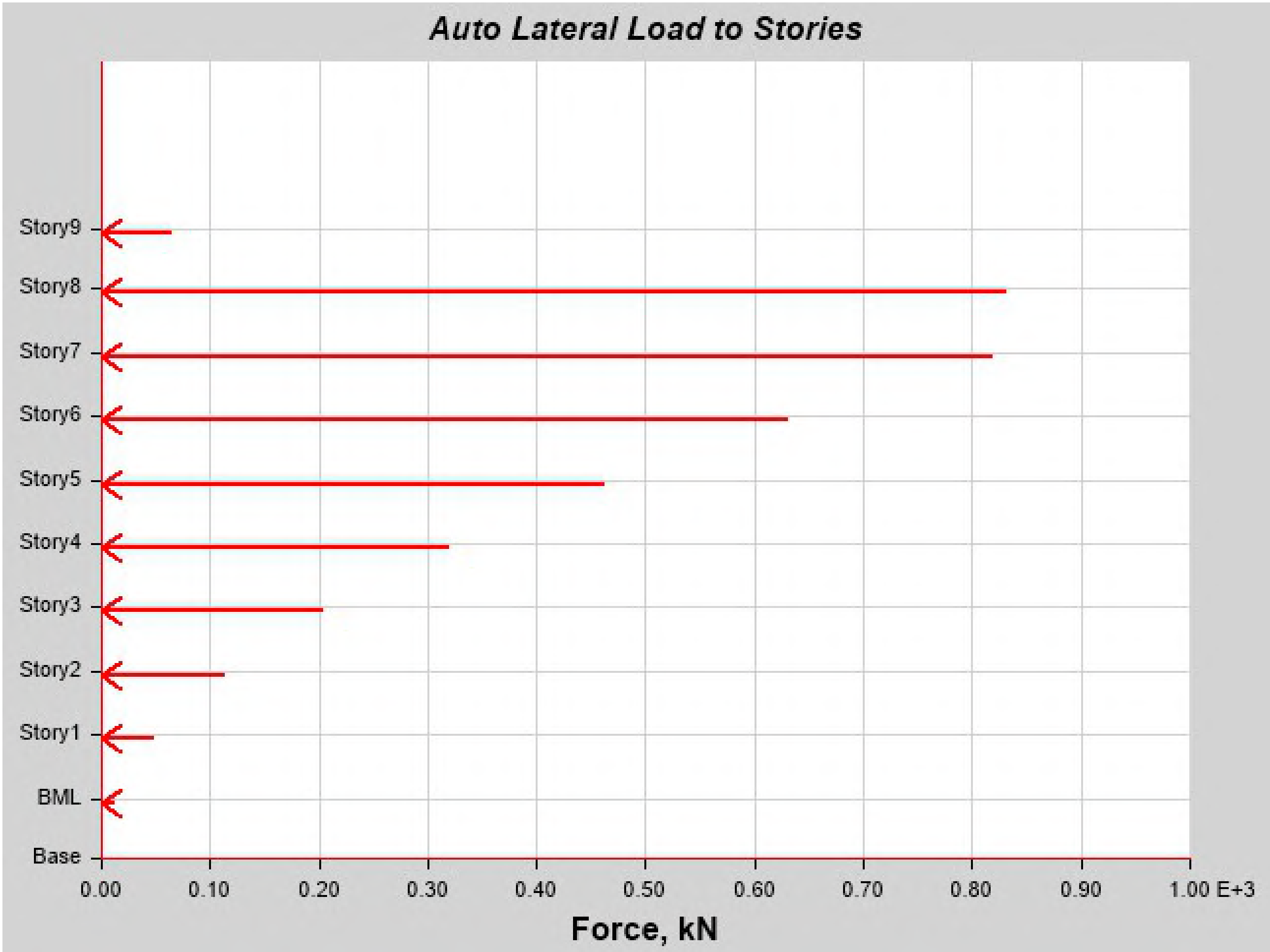
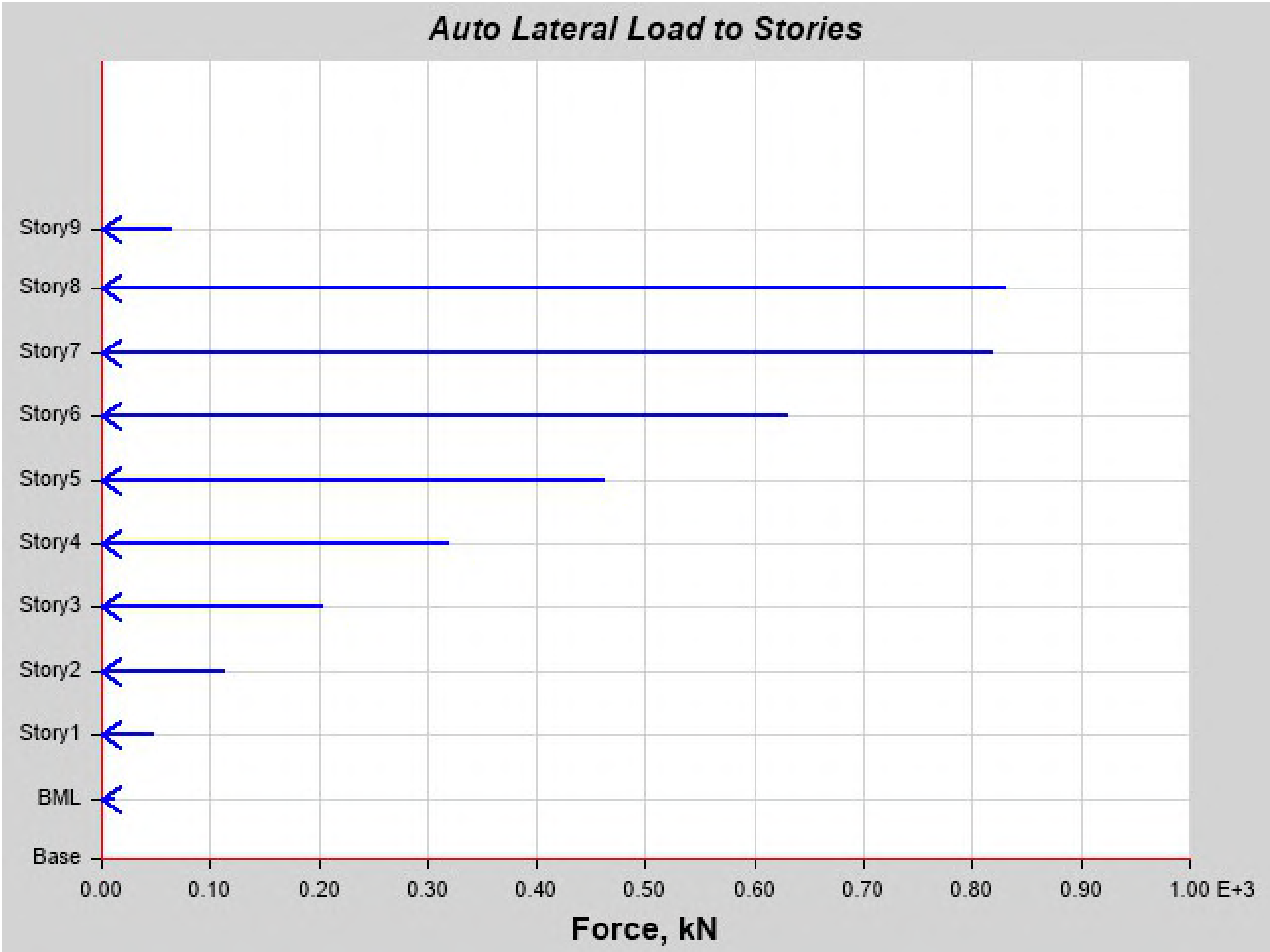


Fig.Maximum Story displacement in EQy direction


Auto Lateral Load to Stories
Story9
Story8
Story7
Story6
Story5
Story4
Story3
Story2
Story1
BML
Base
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00 E+3
Force, kN


Auto Lateral Load to Stories
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BML
Base
0.00
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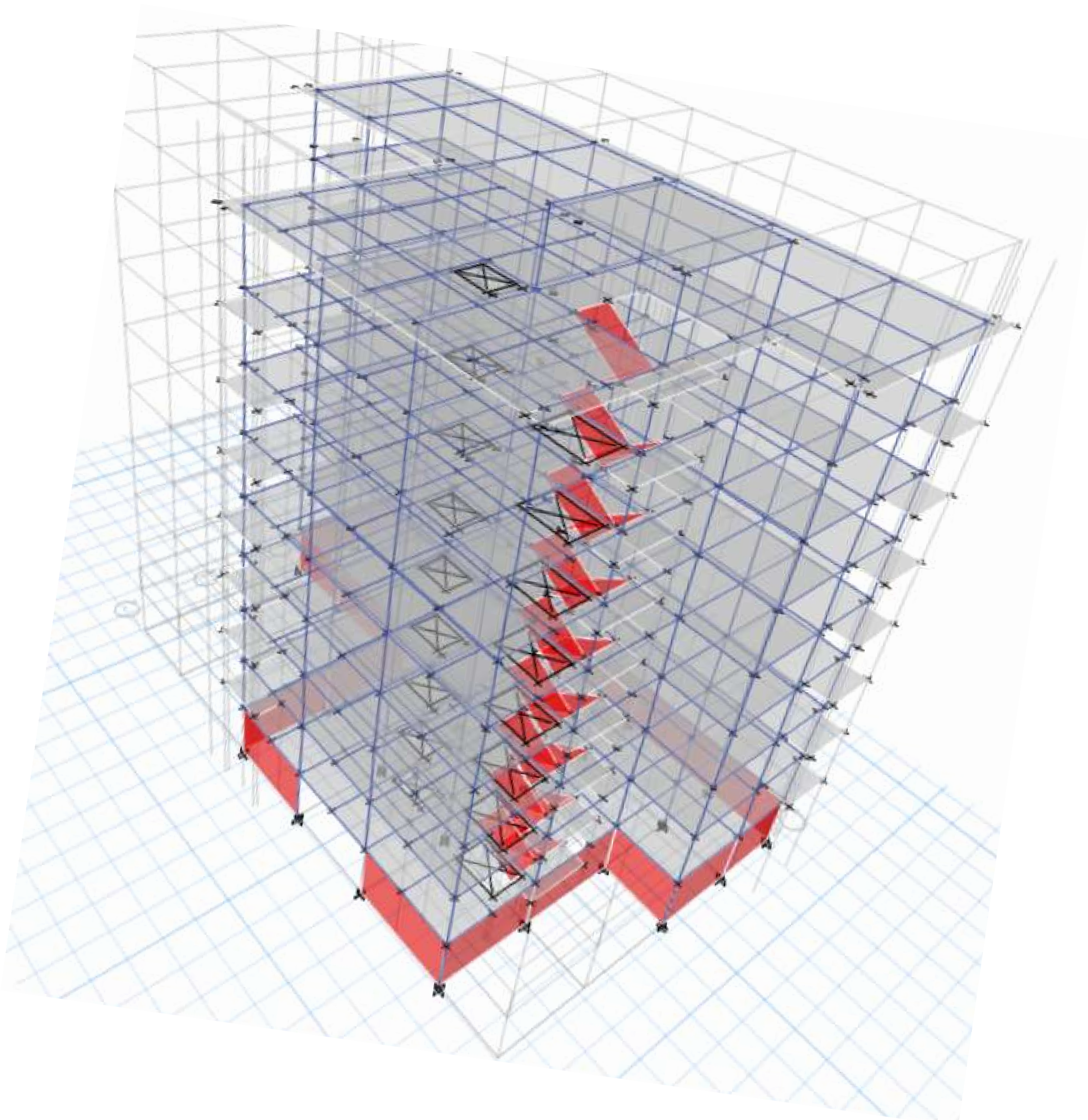
0.80
0.90
1.00 E+3
Force, kN



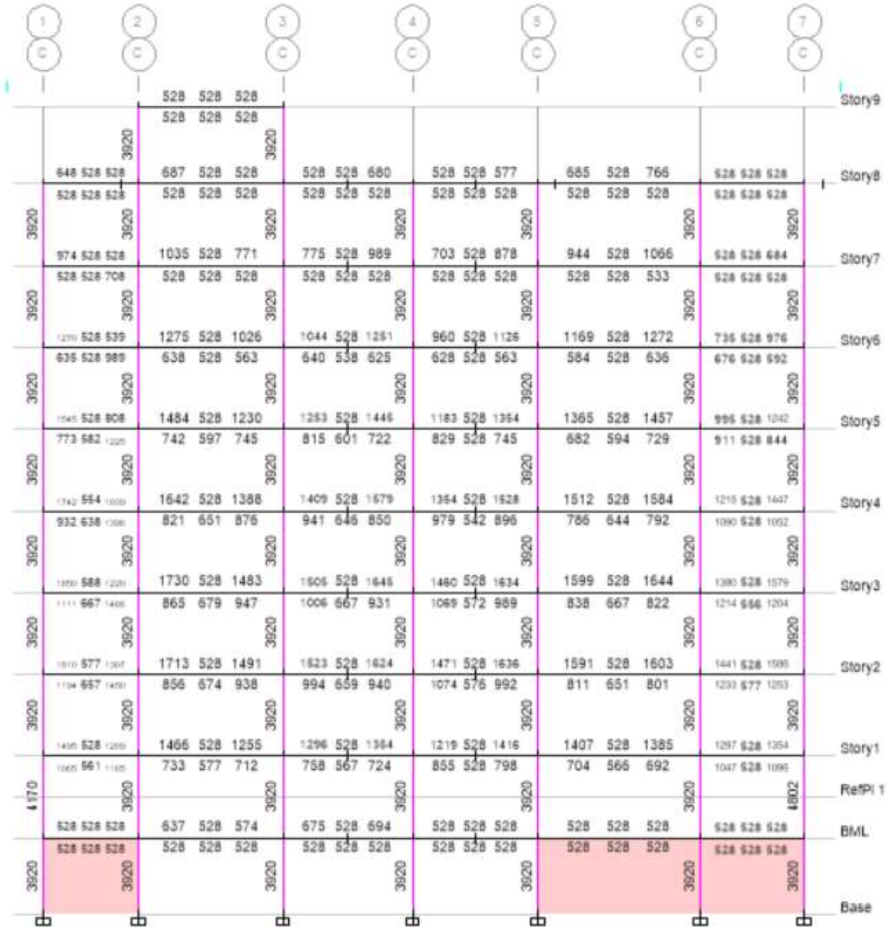


Fig.Longitudinal Reinforcement

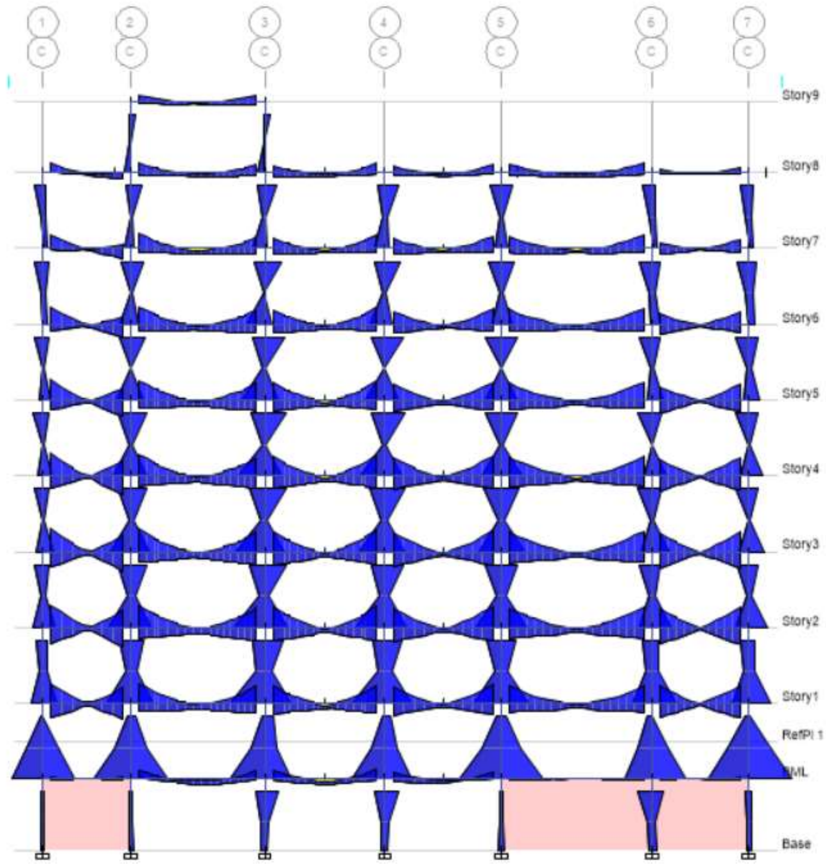


Fig.Bending Moment Diagram

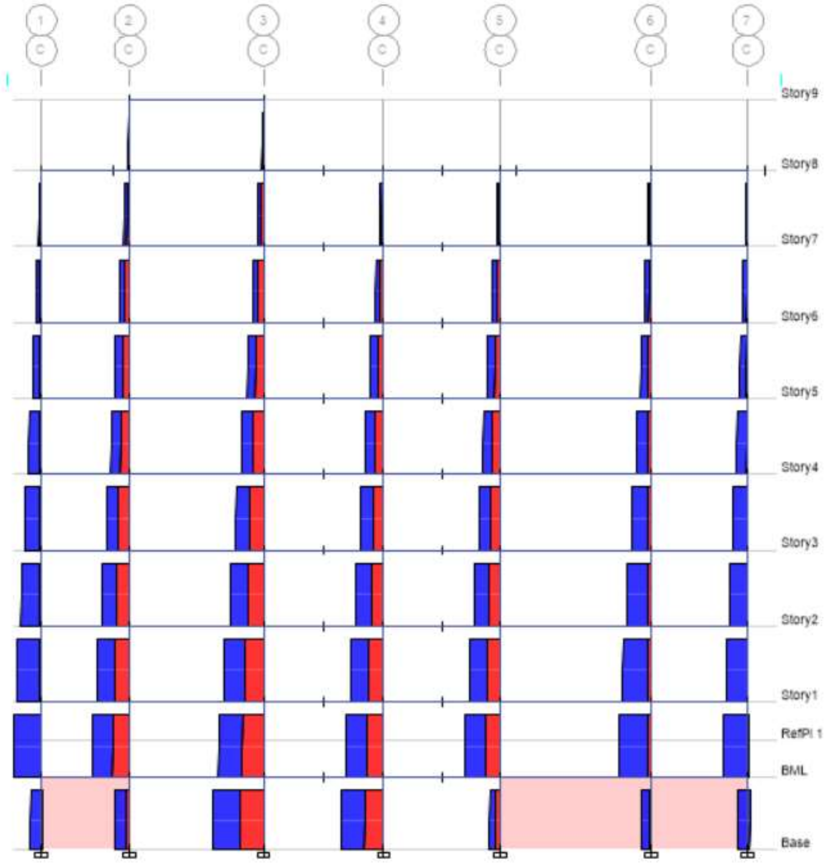


Fig.Axial Force Diagram