Structural Analysis & Design of a Seven Storied Industrial building Using ETABS

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A thesis submitted to the Department of Civil Engineering in partial fulfillment for the degree of Bachelor of Science in Civil Engineering



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DECLARATION

It is hereby declared that this thesis/project or any part of it has not been submitted elsewhere for the award of any degree or diploma.

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ABSTRACT

Structural Design of a building is the responsibility of a civil engineer. In this project A Seven storied building situated in Dhaka zone was designed. this study has been performed following USD method. All the slabs are designed by ACI moment co-efficient methods and Bangladesh National Building Code. Here finite element analysis software ETABS is used to complete this project. Under the study, Design of Seven storied residential Building has been analyses by using the Software ETABS. At the early stage of this report we have shared some information about RCC Building materials. Calculation has been started by the assumption of load, i.e. Dead load, Live load. In case of Environmental loads, i.e. seismic load, wind load. Portal frame analysis has been performed here. The entire load has been factorized by the load factor and load combinations has been chosen from the BNBC/Bangladesh National Building Code.BNBC 2020 (Seismic zone co-efficient Z=0.20,Ct=0.0466) for Earthquake Load.BNBC 2020 (Wind speed = 65.7 mph) with respect to basic wind speeds using the software (ETABS). In beam design in case of moment calculation co-efficient has been used. Required steel has been calculated on the moment values in difference zones of the continuous beam. In terms of column design both concentric load and moment (due to lateral loading) has been considered by checking column strength interaction diagram. Some of the cases, values have been taken being in the conservative sides for ensuring more safety.

ABBREVIATIONS

ACI= American Concrete Institute.

AFD = Axial Force Diagram.

ASTM=American Society for Testing and Materials.

BNBC= Bangladesh National Building Code.

ETABS=Extended Three-Dimensional Analysis of Buildings System.

DL=Dead Load.

LL=Live Load

PW=Partition Wall.

FF=Floor Finish.

SFD=Shear Force Diagram.

BMD= Bending Moment Diagram

UBC=Universal Building Code.

USD=Ultimate Strength Design.

WSD=Working stress Design.

NOTATOINS

Ag = Gross area of column, in^2

As= Area of flexural tension steel, in^2

Av= Area of shear reinforcement within distance “s, in^2

bw= Web width, inch

db= bar diameter.

E= Eccentricity

Es= Modulus of elasticity of steel.

Ec= Modulus of elasticity of concrete.

f’c= Compressive strength of concrete, psi.

fy= Yielding strength of steel, psi.

h= Total depth of beam or slab or column.

I= Moment of inertia.

Ksi= Kip per square inch.

Mu= Moment strength due to factored loads.

Pu= Factored load.

Psi= Pound per square inc

S= Spacing between bars, stirrups or ties.

Vc= Design shear strength of concrete.

Vu= Shear force due factored loads.

RF= Reduction factor.

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CHAPTER-1

INTRODUCTION

1.1:Background of Study

At present world, there is rapid emersion of the new technologies related to safe, economical, stable construction. In such a trend, earthquake has become one of the natural challenging factors for the efficient construction works. It is one of the dominant constraints while designing the frame building in the earthquake prone zone like Bangladesh is a natural phenomenon as old as the history of the earth itself and is considered to be most unpredictable one among all other natural disasters. Now a days, designers and engineers are giving more emphasis towards the earthquake resistance while analyzing and designing any structure to minimize the seismic impact. One being a good designer has to deal with variousstructures ranging from simple ones like the curtailment rods, and electric poles to more complex ones like multi-storied frame buildings, shell roofs, bridges etc. These structures aresubjected to various loads like concentrated loads, uniformly distributed loads, uniformly varying loads, internal or earthquake load and dynamic forces which are considered during the design phase. The structure transfers its load to the supports and ultimately to the ground.While transferring the loads, the members of the structure are subjected to internal forces likeaxial forces, shearing forces, bending and tensional moments which are discussed while analyzing the structures. First of all the most appropriate structural system and initial proportioning of members is done. Generally the initial drawing of architecture is referred.After that the loads are estimated based on functionality and purpose of building based on codes. Then the process of structural analysis and design evolves. Structural Analysis deals with the prediction of performance of a given structure under stipulated loads and other external effects. Structural design deals with the designing (sizing: member sizes, reinforcement details and grading of materials) various members of the structure to resist various forces to which they might be subjected during the life period of the structure. While analysis, frames are analyzed for earthquake as lateral or horizontal load. During the earthquake, structural and non-structural damages occur in which both of them are hazardous to occupants. When earthquake shaking occurs, a building gets thrown from side to side or upand down i.e. ground moves violently side Structural Analysis and Design of Commercial and Residential Building for Earthquake Resistance by side and building stay at rest. In our context, the analysis of earthquake is based on seismic coefficient design method as describedin BNBC 2020. The project report has been prepared in complete conformity with various provisions in Bangladesh National Building code.

1.2 Objectives of study:

- To study about the various components of a Seven storied Factory Building.
- To study different types of loads and their impact.
- To analyze and design of a Seven storied Factory Building using ETABS software.
- To design the pile foundation based on soil test repor

1.3 Scope of study:

A typical Factory Building has been chosen for the study of building frame structure construction in Bangladesh at Dhaka. Hence one can easily design structural component of a building. All types of loads have to considered to the building frames and analyze. ACI/BNBC building design codes specification will follow to get perfect result so that the objective of this study will be fulfilled.

1.4 Possible outcome:

Structural design is an important thing for a building. By providing a proper design of structural thing an economic building could be built. Under design is very harmful for any kind of structure in which animals are dwelling in. But over design in also harmful for such kind of structures at times. So, it is mandatory to design any building optimistically. This optimistic design is the possible Outcome of the study as design as given below:

1. Safe and economic design of all structural elements.
2. A safe foundation may be found by providing pile foundation.
3. Pile foundation will reduce possibility of settlement.

1.5 Outline of the Methodology

First of all we design the slab of the building considering maximum size of panel and then we design the beam and column considering the loads and moments coming from ETABS analysis. After that we design the stair of the building with stair beam. Finally we design the pile and pile cap for the building considering column load receipt from ETABS.

1.6 Organization of the Thesis

This research paper contains seven chapters

Chapter 1 Introduction The first chapter of the research paper comprises background, objectives, scope of the study and outline of the methodology.

Chapter 2 Literature Review This chapter includes the description of terminology related to the study such as types of building and their uses, components of building, design criteria, design method, design loads which have to considered for design of a building, design codes and specifications, software used to analyze the buildings, previous researches etc.

Chapter 3 Methodology In chapter three, we discuss about the methodology flow chart and described the analyzing methods

Chapter 4 Architectural Plan and Design and Analysis of Building This chapter includes an architectural plan for residential building, beam and column layout, load combinations, model development and finally analyzes the model. includes analysis of building frame by etabs. Beam, Column analysis by axial force and lateral force include vertical loads, earthquake and wind loads.

Chapter 5 Structural Design and Conclusion and Recommendation the chapter six includes the analysis result and design beam column slab, pile and pile cap detailing by design of Etabs data and analysis. includes the summary of the study and recommendation for the future study.

CHAPTER-2
LITERATURE REVIEW

2.1:General:

Concrete is one of the most popular materials for buildings because it has high compressive strength, flexibility in its form and it is widely available. The history of concrete usage dates back for over a thousand years. Contemporary cement concrete has been used since the early nineteenth century with the development of Portland cement. Despite the high compressive strength, concrete has limited tensile strength, only about ten percent of its compressive strength and zero strength after cracks develop. In the late nineteenth century, reinforcing materials, such as iron or steel rods, began to be used to increase the tensile strength of concrete. Today steel bars are used as common reinforcing material. Usually steel bars have over 100 times the tensile strength of concrete; but the cost is higher than concrete. Therefore, it is most economical that concrete resists compression and steel provides tensile strength. Also it is essential that concrete and steel deform together and deformed reinforcing bars are being used to increase the capacity to resist bond stresses.

2.2:Structure

Structures can include buildings, bridges, in-ground structures, footings, frameworks and space frames, including those for motor vehicles, space vehicles, ships, aero planes and cranes. They can be composed of any structural material including composites and novel materials. Structural engineering is a creative profession that makes a significant contributionto infrastructure, indust -ry as well as residential and recreational developments.

Types of Structure: There are two types of structures. These are given below:

1. Super structure &
2. Sub structure.

2.3:Super Structure

A superstructure is an upward extension of an existing structure above a baseline. This term is applied to various kinds of physical structures such as buildings, bridges or ships having the degree of freedom zero. It also a part of structure that is above plinth level (P.L).
Generally columns are constructed in super structure. There are many characteristics involve in super structures-

A physical or conceptual structure extended or developed from a basic form.

- The part of a building or other structure above the foundation.
- The parts of a ship's structure above the main deck.
- The rails, sleepers, and other parts of a railway.
- In Marxist theory, the ideologies or institutions of a society as distinct from the basic processes and direct social relations of material production and economics.

Important Components of a Super Structure A super structure consists of various components. The major components of a super structure are as below-

- Roof
- Floor/slab
- Beam

- Column
- Stair
- Wall

2.4:Sub structure (Foundation)

Foundation is the part of structure below plinth level up to the soil. It is in direct contact of soil and transmits load of super structure to soil. Generally it is below the ground level. If some part of foundation is above ground level, it is also covered with earth filling. This portion of structure is not in contact of air, light etc. or to say that it is the hidden part of the structure.

Foundations can be divided into mainly two categories-

- Shallow foundation
- Deep foundation

Shallow foundation

Shallow foundations are those founded near to the finished ground surface; generally where the founding depth (D_f) is less than the width of the footing and less than 3m. Shallows foundations are used when surface soils are sufficiently strong and stiff to support the imposed loads; they are generally unsuitable in weak or highly compressible soils, such as poorly-compacted fill peat etc.

Types of shallow foundation.

Following are the major types of shallow foundations -

- Mat/Raft footings
- Cantilever/Strap footings
- Pad footing/Column footing
- Wall Footings

Deep foundations

Deep foundations are those founding too deeply below the finished ground surface for their base bearing capacity to be affected by surface conditions, this is usually at depths >3 m below finished ground level. They include piles, piers and caissons or compensated foundations using deep basements and also deep pad or strip foundations. Deep foundations can be used to transfer the loading to deeper, more competent strata at depth if unsuitable soils are present near the surface. Deep foundation can be broadly divided into the following types-

- Piles.
- Caissons.
- Shaft foundations.
- Cylinders
- Buoyancy rafts (hollow box foundations)
- Basements

Pile Foundation

Pile foundation is more commonly used in building construction. Piles are relatively long, slender members that transmit foundation loads through soil strata of low bearing capacity to deeper soil or rock strata having a high bearing capacity.

- The load of the super structure is heavy and its distribution is uneven.
- The top soil has poor bearing capacity.
- The subsoil water level is high.
- There is large fluctuations in subsoil water level.
- Canal or deep drainage lines exist near the foundation.
- The structure is situated on the sea shore or river bed.

In addition to supporting structures, piles are also used to anchor structures against uplift forces and to assist structures in resisting lateral and overturning forces. There are various types of pile foundation depending upon some factors. Based on the function or use, piles may be classified as-

a) Bearing pile.
b) Friction piles.
c) Compaction piles
d) Tension or Uplift piles
e) Anchor pile
f) Fender piles
g) Sheet piles
h) Batter piles

Based on materials and composition, piles can be classified as follows-

a) Concrete piles
b) Timber piles
c) Steel piles
d) Composite piles

Necessity of pile foundations

a) At the time when the strata at or just beneath the ground surface is greatly compressible and very weak or loose for carrying the load transferred from the structure above.
b) When the plan of the structure is irregular relative to its outline and load distribution. This may lead to the unpleasant non-uniform settlement when the shallow foundation is constructed. Thus, a pile foundation eliminates differential settlement.
c) The pile foundations are perfect for transmission of different structural loading/forces via deep water to the firm stratum below.
d) The pile foundations are proficiently used to withstand the horizontal forces as well as supports. The vertical forces/loads in the earth retaining structures and tall structure which are subjected to horizontal forces of wind and earthquake.
e) The plies are extremely useful when the soil conditions are in very loose/weak conditions that a washout, erosion and scour of soil underneath leads to the collapse of shallow foundations.

f) Pile foundation is required for the transmission of structural load though deep water to a firm stratum. V.

g) Pile foundation is used to resist horizontal forces in addition to support the vertical load in earth retaining structure & tall structure that are subjected to horizontal forces due to wind and earth quake.

2.5:Loads on building

The determination of the loads acting on a structure is a complex problem. The nature of the loads varies essentially with the architectural design, the materials and the location of the structure. Loading conditions on the same structure may change from time to time or may change rapidly with time. There are various types of loads acting on structures are as-

a) Dead Loads (DL)
b) Live Loads (LL)
c) Impact loads
d) Earthquake loads.
e) Wind loads

a)Dead Load: Structures are designed to withstand forces that are placed on the structure,whether it is a bridge or a building, by the actual weight of the building materials required to accommodate the construction of the project

b)Live loads: include any temporary or transient forces. That act on a building or structural element. Typically, they include people, furniture, vehicles, and almost everythingelse that can be moved throughout a building.

c)Impact Load: An Impact load is one whose time of application on a material is lessthan one third of the natural period of vibration of that material. This load is applied dynamically.

There are two steps

1. Determine an equivalent mass of the structure being hit by calculating the total kinetic energy of the structure and object causing the impact load.

2. Perform an energy balance assuming all kinetic and potential energy of the structure and impacting object is transferred to strain energy in the structure being hit. First case is difficult. Therefore assume the structure being hitted is massless. So, in this case we required only second step.

d):Earthquake Load

ⅰ: General

Minimum design earthquake forces for buildings, structures or components there of shall be determined in accordance with the provisions of Sec 2.5. Some definitions and symbols relevant for earthquake resistant design for buildings are provided in Sections 2.1.3 and 2.1.4. Section 2.5.2 presents basic earthquake resistant design concepts. Section 2.5.3 describes procedures for soil investigations, while Sec 2.5.4 describes procedures for determining earthquake ground motion for design. Section 2.5.5 describes different types of buildings and structural systems which possess different earthquake resistant characteristics. Static analysis procedures for design are described in Sections 2.5.6, 2.5.7 and 2.5.12. Dynamic analysis procedures are dealt with in Sections2.5.8 to 2.5.11. Section 2.5.13 presents combination of earthquake loading effects in different directions and with other loading effects. Section 2.5.14 deals with allowable drift and deformation limits. Section 2.5.15 addresses design of nonstructural components in buildings. Section 2.5.16 presents design considerations for buildings with seismic isolation systems. Design for soft story condition in buildings is addressed in Sec 2.

ⅱ :Earthquake Resistant Design–Basic Concepts General principles:

The purpose of earthquake resistant design provisions in this Code is to provide guidelines for the design and construction of new structures subject to earthquake ground motions in order to minimize the risk to life for all structures, to increase the expected performance of higher occupancy structures as compared to ordinary structures, and to improve the capability of essential structures to function after an earthquake. It is not economically feasible to design and construct buildings without any damage for a major earthquake event. The intent is therefore to allow inelastic deformation and structural damage at preferred locations in the structure without endangering structural integrity and to prevent structural collapse during a major earthquake. The expected earthquake ground motion at the site due to all probable earthquakes may be evaluated in deterministic or probabilistic terms. The ground motion at the site due to an earthquake is a complex phenomenon and depends on several parameters such as earthquake magnitude, focal depth, earthquake source characteristics, distance from earthquake epicenter, wave path characteristics, as well as local soil conditions at the site. The seismic zoning map divides the country into four seismic zones with different expected levels of intensity of ground motion. Each seismic zone has a zone coefficient which provides expected peak ground acceleration values on rock/firm soil corresponding to the maximum considered earthquake (MCE). The design basis earthquake is taken as 2/3 of the maximum considered earthquake. The effects of the earthquake ground motion on the structure is expressed in terms of an idealized elastic design acceleration response spectrum, which depends on (a) seismic zone coefficient and local soil conditions defining ground motion and (b) importance factor and response reduction factor representing building considerations. The earthquake forces acting on the structure is reduced using the response modification/reduction factor *R* in order to take advantage of the inelastic energy dissipation due to inherent ductility and redundancy in the structure as well as material over-strength. The importance factor I increased design forces for important structures. If suitable lateral force resisting systems with adequate ductility and detailing and good construction are provided, the building can be designed for a response reduction factor *R* which may be as high as 5 to 8. Because of this fact, the provisions of this Code for ductility and detailing need to be satisfied even for structures and members for which load combinations that do not contain the earthquake effect indicate larger demands than combinations including earthquake. The elastic deformaton

calculated under these reduced design forces are multiplied by the deflection amplification factor, Cd to estimate the deformation likely to result from the design earthquake. The seismic design guidelines presented in the section are based on the assumption that the soil supporting the structure will not liquefy, settle or slide due to loss of strength during the earthquake. Reinforced and pre-stressed concrete members shall be suitably designed to ensure that premature failure due to shear or bond does not occur. Ductile detailing of reinforced concrete members is of prime importance. In steel structures, members and their connections should be so proportioned that high ductility is obtained, avoiding premature failure due to elastic or inelastic buckling of any type. The building structure shall include complete lateral and vertical force-resisting systems capable of providing adequate strength,stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. The design ground motions shall be assumed to occur along any horizontal direction of a building structure. The adequacy of the structural systems shall be demonstrated through the construction of a mathematical model and evaluation of this model for the effects of design ground motions. The design seismic forces, and their distribution over the height of the building structure, shall be established in accordance with one of the applicable procedures indicated in Sec 2.5 and the correspondinginternal forces and deformations in the members of the structure shall be determined. The deformation of the structure shall not exceed the prescribed limits under the action of the design seismic forces.

iii: Earthquake Ground Motion

Bangladesh can be affected by moderate to strong earthquake events due to its proximity tothe collision boundary of the Northeast moving Indian plate and Eurasian Plate. Strong historical earthquakes with magnitude greater than 7.0 have affected parts of Bangladesh inthe last 150 years, some of them had their epicenters within the country. A brief descriptionof the local geology, tectonic features and earthquake occurrence in the region is given in Appendix B

Table 2.1: Site Classification Based on Soil Properties(reference: "Bangladesh National Building Code" (2020)).

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity, (m/s)	SPT Value, (blows/30cm)	Undrained shear strength (kPa)
SA	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface	> 800	--	--
SB	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800	> 50	> 250
SC	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
SD	Deposits of loose-to-medium cohesion less soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70

SE	A soil profile consisting of a surface alluvium layer with Vs values of type SC or SD and thickness varying between about 5 m and 20 m, underlain by stiffer material with Vs > 800 m/s.	--	--	--
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	--	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types SA to SE or S_1	--	--	--

iv : Seismic zoning

The intent of the seismic zoning map is to give an indication of the Maximum Considered Earthquake (MCE) motion at different parts of the country. In probabilistic terms, the MCE motion may be considered to correspond to having a 2% probability of expedience within a period of 50 years. The country has been divided into four seismic zones with different levels of ground motion. Table 6.2.14 includes a description of the four seismic zones. Figure 6.2.24 presents a map of Bangladesh showing the boundaries of the four zones. Each zone has a seismic zone coefficient (Z) which represents the maximum considered peak ground acceleration (PGA) on very stiff soil/rock (site class SC) in units of g (acceleration due to gravity). The zone coefficients (Z) of the four zones are: ,Z=0.20 (Zone 2). Table 2.3lists zone coefficients for some important towns of Bangladesh. The most severe earthquake prone zone, Zone 4 is in the northeast which includes Sylhet and has a maximum PGA value of 0.36g. Dhaka city falls inthe moderate seismic intensity zone with Z=0.2,.

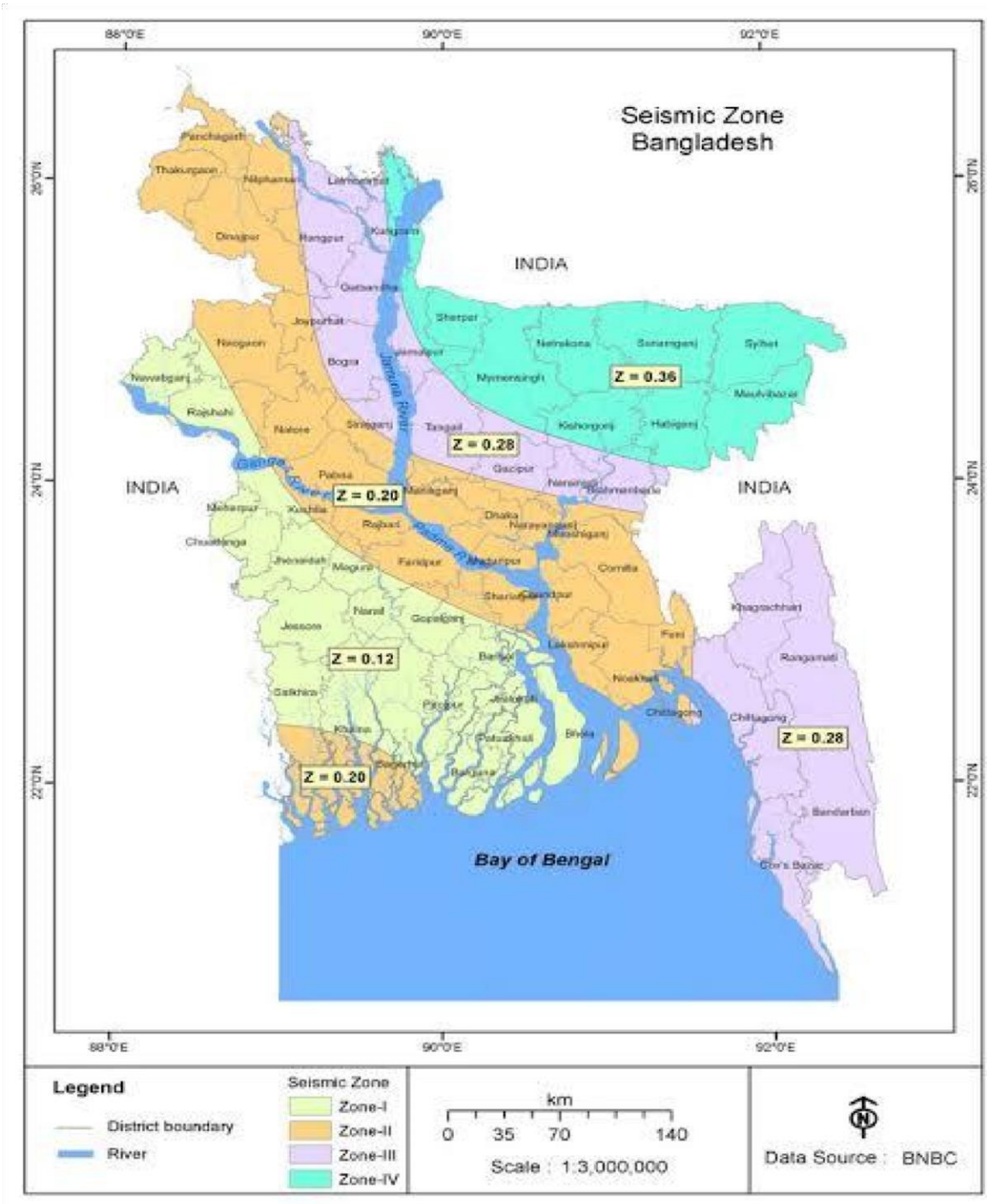


Figure 2.1 Seismic zoning map of Bangladesh

(reference:Geological Survey of Bangladesh (Nov.1979). "Seismic Zoning Map of Bangladesh and out of a code for Earthquake Resistance Design of structure".Survey of Bangladesh, Dhaka, p.p. 1-8.)

Table 2.2: Description of Seismic Zones

Seismic Zone	Location	Seismic Intensity	Seismic Zone Co-efficient, Z
1	Southwestern part including Barisal, Khulna, Jessore, Rajshahi	Low	0.12
2	Lower Central and Northwestern part including Noakhali, Dhaka, Pabna, Dinajpur, as well as Southwestern corner including Sundarbans	Moderate	0.20
3	Upper Central and Northwestern part including Brahmanbaria,Sirajganj, Rangpur	Severe	0.28
4	Northeastern part including Sylhet, Mymensingh, Kurigram	Very Severe	0.36

Table 2.3 Seismic Zone Co-efficient *Z* for Some Important Towns of Bangladesh

Town	Z	Town	Z	Town	Z	Town	Z
Bagerhat	0.12	Gaibandha	0.28	Magura	0.12	Patuakhali	0.12
Bandarban	0.28	Gazipur	0.20	Manikganj	0.20	Pirojpur	0.12
Barguna	0.12	Gopalgonj	0.12	Maulvibazar	0.36	Rajbari	0.20
Barisal	0.12	Habiganj	0.36	Meherpur	0.12	Rajshahi	0.12
Bhola	0.12	Jaipurhat	0.20	Mongla	0.12	Rangamati	0.28
Bogra	0.28	Jamalpur	0.36	Munshiganj	0.20	Rangpur	0.28
Brahmanbaria	0.28	Jessor	0.12	Mymensing	0.36	Satkhira	0.12
Chandpur	0.20	Jhalokati	0.12	Narail	0.12	Shariatpur	0.20
Chapainabaganj	0.12	Jhenaidah	0.12	Narayanganj	0.20	Sherpur	0.36
Chittagong	0.28	Khagrachari	0.28	Narsingdi	0.28	Sirajganj	0.28
Chuadanga	0.12	Khulna	0.12	Natore	0.20	Srimangal	0.36
Comilla	0.20	Kishoreganj	0.36	Naogaon	0.20	Sunamganj	0.36
Cox's Bazar	0.28	Kurigram	0.36	Neterakona	0.36	Sylhet	0.36
Dhaka	0.20	Kushtia	0.20	Nilphamari	0.12	Tangail	0.28
Dinajpur	0.20	Lakshmipur	0.20	Noakhali	0.20	Thakurgaon	0.20
Faridpur	0.20	Lalmanirhat	0.28	Pabna	0.20		
Feni	0.20	Madaripur	0.20	Panchagarh	0.20		

We know, The special acceleration for the design earthquake is given by the following equation:

$$Sa = \frac{2}{3}\frac{ZI}{R}Cs \qquad \text{BNBC Eq. 6.2.34}$$

Where

S_a= Design spectral acceleration (in units of g which shall not be less than *0.67ßZIS*

ß= coefficient used to calculate lower bound for S_a. Recommended value for *ß* is 0.15

Z = Seismic zone coefficient, as defined in Sec 2.5.4.2

I= Structure importance factor, as defined in Sec 2.5.5.1

R= Response reduction factor which depends on the type of structural system given in Table 6.2.19.

The ratio $\frac{I}{R}$ cannot be greter then one.

C_S= Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class) as defined by Equations 6.2.35a to 6.2.35d.

$Cs = S(1 + \frac{T}{T_B}(2.5\eta - 1))$	for	$0 \le T \le T_B$	BNBC Eq. 6.2.35a
$Cs = 2.5S\eta$	for	$T_B \le T \le T_C$	BNBC Eq. 6.2.35b
$Cs = 2.5S\eta(\frac{T_C}{T})$	for	$T_C \le T \le T_D$	BNBC Eq. 6.2.35c
$Cs = 2.5S\eta(\frac{T_C T_D}{T^2})$	for	$T_D \le T \le 4\ sec$	BNBC Eq. 6.2.35d

C_S depends on *S* and values of T_B, T_C and T_D, (Figure 6.2.25) which are all functions of the site class. Constant C_S value between periods T_B and T_C represents constant spectral acceleration.

S= Soil factor which depends on site class and is given in Table 6.2.16

T= Structure (building) period as defined in Sec 2.5.7.2

T_B = Lower limit of the period of the constant spectral acceleration branch given in Table 6.2.16 as a function of site class.

T_C =Upper limit of the period of the constant spectral acceleration branch given in Table 6.2.16 as a function of site class.

T_D =Lower limit of the period of the constant spectral displacement branch given in Table 6.2.16 as a function of site class.

η = Damping correction factor as a function of damping with a reference value of η=1 for 5% viscous damping. It is given by the following expression:

$$\eta = \sqrt{10/(5 + \xi)} \geq 0.55 \quad \text{(for 5\% viscous damping)}$$

Where, ξ is the viscous damping ratio of the structure, expressed as a percentage of critical damping. The value of η cannot be smaller than 0.55.

Table 2.4: Site Dependent Soil Factor and Other Parameters Defining Elastic Response Spectrum

Soil Type	S	T_B (S)	T_C	T_D
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

Table 2.5 Importance Factors for Buildings and Structures for Earthquake design

Occupancy Category	Importance Factor I
I, II	1.00
III	1.25
IV	1.50

Table 2.6 Seismic Design Category of Buildings(reference: PED Technical Training Institute (1988). "Tall structures design & allidedtopics".Public works Department Government Republic of Bangladesh, Dhaka, p.p.68, 139-140)

Site Class	Occupancy Category I,II and III				Occupancy Category IV			
	Zone 1	Zone 2	Zone 3	Zone 4	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S1, S2	D	D	D	D	D	D	D	D

e) WINDLOAD

ⅰ:General

Scope: Buildings and other structures, including the Main Wind-Force Resisting System (MWFRS) and all components and cladding thereof, shall be designed and constructed to resist wind loads as specified herein. Allowed Procedures: The design wind loads for buildings and other structures, including the MWFRS and component and cladding elements thereof, shall be determined using one of the following procedures: Method 1: Simplified

Procedure as specified in Sec 2.4.2 for buildings and structures meeting the requirements specified therein;
Method 2: Analytical Procedure as specified in Sec 2.4.3 for buildings and structures meetingthe requirements specified therein;
Method 3: Wind Tunnel Procedure as specified in Sec 2.4.16.
Wind Pressures: Acting on opposite faces of each building surface. In the calculation of design wind loads for the MWFRS and for components and cladding for buildings, the algebraic sum of the pressures acting on opposite faces of each building surface shall be takeninto account.

Minimum Design Wind Loading:
The design wind load, determined by any one of the procedures specified in Sec 2.4.1, shallbe not less than specified in this Section.
Main Wind-Force Resisting System: The wind load to be used in the design of the MWFRSfor an enclosed or partially enclosed building or other structure shall not be less than 0.5 kN/m2 multiplied by the area of the building or structure projected onto a vertical plane normal to the assumed wind direction. The design wind force for open buildings and other structures shall be not less than 0.5 kN/m2 multiplied by the area Af. Components and Cladding**:** The design wind pressure for components and cladding of buildings shall not be less than a net pressure of 0.5 kN/m^2 acting in either direction normal to the surface.

ii :Method 2: Analytical Procedure

Scopes and limitations

A building or other structure whose design wind loads are determined in accordance with this Section shall meet all of the following conditions:

1. The building or other structure is a regular-shaped building or structure as defined in Sec 2.1.3.
2. The building or other structure does not have response characteristics making it subject to a cross-wind loading, vortex shedding, instability due to galloping or flutter; or does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

The provisions of this Section take into consideration the load magnification effect caused bygusts in resonance with a long-wind vibrations of flexible buildings or other structures.
Buildings or other structures not meeting the requirements of Sec 2.4.2, or having unusual shapes or response characteristics shall be designed using recognized literature documentingsuch wind load effects or shall use the wind tunnel procedure specified in Sec 2.4.16.2.

Shielding

There shall be no reductions in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

Air permeable cladding

Design wind loads determined from Sec 2.4.3 shall be used for air permeable cladding unless approved test data or recognized literature demonstrate lower loads for the type of air permeable cladding being considered.

iii : Design procedure

(1) The basic wind speed V and wind directionality factor Kd shall be determined in accordance with Sec 2.4.4.
(2) An importance factor I shall be determined in accordance with Sec 2.4.5.
(3) An exposure category or exposure categories and velocity pressure exposure coefficient Kz or K_h, as applicable, shall be determined for each wind direction in accordance with Sec 2.4.6.
(4) A topographic factor Kzt shall be determined in accordance with Sec 2.4.7.
(5) A gust effect factor G or Gf, as applicable, shall be determined in accordance with Sec 2.4.8.
(6) An enclosure classification shall be determined in accordance with Sec 2.4.9.
(7) Internal pressure coefficient GC_{pi} shall be determined in accordance with Sec 2.4.10.1.
(8) External pressure coefficients C_p or GC_{pi}, or force coefficients C_f, as applicable, shall be determined in accordance with Sections 2.4.10.2 or 2.4.10.3, respectively.
(9) Velocity pressure q_z or q_h, as applicable, shall be determined in accordance with Sec 2.4.9.5.
(10) Design wind load P or F shall be determined in accordance with Sec 2.4.11

iv :Basic Wind Speed

The basic wind speed, used in the determination of design wind loads on buildings and other structures shall be as given in Figure 6.2.1 except as provided in Sec 2.4.4.1. The wind shall be assumed to come from any horizontal direction.

v :Special wind regions

The basic wind speed shall be increased where records or experience indicate that the wind speeds are higher than those reflected in Figure 6.2.1. Mountainous terrain, gorges, and special regions shall be examined for unusual wind conditions. The authority having jurisdiction shall, if necessary, adjust the values given in Figure 6.2.1 to account for higher local wind speeds. Such adjustment shall be based on adequate meteorological Information and other necessary data

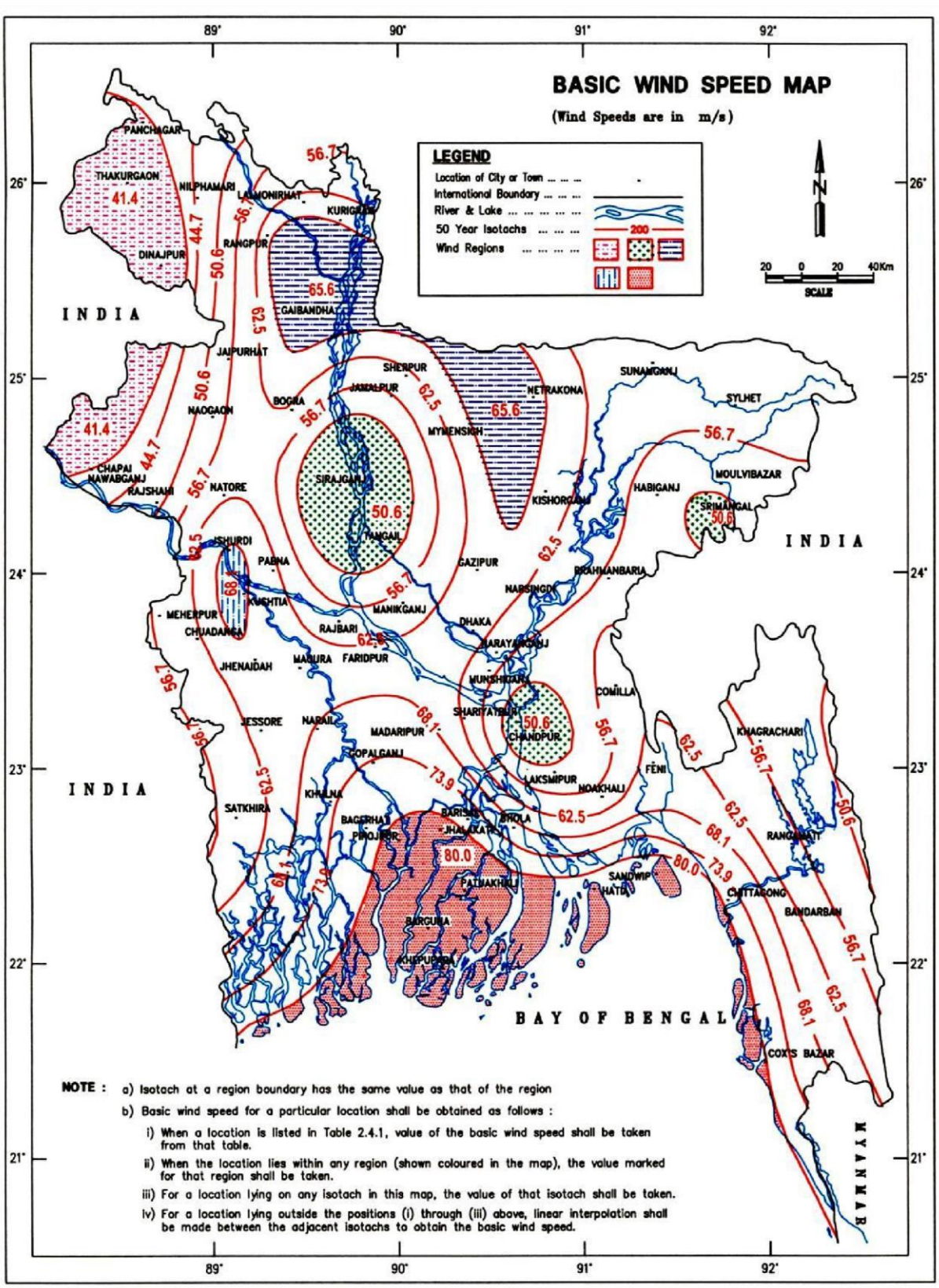


Figure 2.2 Basic Wind Speed map of Bangladesh

(reference:Bangladesh National Building Code" (2020).)

Table 2.7 Basic Wind Speeds, V, for Selected Locations in Bangladesh (reference:BNBC 2020)

Location	Basic Wind Speed (m/s)	Location	Basic Wind Speed (m/s)
Angarpota	47.8	Lalmanirhat	63.7
Bagerhat	77.5	Madaripur	68.1
Bandarban	62.5	Magura	65.0
Barguna	80.0	Manikganj	58.2
Barisal	78.7	Meherpur	58.2
Bhola	69.5	Maheshkhali	80.0
Bogra	61.9	Maulvibazar	53.0
Brahmanbaria	56.7	Munshiganj	57.1
Chandpur	50.6	Mymensing	67.4
Chapainabaganj	41.4	Naogaon	55.2
Chittagong	80.0	Narail	68.6
Chuadanga	61.9	Narayanganj	61.1
Comilla	61.4	Narsingdi	59.7
Cox's Bazar	80.0	Natore	61.9
Dahagram	47.8	Neterakona	65.6
Dhaka	65.7	Nilphamari	44.7
Dinajpur	41.4	Noakhali	57.1
Faridpur	63.1	Pabna	63.1
Feni	64.1	Panchagarh	41.4
Gaibandha	65.6	Patuakhali	80.0
Gazipur	66.5	Pirojpur	80.0
Gopalgonj	74.5	Rajbari	59.1
Habiganj	54.2	Rajshahi	49.2
Hatiya	80.0	Rangamati	56.7
Ishurdi	69.5	Rangpur	65.3
Joypurhat	56.7	Satkhira	57.6
Jamalpur	56.7	Shariatpur	61.9
Jessor	64.1	Sherpur	62.5
Jhalokati	80.0	Sirajganj	50.6
Jhenaidah	65.0	Srimangal	50.6
Khagrachari	56.7	St. Martin’s Island	80.0
Khulna	73.3	Sunamganj	61.1
Kutubdia	80.0	Sylhet	61.1
Kishoreganj	64.7	Sandwip	80.0
Kurigram	65.6	Tangail	50.6
Kushtia	66.9	Teknaf	80.0
Lakshmipur	51.2	Thakurgaon	41.4

ETABS

ETABS means Extended Three-dimensional Analysis of Building Systems. ETABS is a special-purpose computer program developed specifically for building structures. It provides the Structural Engineer with all the tools necessary to create, modify, analyze, design and optimize building models. These features are fully integrated in a single, Windows-based, graphical user interface that is unmatched in terms of ease-of-use, productivity and capability.

Codes and specifications

Structures must be designed and constructed according to the provisions of a code, which is a legal document containing requirements related to such things as structural safety, fire safety, plumbing, ventilation, and accessibility to the physically disabled. The American Concrete Institute (ACI) has published the ACI Building Code Requirements for Reinforced Concrete, which is usually referred to as the "ACI Code". This is widely used as a legal set of rules by which RCC buildings are designed. Bangladesh has also published Bangladesh National Building code following "ACI Code". If a structural designer correctly follows this set of rules and the building, which the designer has designed, has structural problems or failures, then the designer cannot be balm.

2.6: Slab Design:

2.6.1 Slab: A slab is a structural element, made of concrete that is used to create flat horizontal surfaces such as floors, roof decks and ceilings. A slab is generally several inches thick and supported by beams, columns, walls, or the ground. There are two types of slabs described as follow:

1. One-way slab
2. Two-way slab

2.6.1a One-way slab:

When a slab is supported on all four sides and the ratio of long span to short span is equal or greater than two, it will be considered as one way slab. The load on the slab is carried by the short span in one direction. However main reinforcement bar and distribution bar in transverse direction.

Longer span (l)/Shorter span (b) ≥ 2

2.6.1b Minimum Slab Thickness:

To control deflection, ACI Code 9.5.2.1 specifies minimum thickness values for one-way solid slabs.

Element	Simply supported	One end continuous	Both end continuous	Cantilever
One-way solid slabs	l/20	l/24	l/28	l/10

2.6.1c Span:

According to ACI code 8.7.1 If the slab rests freely on its supports, the span length may be taken as equal to the clear span plus the depth of the slab but need not exceed the distance between centers of supports.

2.6.1d Bar Spacing:

The lateral spacing of the flexural bars should not exceed 3 times the thickness h or 18 inch According to ACI code 7.6.5 the lateral spacing of temperature and shrinkage reinforcement should not be placed farther apart than 5 times the slab thickness or 18 inch according to ACI code 7.12.2

2.6.1e Maximum Reinforcement Ratio:

Reinforcement ratio is the ratio of reinforcement area to gross concrete area based on total depth of slab. One-way solid slabs are designed as rectangular sections subjected to shear and moment. Thus, the maximum reinforcement ratio corresponds to a net tensile strain in the reinforcement, €t of 0.004

2.6.1f Minimum Reinforcement Ratio:

For temperature and shrinkage reinforcement:
According to ACI Code 7.12.2.1
Slabs with Grade 40 or 50 deformed bars –> 0.0020
Slabs with Grade 60 deformed bars –> 0.0018

2.6.2.g Two-way slab:

The slab is said to be spanning in two directions, when the length to breadth ratio falls below 2. The deflection and bending moments in a two-way slab are considerably reduced as compared to those in one- way slab. In a square slab, the two-way action is equal in each direction. In such case, main reinforcement is provided in both directions in the form of a mesh.

CHAPTER 3

METHODOLOGY

3.1:General

A thesis paper will never be complete without methodology part. This chapter gives the outline of the procedure which will followed to complete the study. In this chapter, ten storied commercial building has been considered to study the behavior of sudden variation of structural component.

3.2:Working flowchart

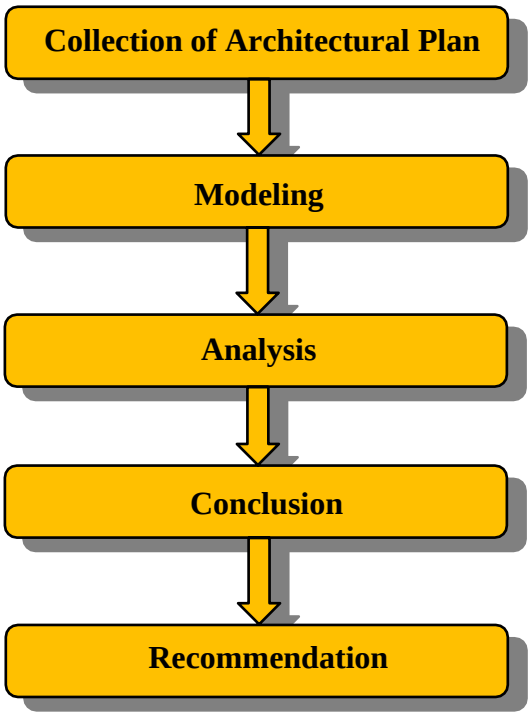


Figure 3.1 Working Flowchart

(reference:Bungale, S.T.(1988). "Structure Analysis & Design of Tall Buildings".McGrawHillBook Co:-Singapore, P.P. 8-11, 45-46, 65-78

3.3:Method:Ultimate Strength Design (USD)

The design of reinforced concrete structure can be performed by either of two alternative methods. The first of these direct attention to stress conditions within the structural memberunder working loads and is known working stress design. The second focus on the strength capacity of conditioned corresponding to failure and is known as ultimate strength design.

In addition concrete is used in composite design. Composite design is used for any structuralmember, such as beams or columns, when the member contains combination of concrete andshapes.

1. Members are proportioned so that the full strength of the cross section is just utilizedwhen the ultimate loads is applied.
2. USD methods permit and accurate appraisal of strength capacities of member and sopermit and evaluation of actual factor of safety of against failure.
3. USD methods include adequate safeguards to ensure that conditions at working loads.

("Building code Requirements for Reinforced Concrete" ACI publication 318-63,American Concrete Institute, Farmington Hills, MI, 1963.)

CHAPTER-4

DESIGN SPECIFICATION AND ANALYSIS OF BUILDING FRAME BY ETABS

4.1:General

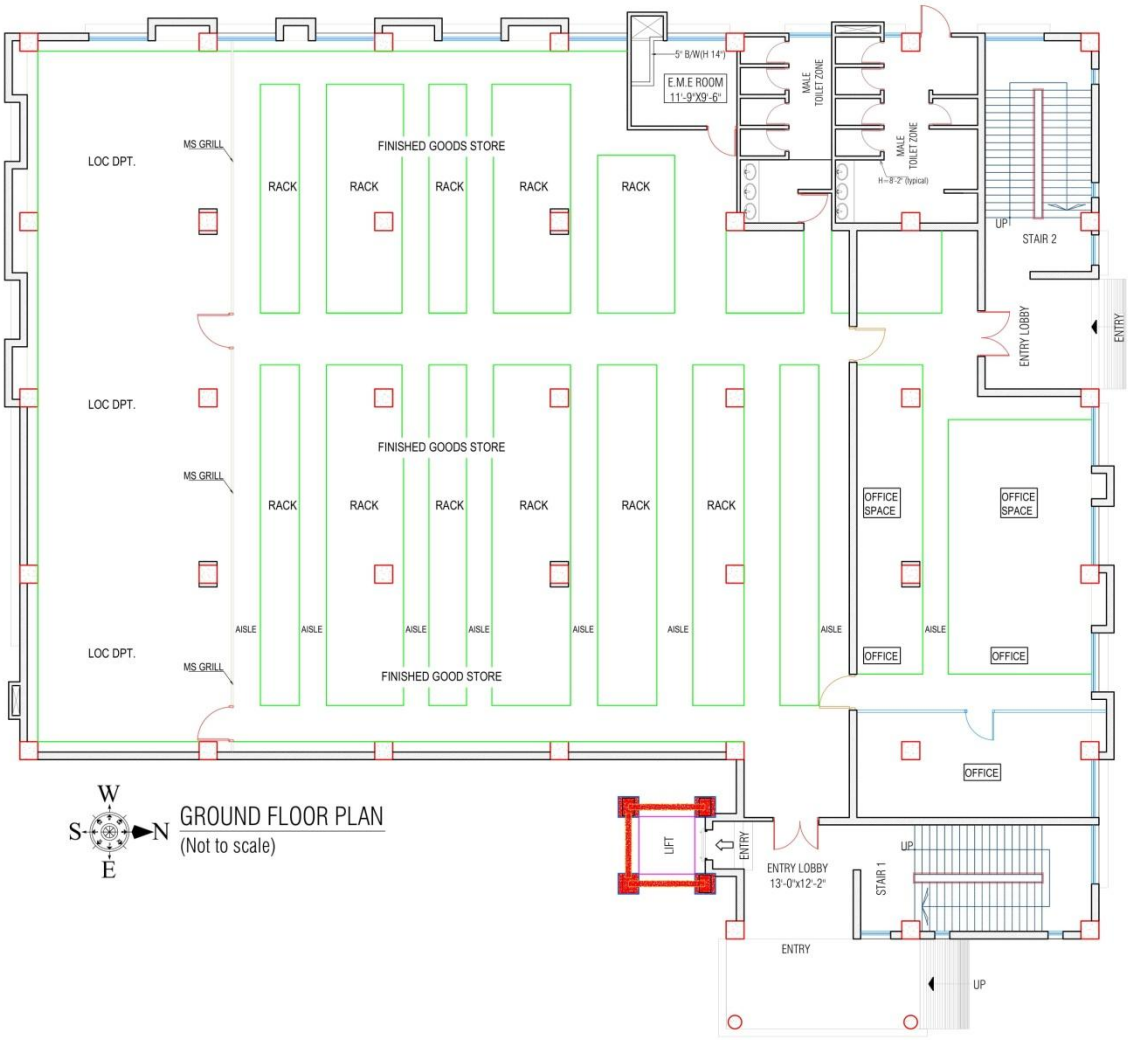


Figure 4.1.1: Ground Floor Plan

This chapter includes an architectural plan for Industrial Building, beam and column layout, load combinations, model development and finally analyzes the model for lateral load such as earthquake and wind load. In short it can be said that, this chapter carried out a review of the methodology adopted for this study.

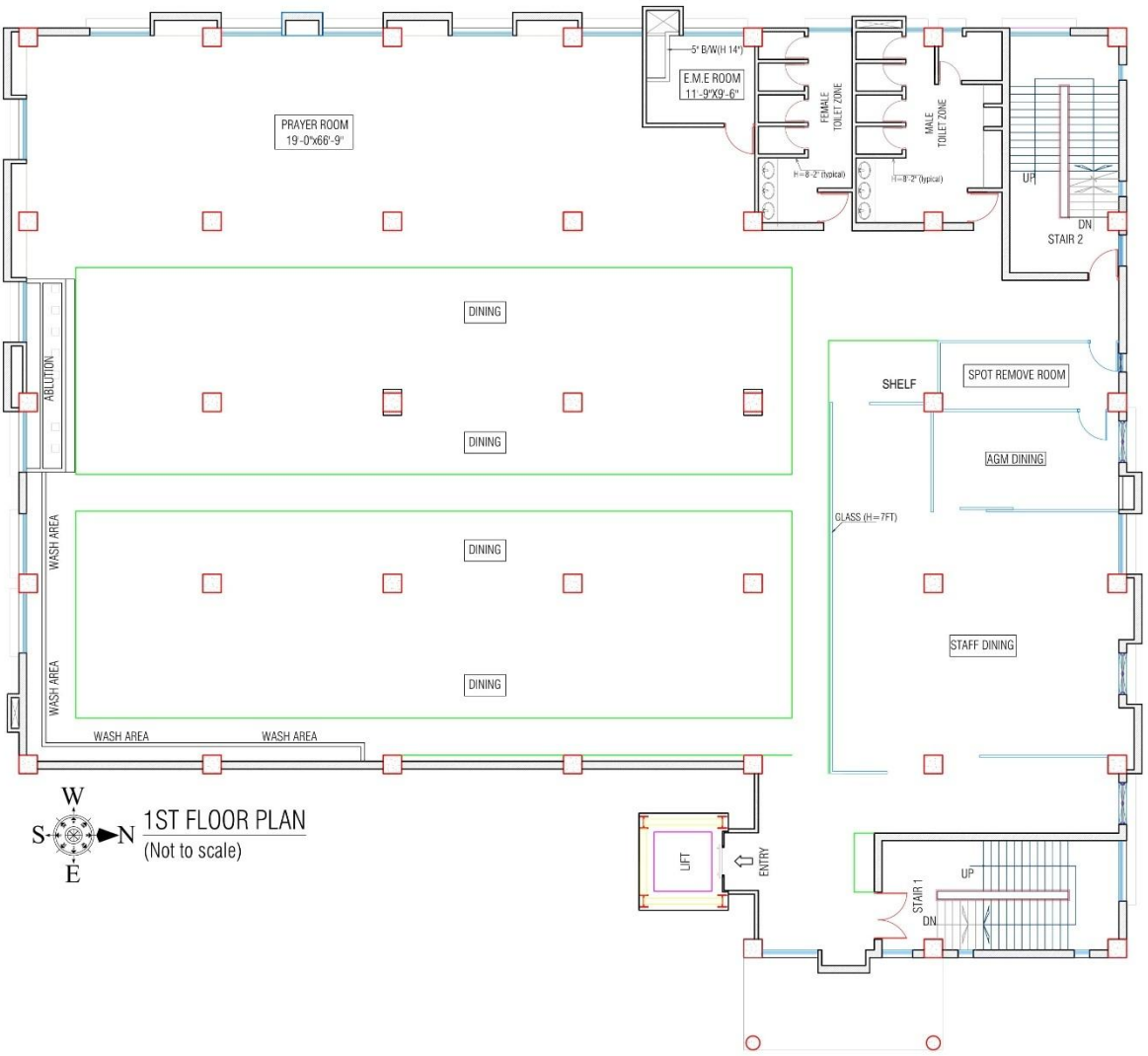


Figure 4.1.2:1ST Floor Plan

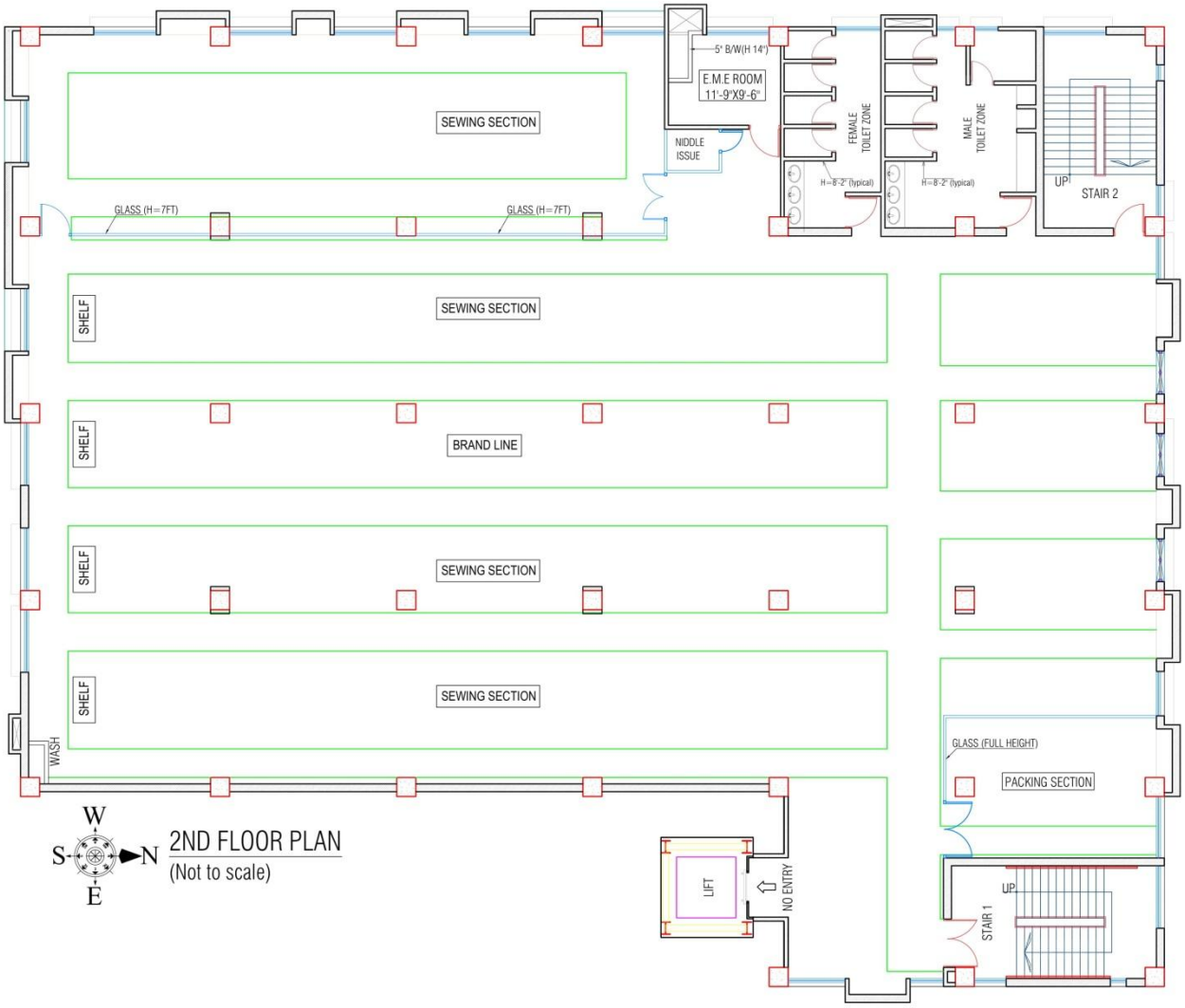


Figure 4.1.3:2nd Floor Plan

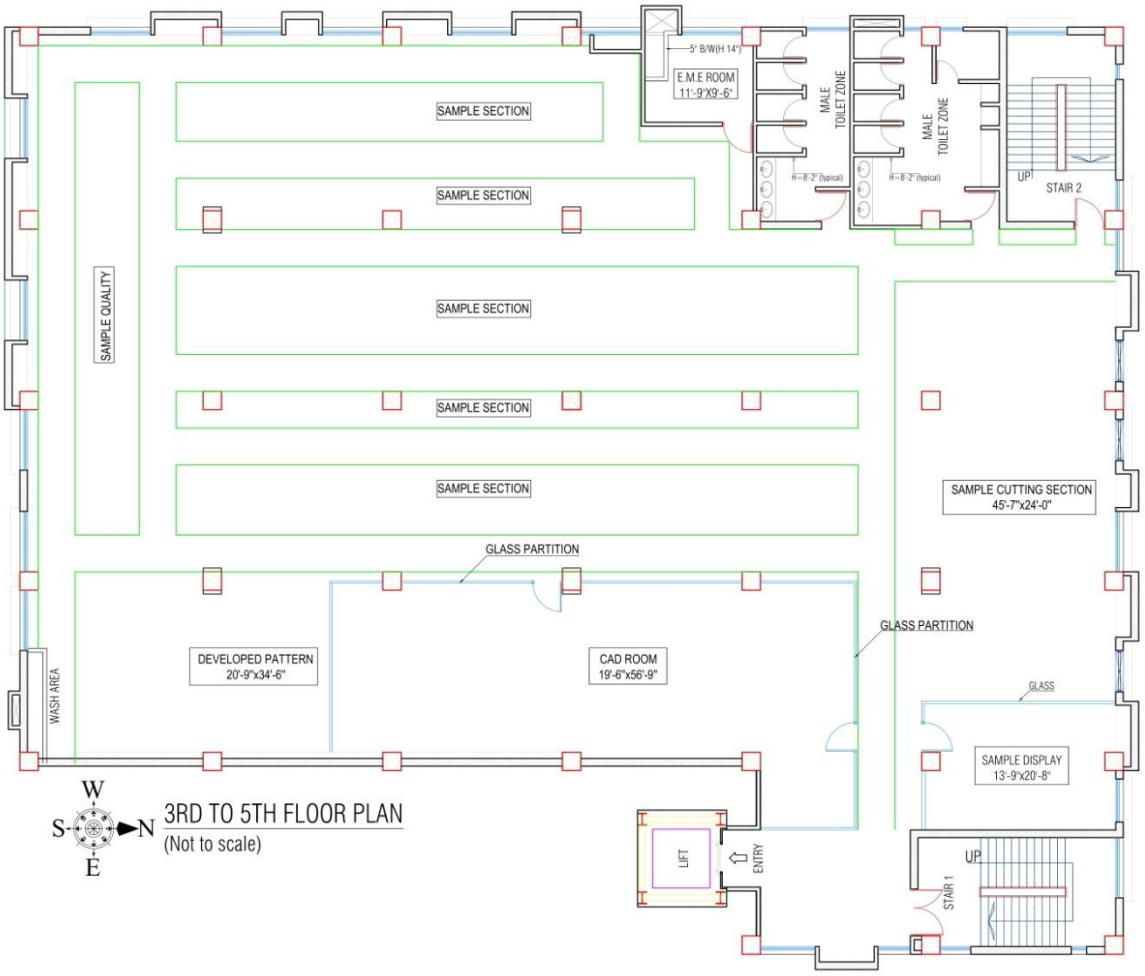


Figure 4.1.4:3RD To 5th Floor Plan

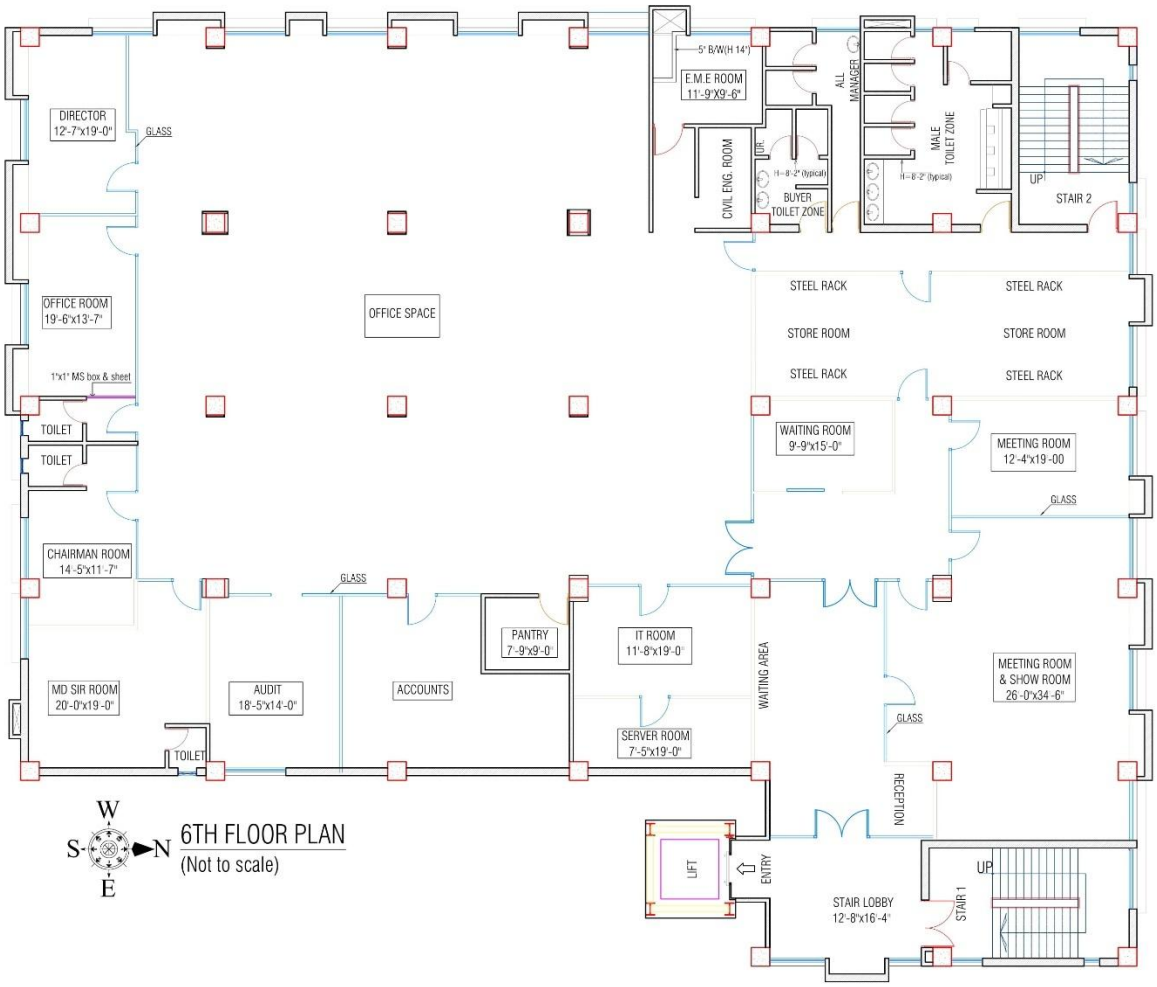


Figure 4.1.5:6th Floor Plan

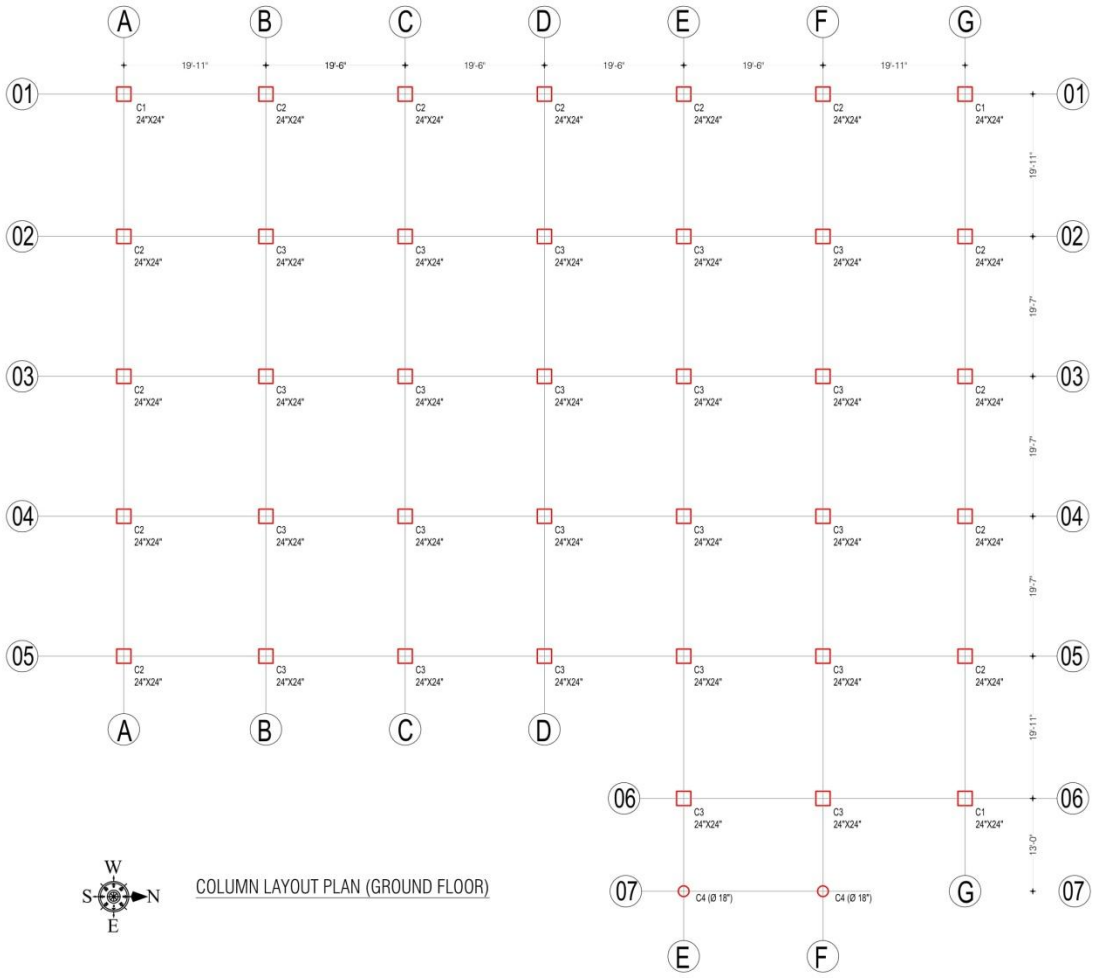


Figure 4.1.6:Column Layout Plan (Ground Floor)

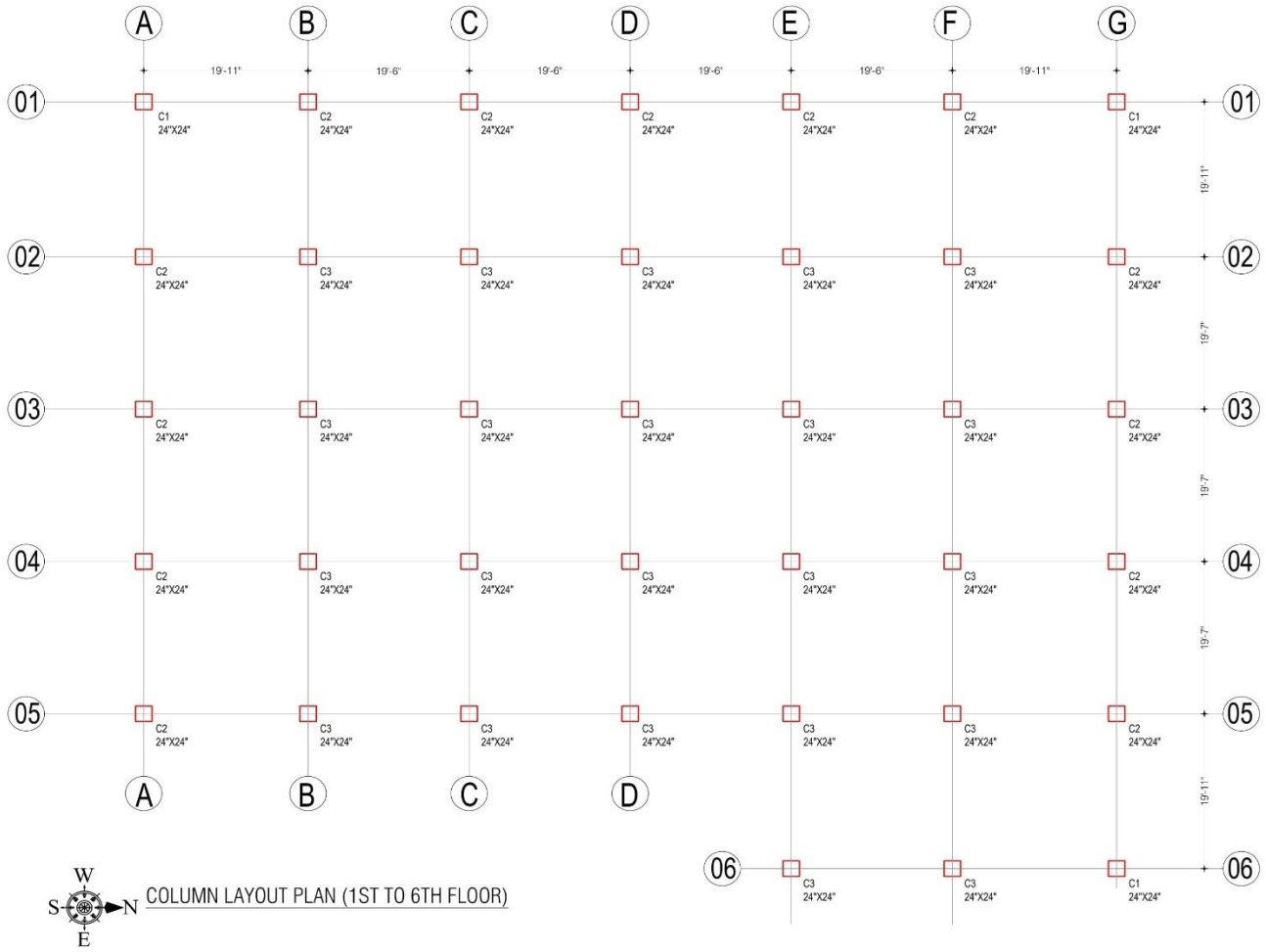


Figure 4.1.7:Column Layout Plan (1st to 6th Floor)

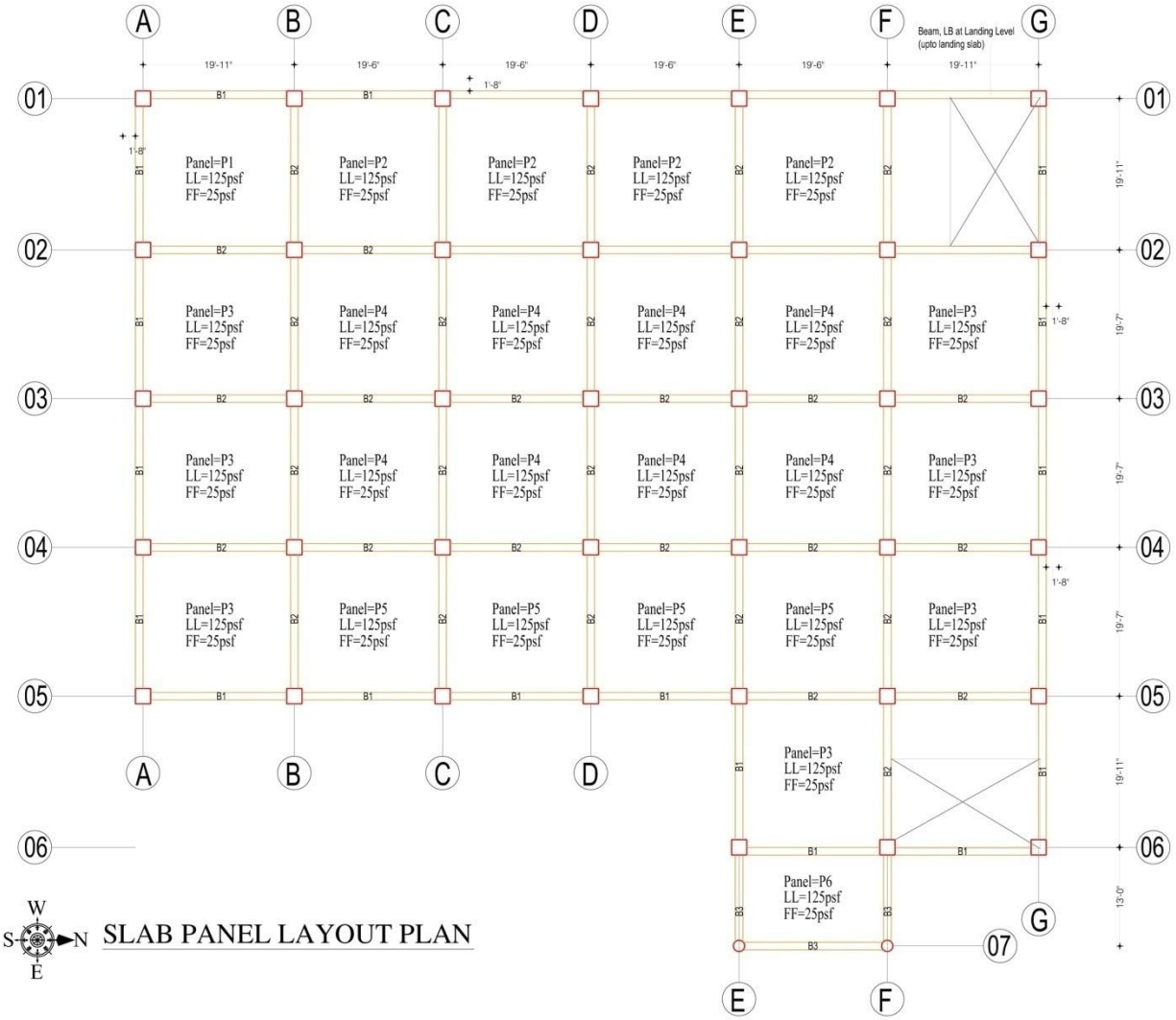


Figure 4.1.8: Slab Panel Layout Plan

3D view from ETABS.

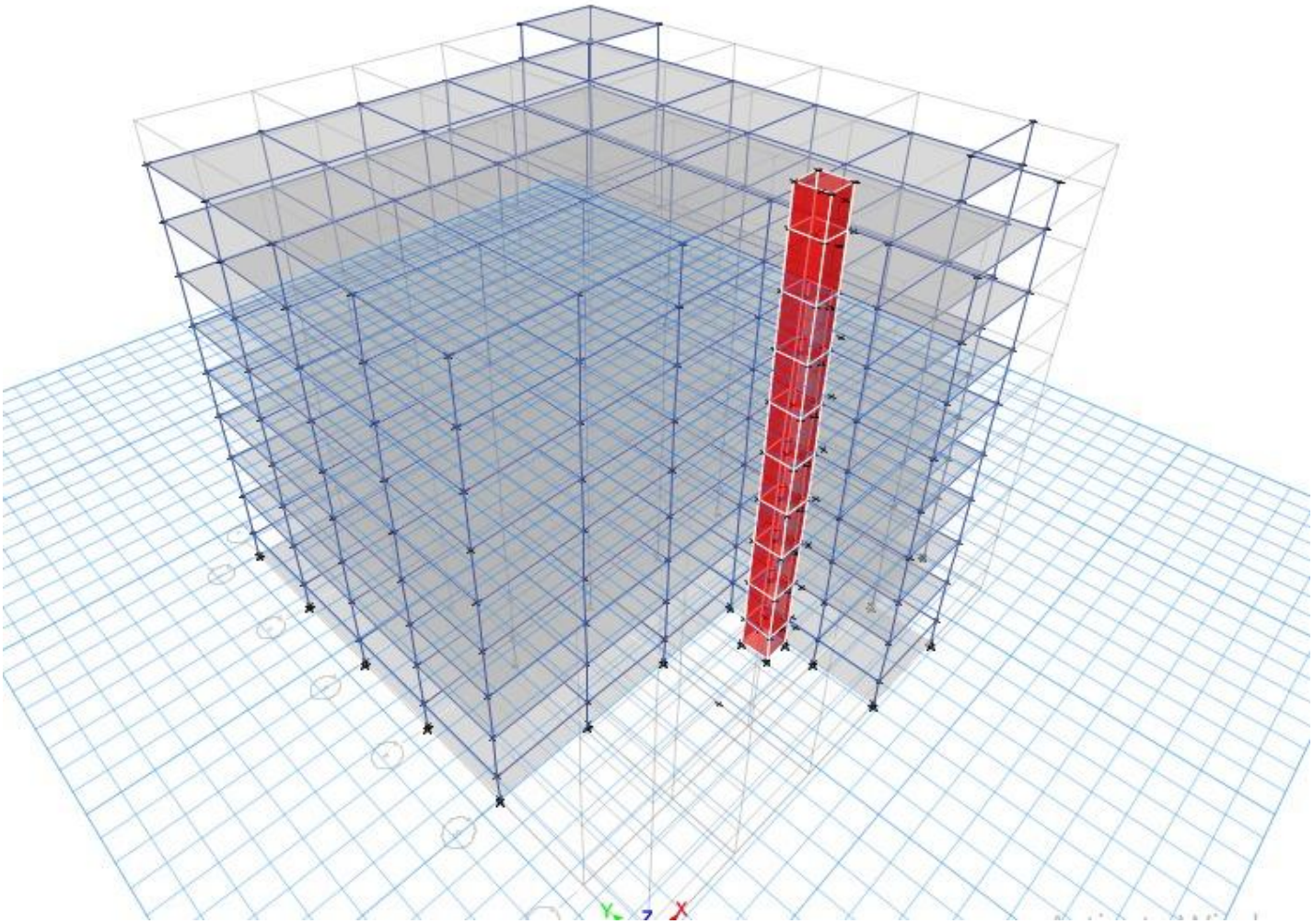


Figure 4.1.9: 3D view from ETABS.

4.2.Architectural plan

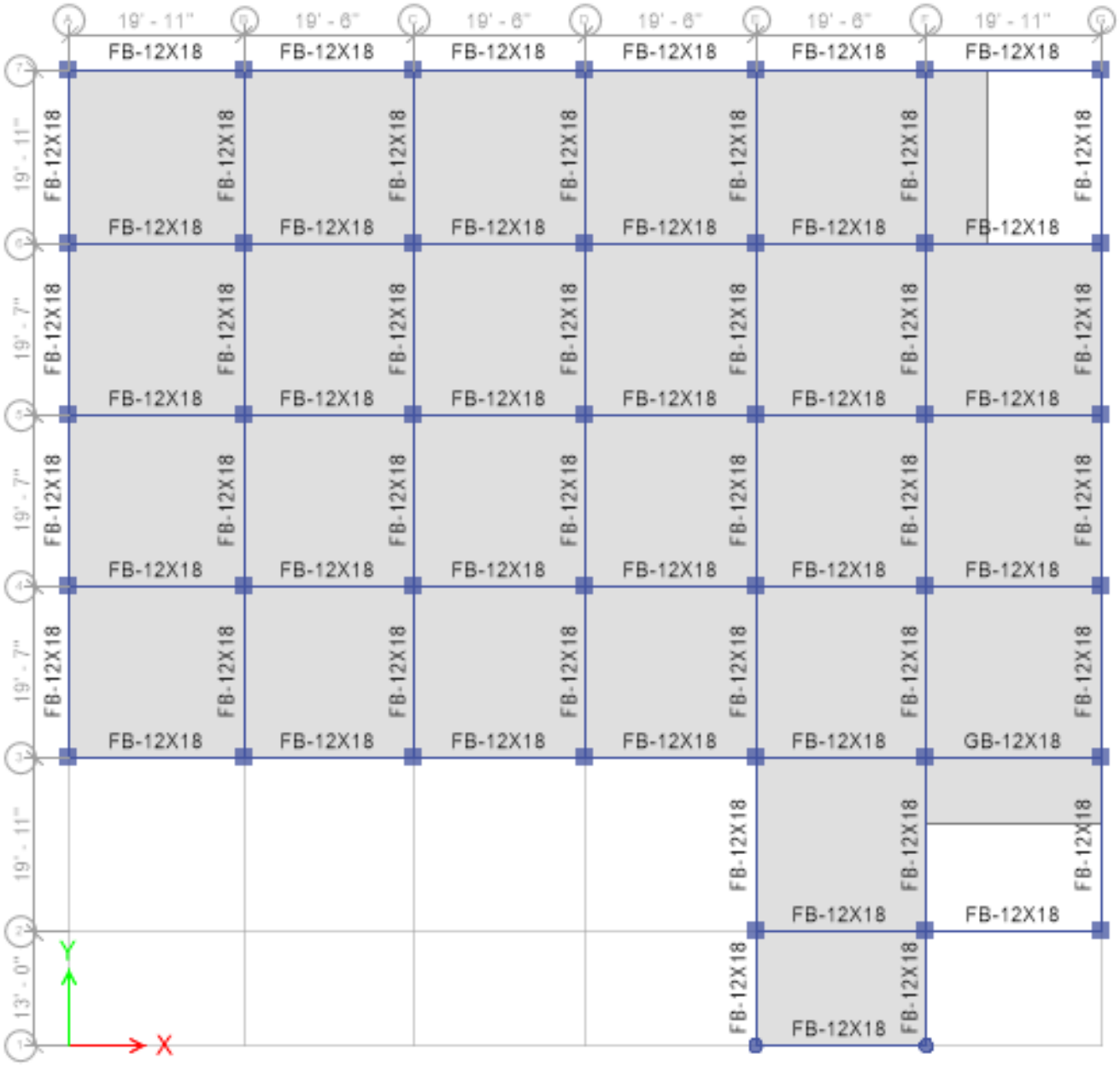
Building type: Seven Storied Factory Building.
Zone: Dhaka. Building Area: 121'-8" X 110'-8"

Design data consideration & specification

The whole study was carried out based on few considerations & specifications that are given in below:

Table-4. 1 Summary of the design considerations & specification of the study:

Type of structure	**Factory Building**
Items	Description
Design method	Ultimate Strength Design (USD).
Design procedure	ACI Moment Co-efficient Method.
Analysis procedure	Extended Three-dimensional Analysis of Building System (ETABS).
Design code	American Concrete Institute (ACI 318-08) American Society Of Civil Engineers (ASCE 7-05) Bangladesh National Building Code (BNBC 2020)
Material properties	Yield strength of steel, f_y = 60 ksi. Compressive strength of concrete, $f_{c,}$ = 4 ksi. For Column Compressive strength of concrete, f_c' = 4 ksi for Beam and Slab. Unit weight of concrete = 150 psf. Unit weight of brick = 120 psf
Member sectional properties	Slab type = Two –way. Column type = Square/Rectangular Beam type = Square/Rectangular Thickness of all walls = 10 inch. Toilet Partition wall 5in. Position of grade beam 6 ft. from column base level
Loading condition	Live load = 125psf (As Per Architectural Plan) Live load = 100 psf (for stair slab) Floor finish = 25 psf Partition Wall= On Beam line



Beam & Column Layout

2:Axial Force of Column (SMF):

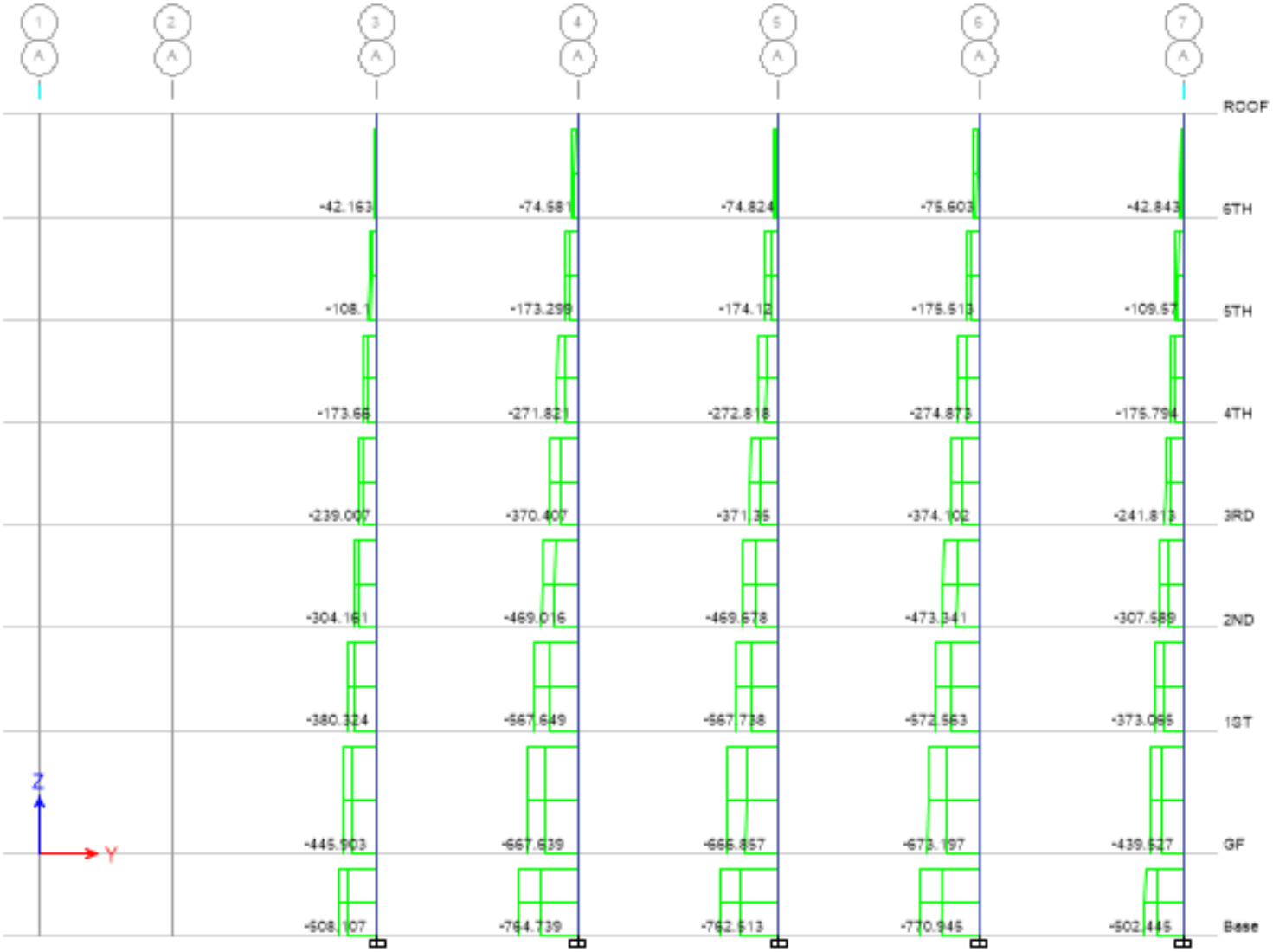


Figure 4.2.1: Axial Force of A-1234567

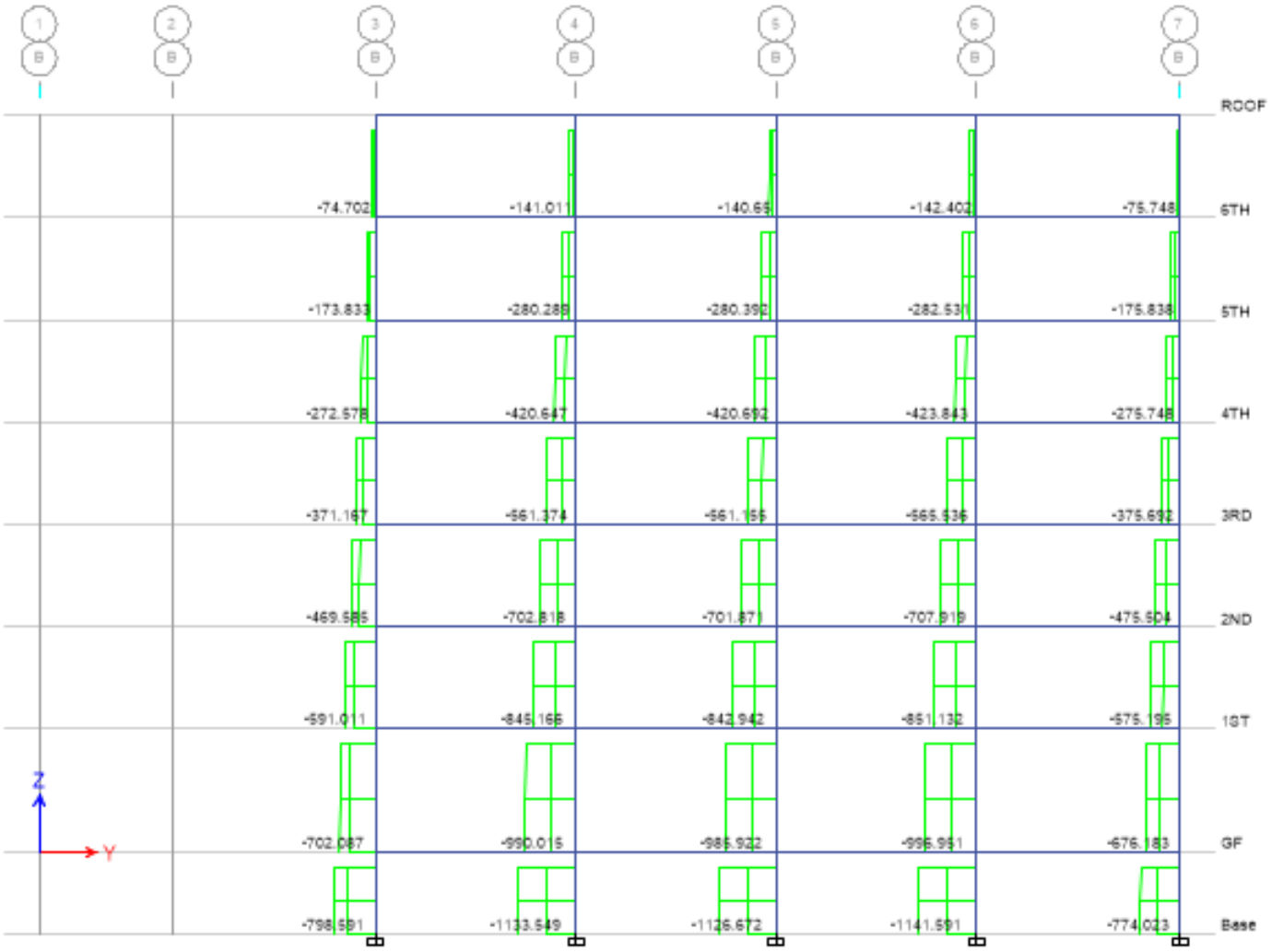


Figure 4.2.2: Axial Force of B-1234567

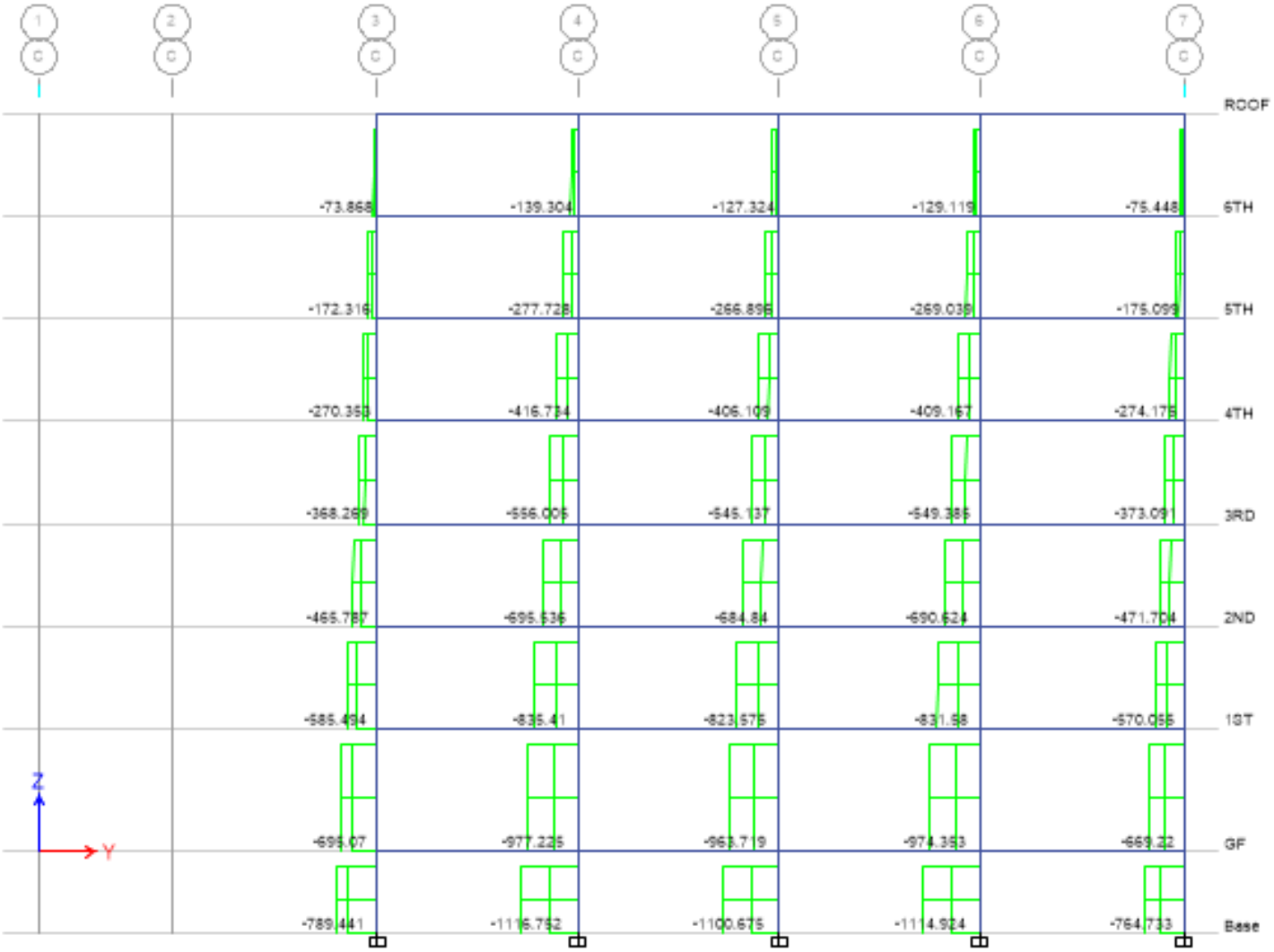


Figure 4.2.3: Axial force for grid C – 1234567

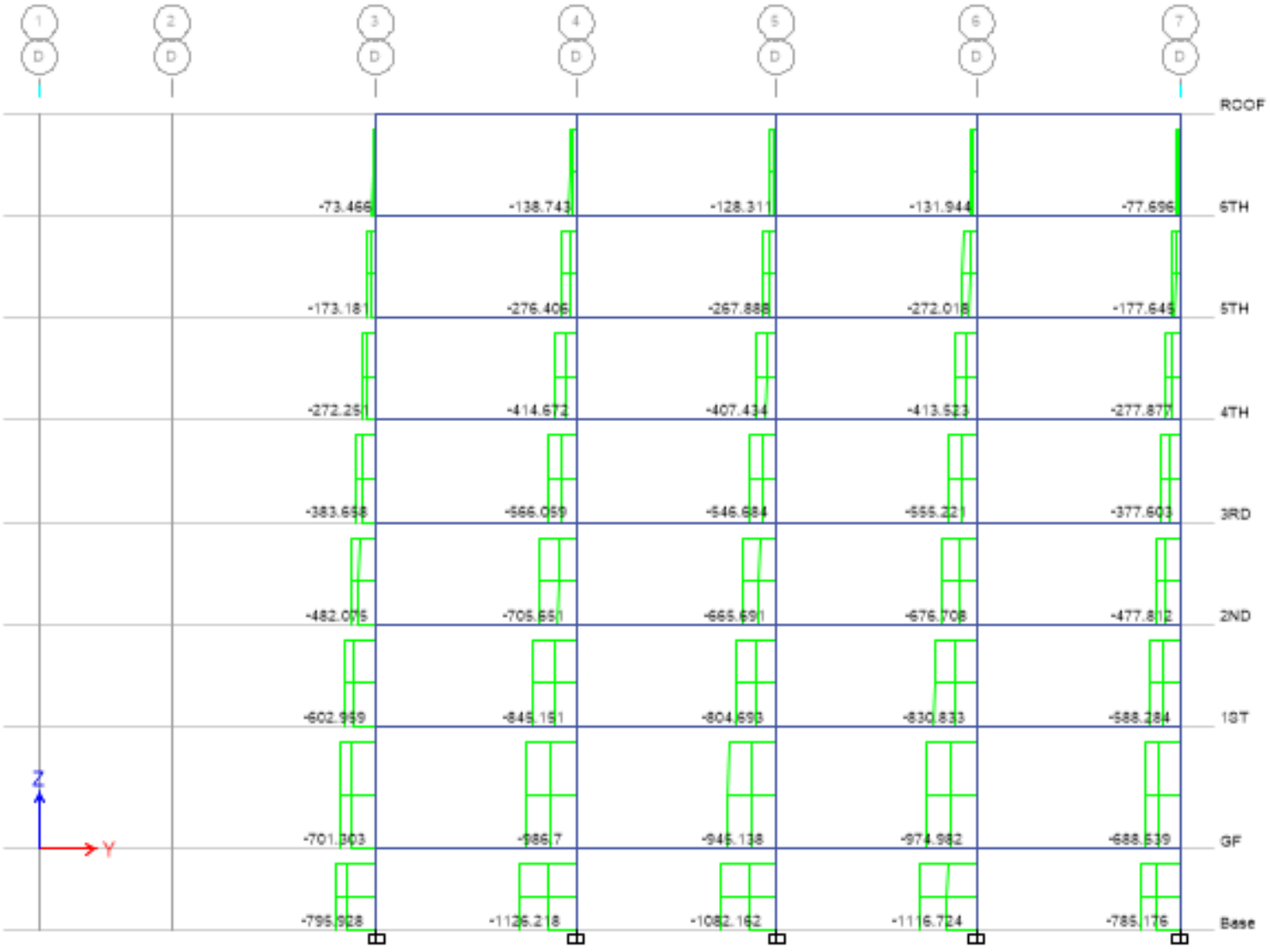


Figure 4.2.4: Axial force for grid D – 1234567

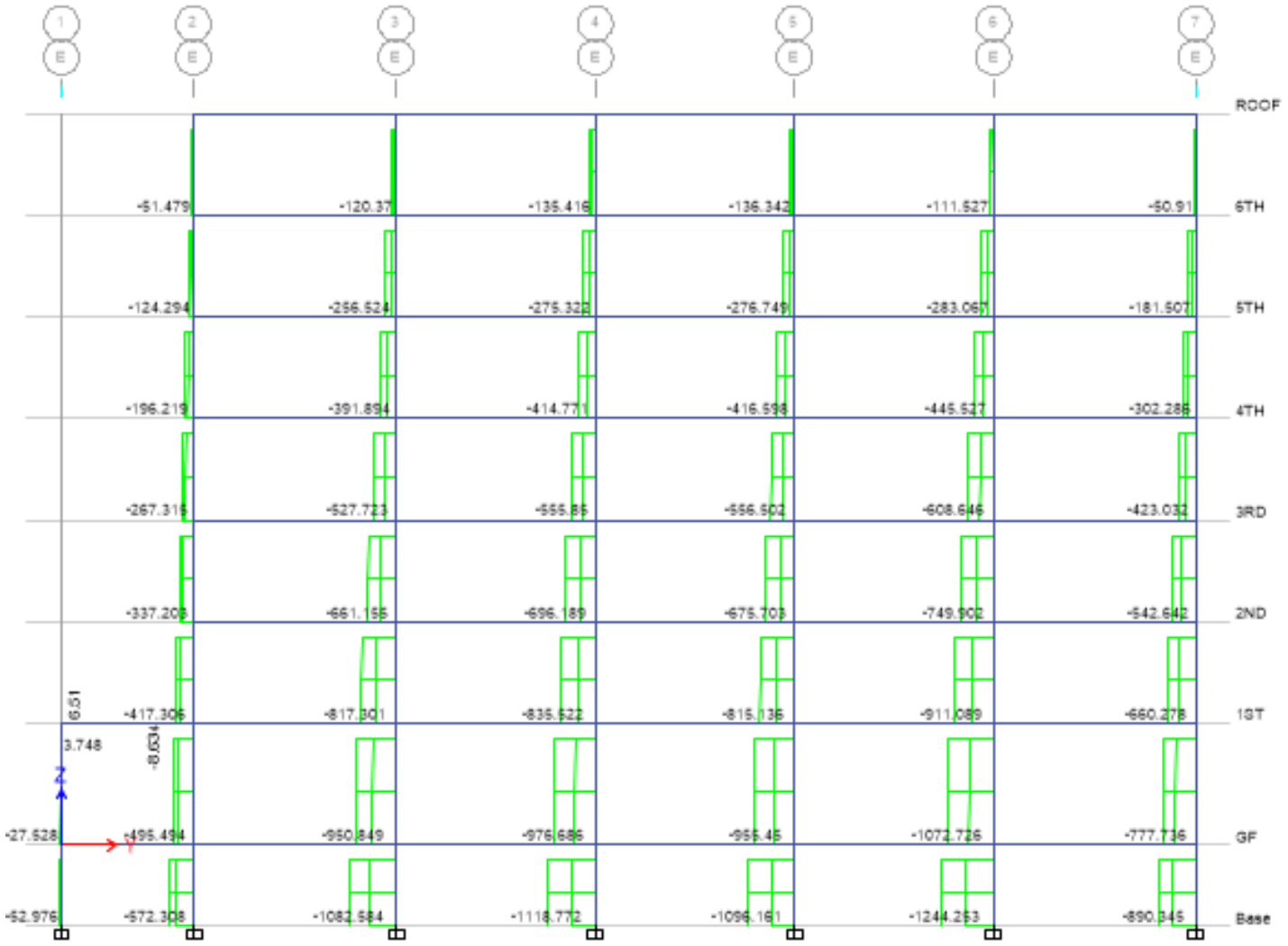


Figure 4.2.5: Axial force for grid E – 1234567

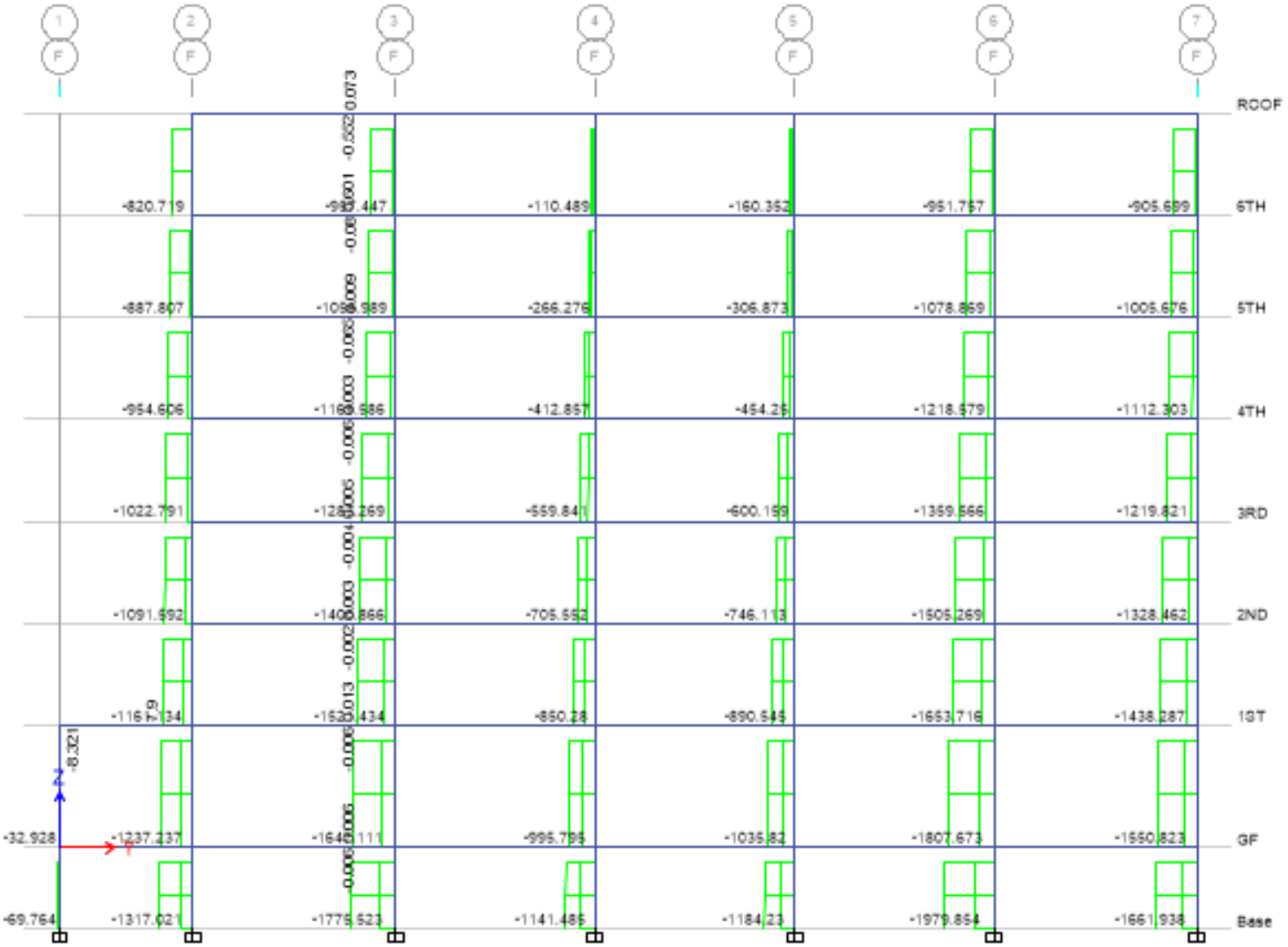


Figure 4.2.6: Axial force for grid F – 1234567

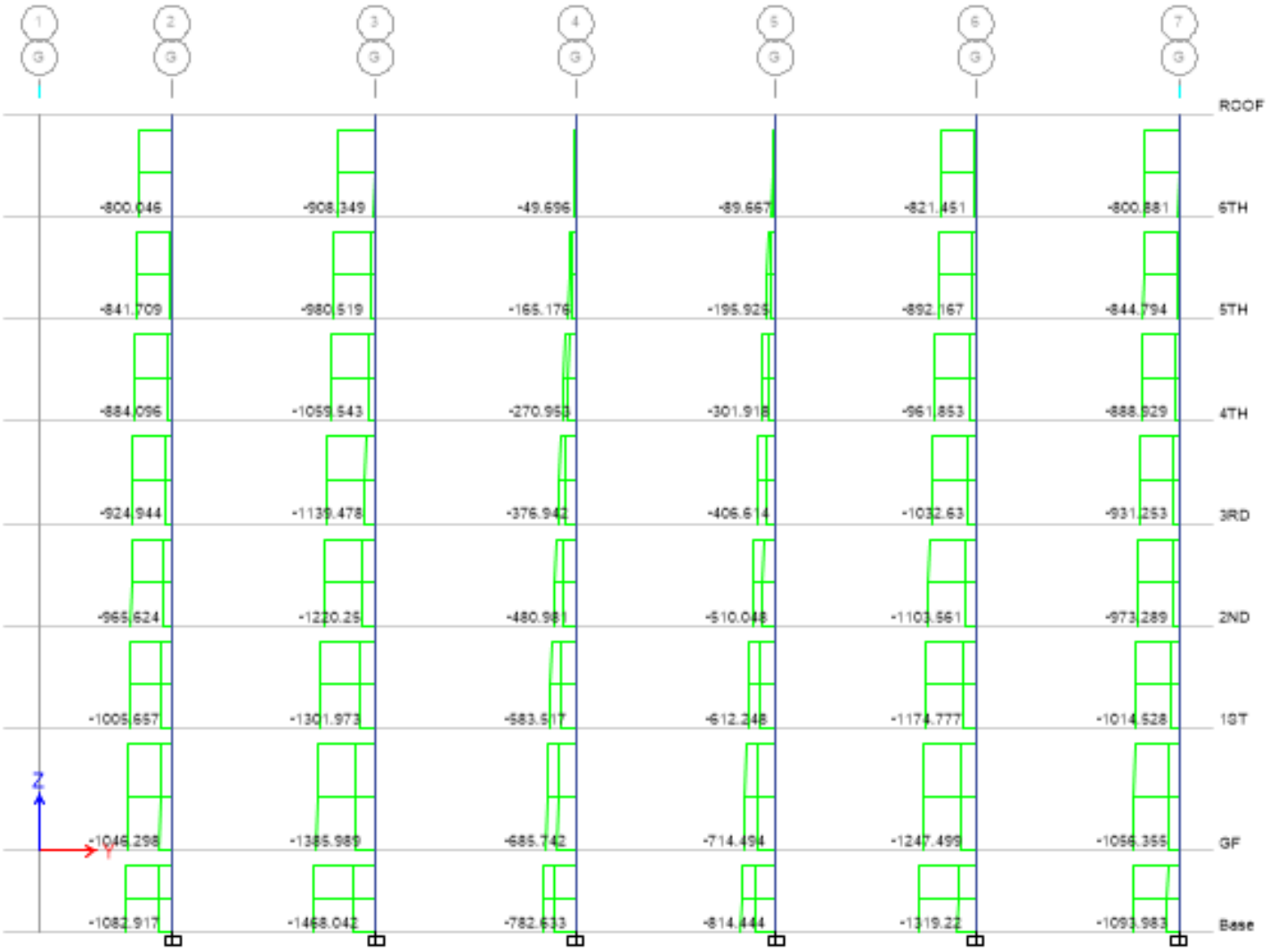


Figure 4.2.6: Axial force for grid G – 1234567

3.Banding moment diagram of beam:

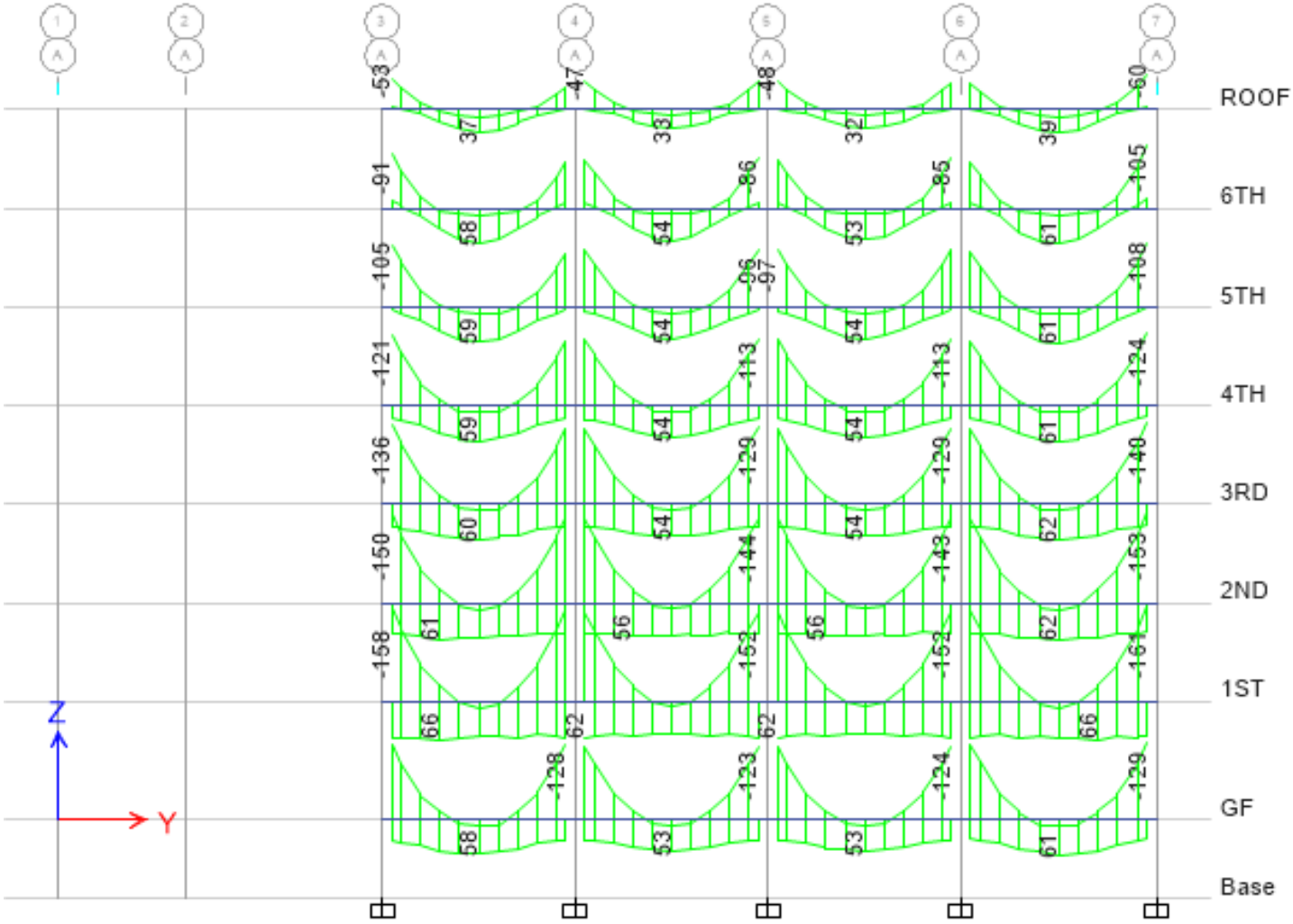


Figure 4.3.1: Bending Moment Diagram at grid A – 1234567

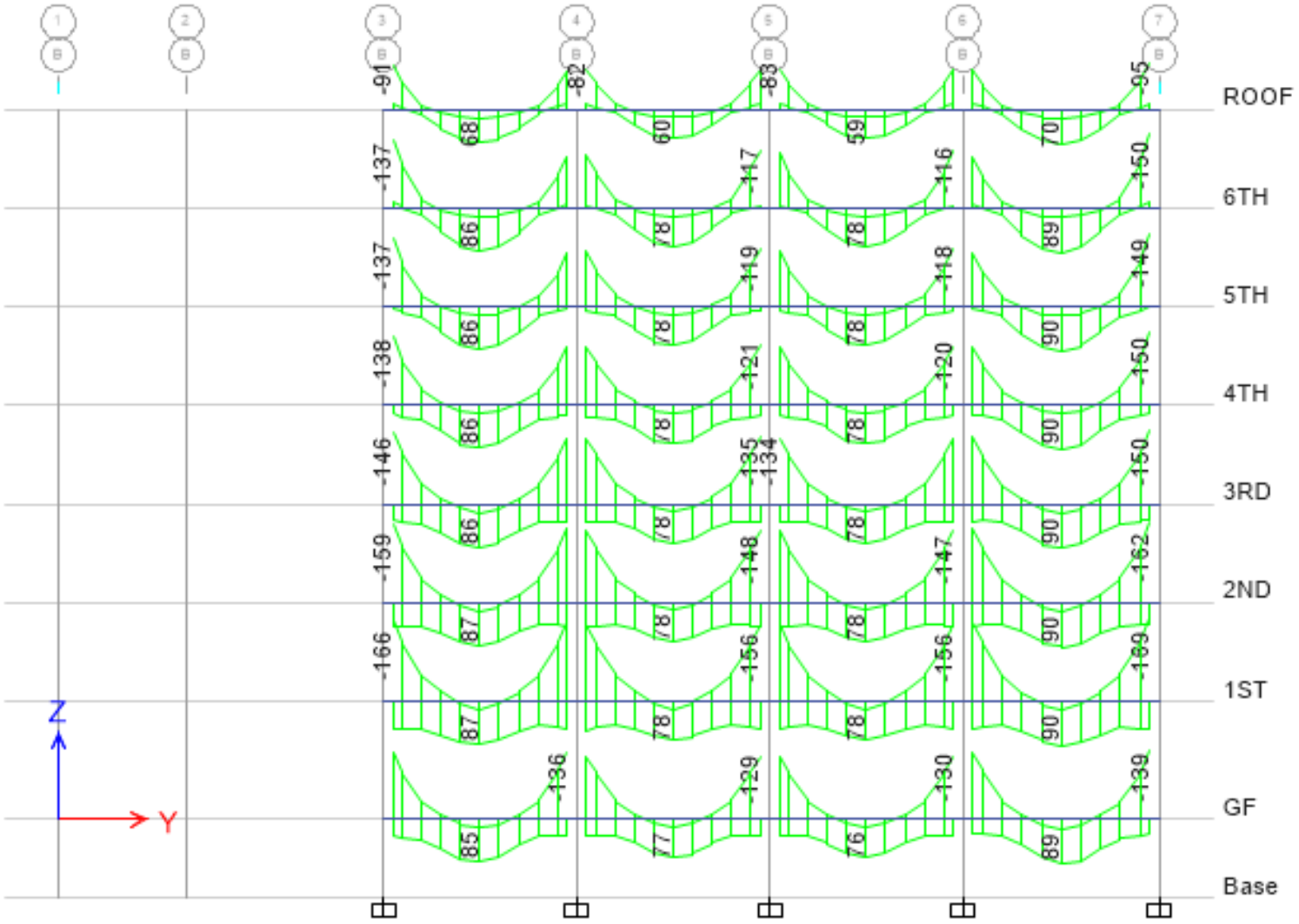


Figure 4.3.2: Bending Moment Diagram at grid B – 1234567

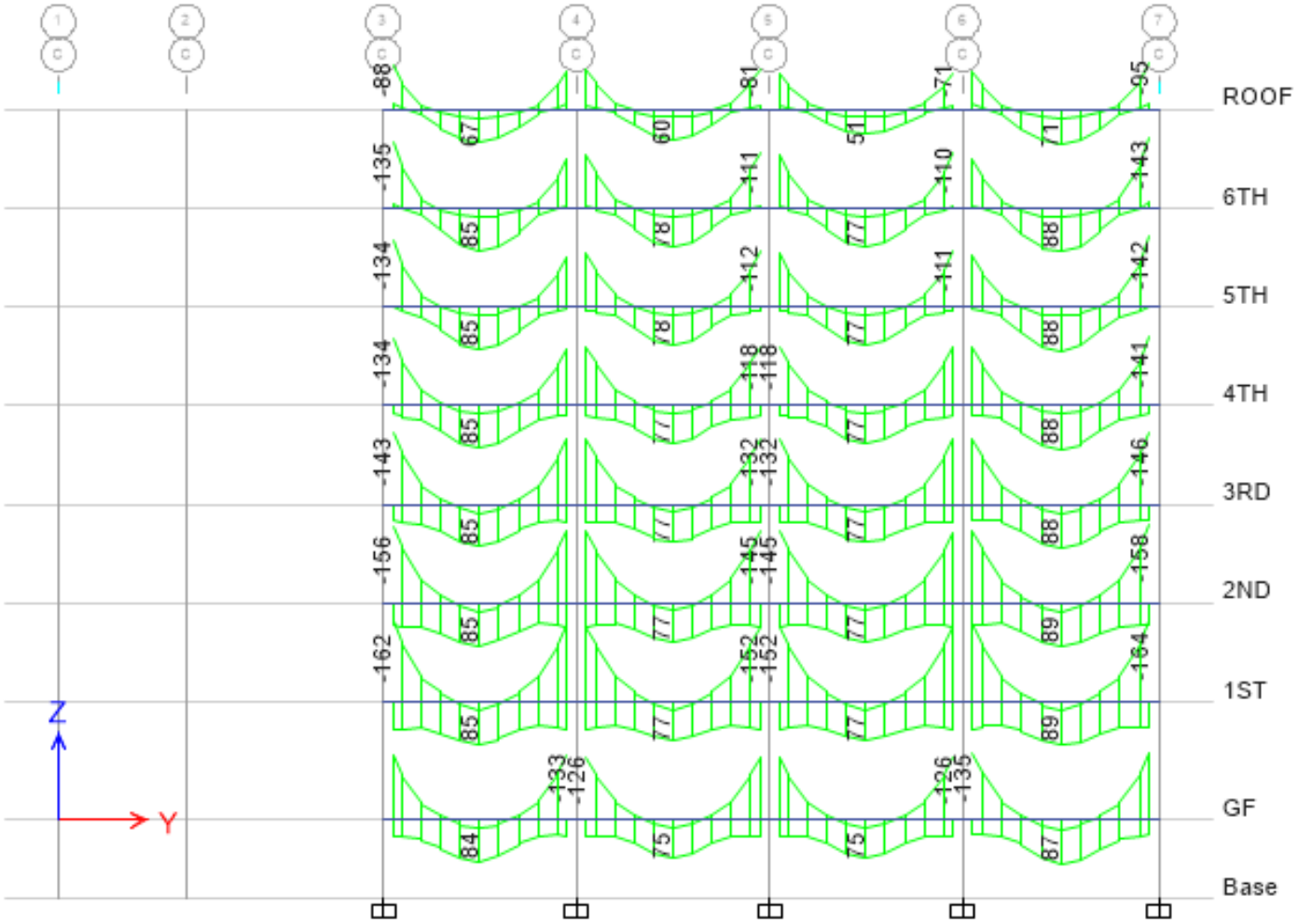


Figure 4.3.3: Bending Moment Diagram at grid C – 1234567

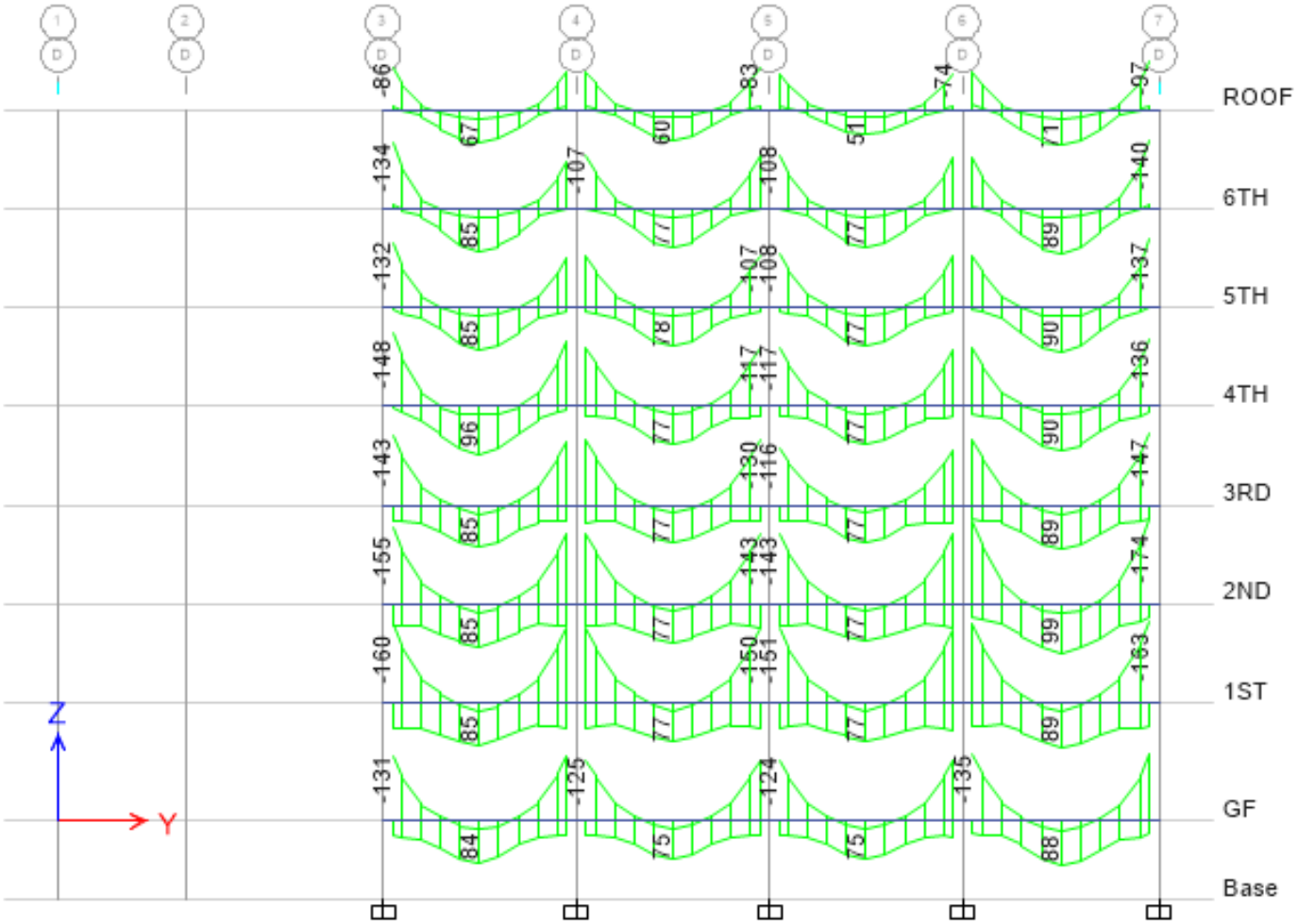


Figure 4.3.4: Bending Moment Diagram at grid D – 1234567

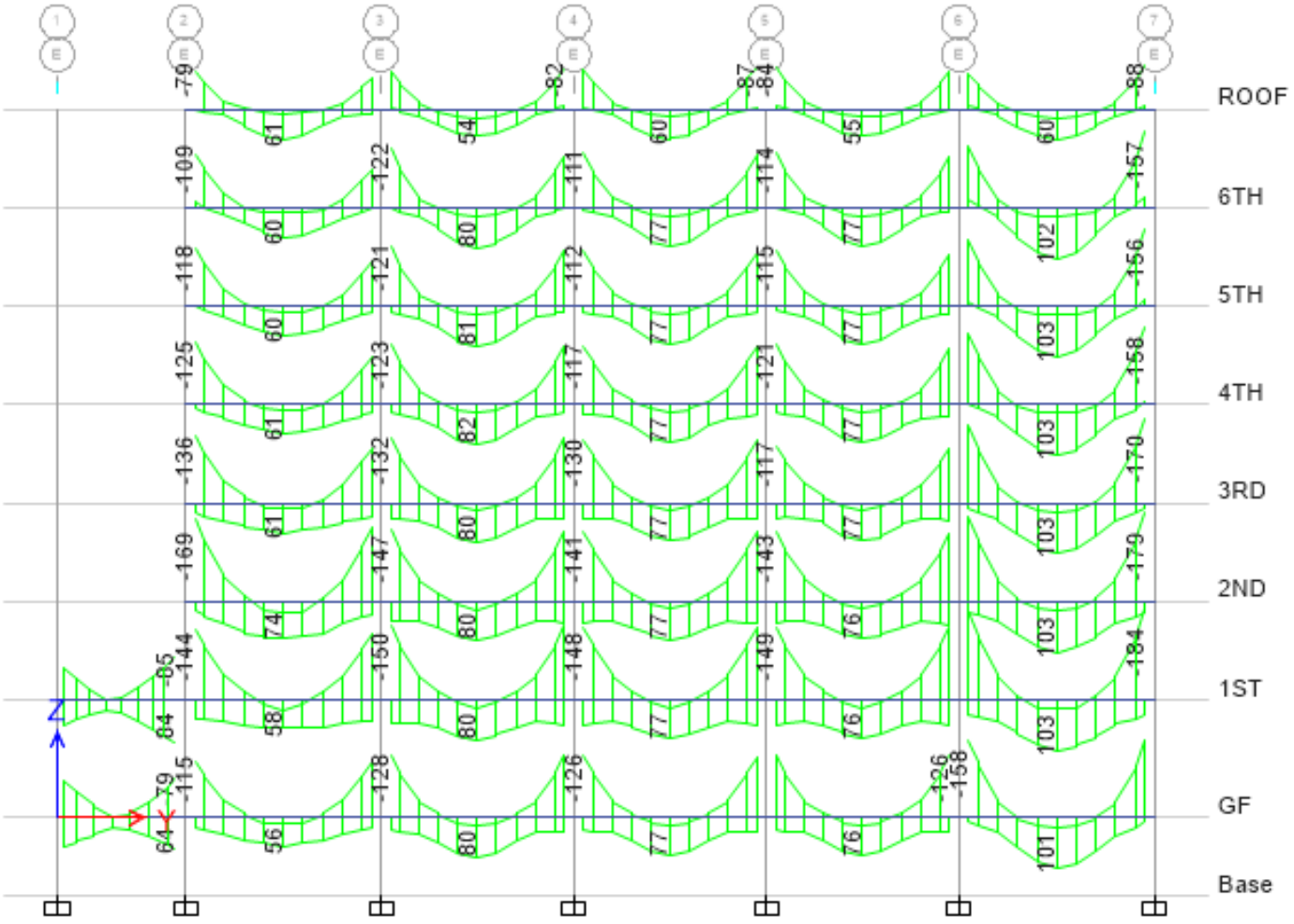


Figure 4.3.5: Bending Moment Diagram at grid E – 1234567

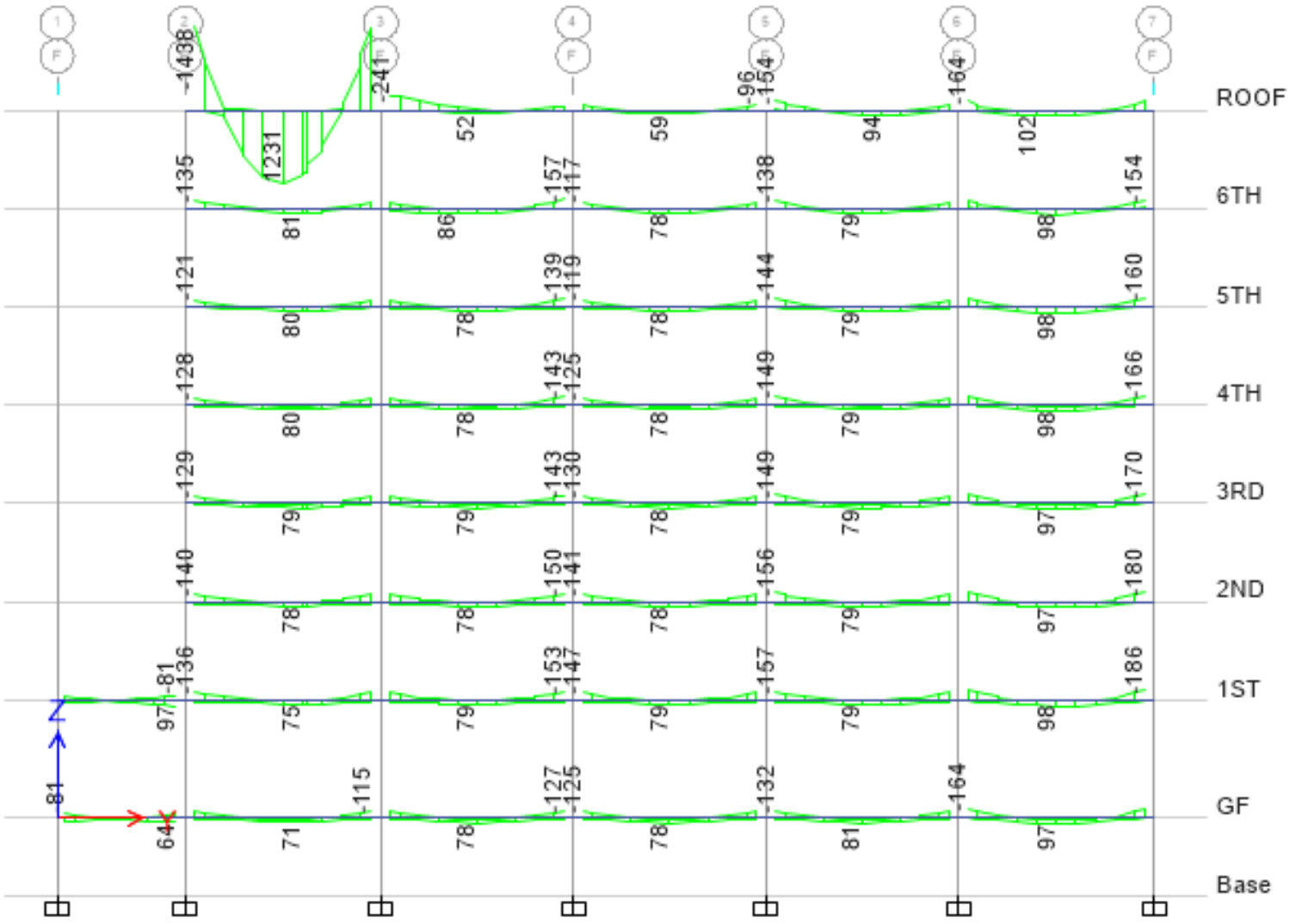


Figure 4.3.6: Bending Moment Diagram at grid F – 1234567

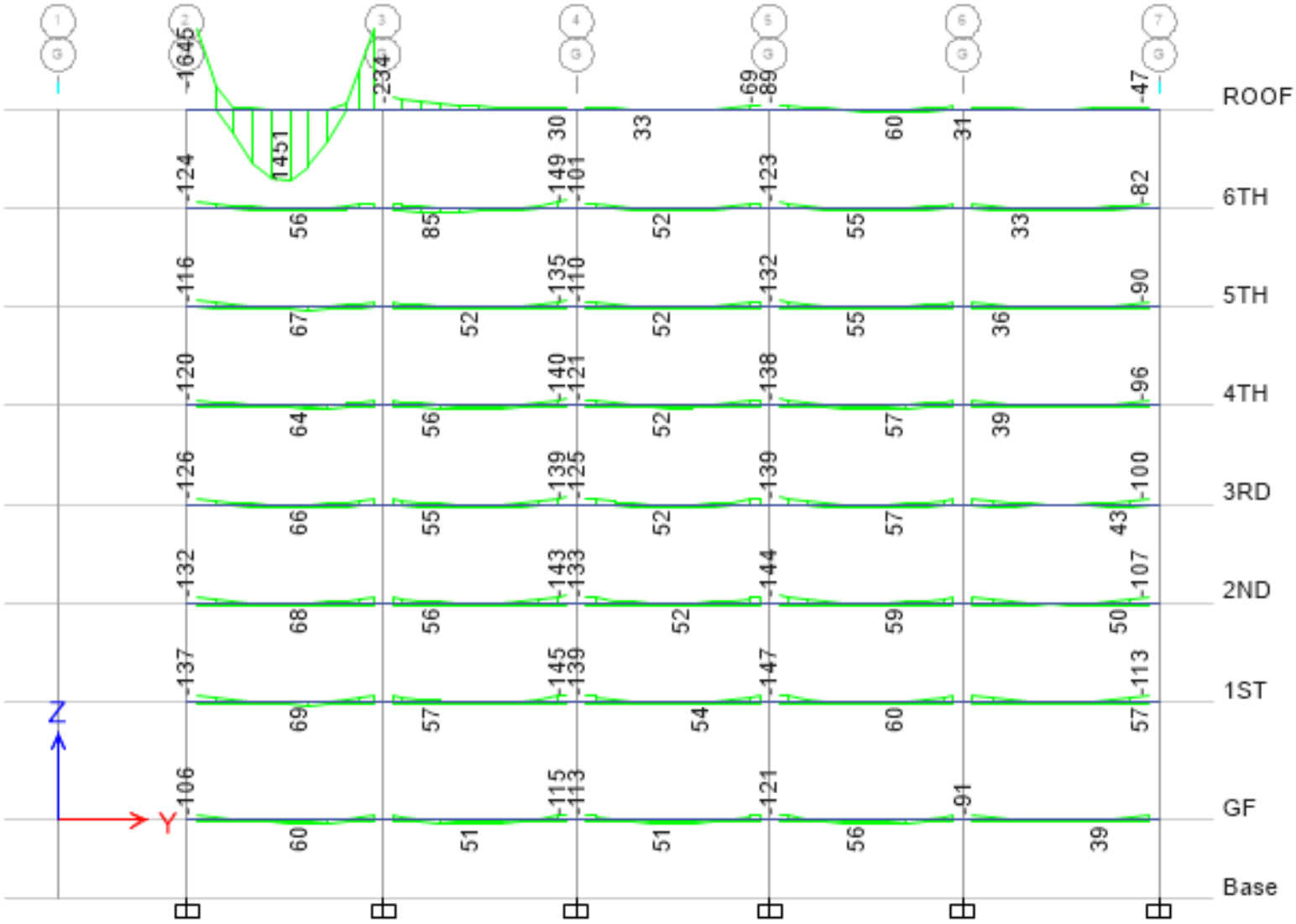


Figure 4.3.7: Bending Moment Diagram at grid G – 1234567

CHAPTER-5

STRUCTURAL DESIGN AND CONCLUSION AND RECOMANDATION

5.1:Slab Design

Floor slab Panel: 01

Yield strength of steel *fy = 60 ksi*
Compressive strength of concrete, *f'c =3.5 ksi*
Live load *=125 psf*
Floor finish = *25 psf*
Two Interior edge continuous:

$$Ratio, ß = \frac{18.92}{18.92} = 1. < 2$$

So, two way slab analysis is required.

$$Slab\ Thickness = \frac{\ln (0.8 + \frac{fy}{200})}{36 + 9ß}$$

$$T = \frac{18.92 X 12 X (0.8 X 60000 / 20000)}{36 + 9 X 1}$$

$$= 5.54$$

$$= 6"$$

Ln = Clear span in long direction.

Load Calculation:

Self Weight of the slab = 6/12X0.15=0.075

DL=SW+FF =0.075+0.025 =0.1 Ksf

LL= 125 psf

Factor load, q = 1.2X0.1+1.6X0.125 ==0.32 ksf

Find out moment co-efficient:

$$m = \frac{Short\ Clear\ Span\ (l_a)}{Long\ Clear\ Span\ (l_b)} = \frac{18.92}{18.92} = 1$$

So, Select Case no-4

Table:5.1:Bending moment coefficients for rectangular panles(referencwe: Parker, H. (1986). "Simplified Design of Reinforced Concrete" (3rd Ed.) M.S.Seawall for Wiley Eastern Limited, New Delhi, p.p. 142-145.)

$C_{a.neg}$	$C_{b.neg}$	$C_{a.dl}$	$C_{a.ll}$	$C_{b.dl}$	$C_{b.ll}$
0.050	*0.050*	*0.027*	*0.032*	*0.027*	*0.032*

Moment calculation:

Negative moment:

$Ma\ neg = CanegWul^2a$

$= 0.050 \text{ X}0.32\text{X}(18.92)^2$

=5.72 k'

Positive moment:

$Ma\ pos = C_{adl}W_{udl}\, l^2_a + Ca_{ll}\, W_{ll}l^2a$

$= 0.027 \text{ X } 0.12\text{X } (18.92)^2+0.032\text{X}0.2\text{X}(18.92)^2$

= 1.15 + 2.29

= 3.44 k- ft

Floor slab Panel: 02

Find out moment co-efficient:

$$m = \frac{Short\ Clear\ Span\ (l_a)}{Long\ Clear\ Span\ (l_b)} = \frac{18.5}{18.92} = 0.97$$

So, Select Case no-8

Table5.2:Bending moment coefficients for rectabgular panles((referencwe: Parker, H. (1986). "Simplified Design of Reinforced Concrete" (3rd Ed.) M.S.Seawall for Wiley Eastern Limited, New Delhi, p.p. 142-145.)

$C_{a.neg}$	$C_{b.neg}$	$C_{a.dl}$	$C_{a.ll}$	$C_{b.dl}$	$C_{b.ll}$
0.033	*0.061*	*0.020*	*0.028*	*0.023*	*0.030*

Moment calculation:

Negative moment:

$M_{a\,neg} = C_{aneg} W_u l^2_a$

$=0.033X0.32X(18.5)^2$

$= 3.61$ k-ft

$M_{b\,neg} = C_{bneg} W_u l^2_a$

$= 0.061X0.32X(18.92)^2$

$= 6.98$ k-ft

Positive moment:

$M_{a\,pos} = C_{adI} W_{dI}\, l^2_a + Ca_{ll}\, W_{ll} l^2 a$

$=0.020X0.12X(18.5)^2+0.020X0.2X(18.5)^2$

$= 0.82+1.43$

$= 2.25$ k-ft

$M_{b\,pos} = C_{bdl} W_{dl} l^2_b + C_{bll} W_{ll} l^2_b$

$=0.023X0.12X(18.92)^2+0.030X0.2(18.92)$

$=3.13$ k-ft

Floor slab Panel: 03

Find out moment co-efficient:

$$m = \frac{Short\ Clear\ Span\ (l_a)}{Long\ Clear\ Span\ (l_b)} = \frac{18.58}{18.92} = 0.98$$

So, Select Case no-9

Table5.3:Bending moment coefficients for rectabgular panles((referencwe: Parker, H. (1986). "Simplified Design of Reinforced Concrete" (3rd Ed.) M.S.Seawall for Wiley Eastern Limited, New Delhi, p.p. 142-145.)

$C_{a.neg}$	$C_{b.neg}$	$C_{a.dl}$	$C_{a.ll}$	$C_{b.dl}$	$C_{b.ll}$
0.061	*0.033*	*0.023*	*0.030*	*0.020*	*0.028*

Moment calculation:

Negative moment:

$M_{a\ neg} = C_{aneg} W_u l^2_a$

$=0.061 \text{ X } 0.32 \text{ X } (18.5)^2$

$= 6.73$ k-ft

$M_{b\ neg} = C_{bneg} W_u l^2_a$

$= 0.033 \text{ X } 0.32 \text{ X} (18.92)^2$

$= 3.78$ k-ft

Positive moment:

$M_{a\ pos} = C_{adI} W_{dI}\ l^2_a + Ca_{ll}\ W_{ll} l^2 a$

$=0.023 \text{ X } 0.12 \text{ X } (18.5)^2 + 0.030 \text{ X } 0.2 \text{ X } (18.5)^2$

$= 0.95 + 2.07$

$= 3.02$ k-ft

$M_{b\ pos} = C_{bdl} W_{dl} l^2_b + C_{bll} W_{ll} l^2_b$

$=0.020 \text{ X } 0.12 \text{ X } (18.92)^2 + 0.028 \text{ X } 0.2 \text{ x } (18.92)^2$

$=2.85$ k-ft

Floor slab Panel: 04

Find out moment co-efficient:

$$\text{m} = \frac{Short\ Clear\ Span\ (l_a)}{Long\ Clear\ Span\ (l_b)} = \frac{18.5}{18.58} = 0.99$$

So, Select Case no-2

Table:5.4:Bending moment coefficients for rectabgular panles((referencwe: Parker, H. (1986). "Simplified Design of Reinforced Concrete" (3rd Ed.) M.S.Seawall for Wiley Eastern Limited, New Delhi, p.p. 142-145.)

$C_{a.neg}$	$C_{b.neg}$	$C_{a.dl}$	$C_{a.ll}$	$C_{b.dl}$	$C_{b.ll}$
0.045	*0.045*	*0.018*	*0.027*	*0.018*	*0.027*

Moment calculation:

Negative moment:

$M_{a\,neg} = C_{aneg} W_u l^2_a$

$=0.045 \text{ X } 0.32 \text{ X } (18.5)^2$

$= 4.92$ k-ft

$M_{b\,neg} = C_{bneg} W_u l^2_a$

$= 0.045 \text{ X } 0.32 \text{ X} (18.58)^2$

$= 4.97$ k-ft

Positive moment:

$M_{a\,pos} = C_{adl} W_{dl}\, l^2_a + Ca_{ll}\, W_{ll} l^2 a$

$=0.018 \text{ X } 0.12 \text{ X } (18.5)^2 + 0.027 \text{ X } 0.2 \text{ X } (18.5)^2$

$= 0.73 + 1.84$

$= 2.57$ k-ft

$M_{b\,pos} = C_{bdl} W_{dl} l^2_b + C_{bll} W_{ll} l^2_b$

$=0.018 \text{ X } 0.12 \text{ X } (18.58)^2 + 0.027 \text{ X } 0.2 \text{ x } (18.58)^2$

$=2.6$ k-ft

Floor slab Panel: 05

Find out moment co-efficient:

$$m = \frac{Short\ Clear\ Span\ (l_a)}{Long\ Clear\ Span\ (l_b)} = \frac{18.5}{18.58} = 0.99$$

So, Select Case no-8

Table:5.5:Bending moment coefficients for rectabgular panles((referencwe: Parker, H. (1986). "Simplified Design of Reinforced Concrete" (3rd Ed.) M.S.Seawall for Wiley Eastern Limited, New Delhi, p.p. 142-145.)

$C_{a.neg}$	$C_{b.neg}$	$C_{a.dl}$	$C_{a.ll}$	$C_{b.dl}$	$C_{b.ll}$
0.033	*0.061*	*0.020*	*0.028*	*0.023*	*0.030*

Moment calculation:

Negative moment:

$M_{a\,neg} = C_{aneg} W_u l^2_a$

$=0.033 \text{ X } 0.32 \text{ X } (18.5)^2$

$= 3.61$ k-ft

$M_{b\,neg} = C_{bneg} W_u l^2_a$

$= 0.061 \text{ X } 0.32 \text{ X} (18.58)^2$

$= 6.73$ k-ft

Positive moment:

$M_{a\,pos} = C_{adl} W_{dl}\, l^2_a + Ca_{ll}\, W_{ll} l^2 a$

$=0.020 \text{ X } 0.12 \text{ X } (18.5)^2 + 0.028 \text{ X } 0.2 \text{ X } (18.5)^2$

$= 0.82 + 1.93$

$= 2.57$ k-ft

$M_{b\,pos} = C_{bdl} W_{dl} l^2_b + C_{bll} W_{ll} l^2_b$

$=0.023 \text{ X } 0.12 \text{ X } (18.58)^2 + 0.030 \text{ X } 0.2 \text{ x } (18.58)^2$

$=3.02$ k-ft

Floor slab Panel: 06

Find out moment co-efficient:

$$m = \frac{Short\ Clear\ Span\ (l_a)}{Long\ Clear\ Span\ (l_b)} = \frac{12}{18.5}$$

$=0.64$

So, Select Case no-6

Table:5.6:Bending moment coefficients for rectabgular panles((referencwe: Parker, H. (1986). "Simplified Design of Reinforced Concrete" (3rd Ed.) M.S.Seawall for Wiley Eastern Limited, New Delhi, p.p. 142-145.)

$C_{a.neg}$	$C_{b.neg}$	$C_{a.dl}$	$C_{a.ll}$	$C_{b.dl}$	$C_{b.ll}$
0.091	*0*	*0.054*	*0.064*	*0.007*	*0.010*

Moment calculation:

Negative moment:

$M_{a\,neg} = C_{aneg} W_u l^2_a$

$=0.091 \text{ X } 0.32 \text{ X } (12)^2$

$= 4.19$ k-ft

$M_{b\,neg} = C_{bneg} W_u l^2_a$

$= 0.061 \text{ X } 0.32 \text{ X} (18.58)^2$

$= 6.73$ k-ft

Positive moment:

$M_{a\,pos} = C_{adI} W_{dI}\, l^2_a + Ca_{ll}\, W_{ll} l^2 a$

$=0.054 \text{ X } 0.12 \text{ X } (12)^2 + 0.064 \text{ X } 0.2 \text{ X } (12)^2$

$= 0.93 + 1.84$

$= 2.77$ k-ft

$M_{b\,pos} = C_{bdl} W_{dl} l^2_b + C_{bll} W_{ll} l^2_b$

$=0.054 \text{ X } 0.12 \text{ X } (18.5)^2 + 0.010 \text{ X } 0.2 \text{ x } (18.5)^2$

$=2.89$ k-ft

Panel NO	(+VE) Moment (k-ft)	(-VE) Moment (k-ft)
P1	MA=3.44	MA=5.72
P2	MA=2.25, MB=3.31	MA=3.61,MB=6.98
P3	MA=3.02, MB=2.85	MA=6.73,MB=3.78
P4	MA=2.57, MB=2.6	MA=4.92,MB=4.97
P5	MA=2.75, MB=3.04	MA=3.61,MB=6.73
P6	MA=2.77, MB=0	MA=4.19,MB=2.89

Figure5.1:moment result

d= $\sqrt{\boldsymbol{M/\emptyset Rb}}$

$=\sqrt{\frac{\boldsymbol{6.98\,X\,12}}{\boldsymbol{0.9X.796X12}}}$

=3.12<3.35 in

(ok.)

P= 0.85X.85 $\frac{\boldsymbol{3.5}}{\boldsymbol{60}}X\frac{\boldsymbol{.003}}{\boldsymbol{.003+0.005}}$

= 0.0158

= $\boldsymbol{\rho fy}$ (1-0.59)

Mild Detailing Calculation:

$\mathbf{A_{smin}}$= 0.0018 bt

= 0.0018X12X6

= 0.12 in^2

a= $\frac{AsX60}{0.85f'cb}$

= $\frac{.12X60}{0.85X4X12}$

=0.176 in^2

Minimum Spacing =2t

=2X6 =12 in C/C

A_s-(A)= $\frac{Mu}{\emptyset fy\,(d-\frac{a}{2})}$

$=\frac{6.73X12}{.90X60X(3.5-\frac{0.176}{2})}$

= 0.43 in^2

A_s-(B)= $\frac{Mu}{\emptyset fy\,(d-\frac{a}{2})}$

$=\frac{6.98X12}{.90X60X(3.5-\frac{0.176}{2})}$

= 0.45 in^2

$A_s+(A)= \frac{Mu}{\emptyset fy\,(d-\frac{a}{2})}$

$=\frac{3.44X12}{.90X60X(3.5-\frac{0.176}{2})}$

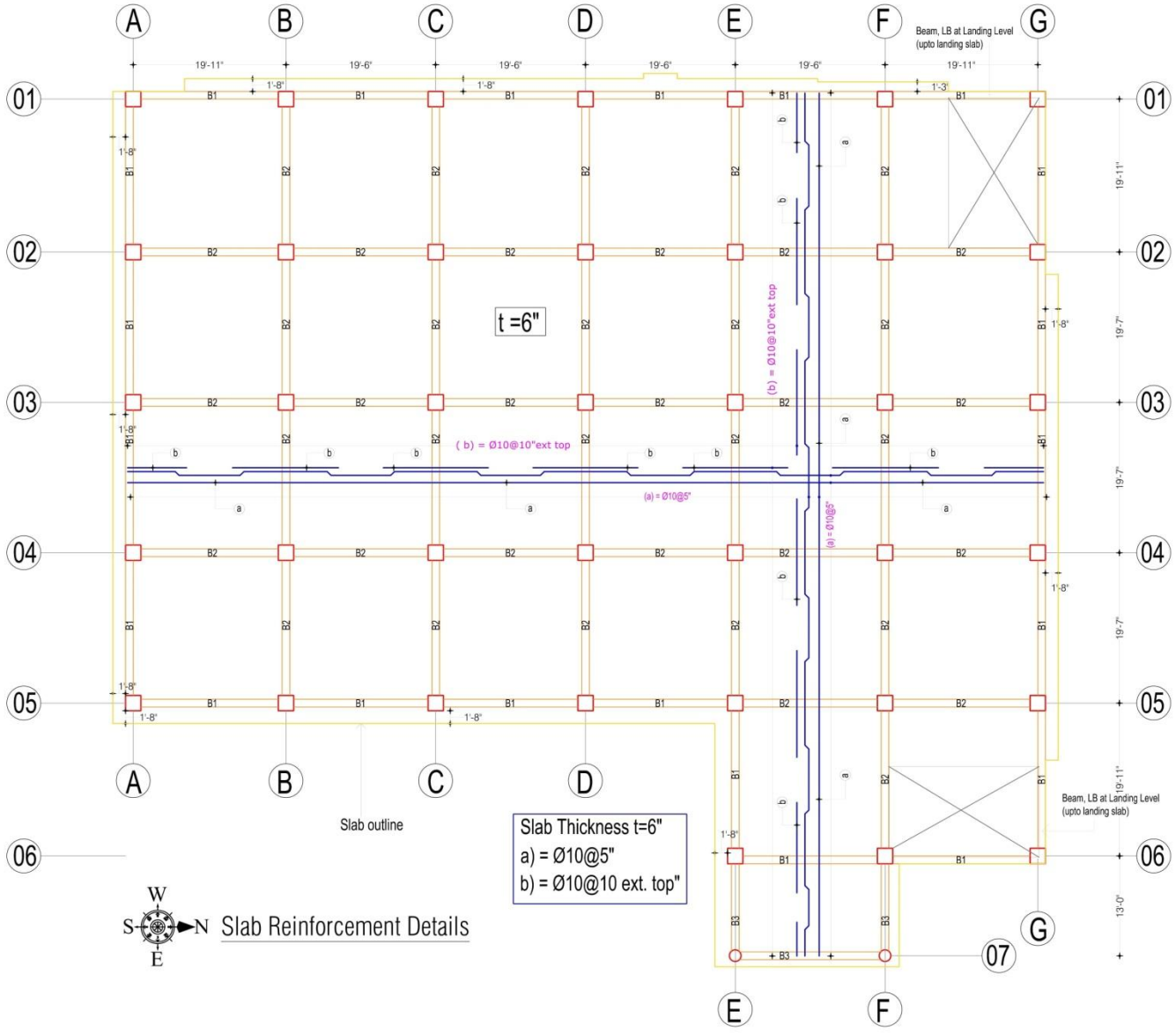
$= 0.22\ in^2$

$A_s+(B)= \frac{Mu}{\emptyset fy\,(d-\frac{a}{2})}$

$=\frac{3.31X12}{.90X60X(3.5-\frac{0.176}{2})}$

$= 0.21\ in^2$

All values are grater than A_{smin} So, Provided Calculated As Value for all Cases.


A
B
C
D
E
F
G
01
02
03
04
05
06
07
Beam, LB at Landing Level
(upto landing slab)
19'-11"
19'-6"
1'-8"
1'-3"
B1
B2
B3
t =6"
(b) = Ø10@10"ext top
(b) = Ø10@10"ext top
(a) = Ø10@5"
Slab outline
Slab Thickness t=6"
a) = Ø10@5"
b) = Ø10@10 ext. top"
W
S
N
E
Slab Reinforcement Details

5.2:Combinations of Load effects for Strength Design Method

When strength design method is used, structural members and foundations shall be designed to have strength not less than that required to resist the most unfavorable effect of the combinations of factored loads listed in the following Sections:

Basic combinations

1. 1.4(D + F)
2. 1.2D + 1.6(Lr or R) + (1.0L or 0.8W)
3. 1.2D + 1.6W + 1.0L + 0.5(Lr or R)
4. 1.2D + 1.0E + 1.0L
5. 0.9D + 1.6W + 1.6H
6. 0.9D + 1.0E + 1.6H

Comparison between BNBC 2017 & ASCE 7-05 for Earthquake & Wind Load

Earthquake Load Calculation:

- Base Shear, V = S_aw ---------- (6.2.37)

Here $S_a = \frac{2}{3} X \frac{ZI}{R} CS$

1. Site Classification:

SPI Value, $\overline{N} = \frac{\sum di}{\sum \frac{di}{Ni}}$ ----------(6.2.32)

$$\overline{N} = \frac{5+5+5+5+5+5+5+5+5+5+5}{\frac{5}{8}+\frac{5}{11}+\frac{5}{13}+\frac{5}{20}+\frac{5}{7}+\frac{5}{14}+\frac{5}{18}+\frac{5}{22}+\frac{5}{24}+\frac{5}{31}}$$

= 13.66

$\overline{N}$ (15m) = 13.66

Site Class = S_c

2. Seismic Zone Coefficient , Z :

For Dhaka Z= 0.20
Seismic Zone = 2

3. Structure important factor I

Occupancy Category = III
Important Factor I = 1.25

- Spectrum , Cs

Building Period $T= C_t (h_n)^m$

Structure type ➡ Concrete moment resisting frame
$C_t = 0.0466$
$m = 0.9$

$\therefore\ T= C_t (h_n)^m$
$= 0.0466*(21.95)^{0.9}$
$= 0.753$

For Soil type

Soil Factor S= 1.15

$T_{B(s)}$	$T_{C(s)}$	$T_{d(s)}$
0.20	0.60	2.0

$C_s = 2.5S_n (\frac{Tc}{T})$

For $T_c \leq T \leq T_d$

$0.60 \leq .75 \leq 2.0$

$C_s = 2.5 \text{ X } 1.15 \text{ X } 1 \text{ X } \frac{0.60}{0.75}$

$C_s = 2.3$

- Response reduction factor , R :

1. For site class S_c& Occupancy category
2. Seismic Design Category = C

3. Intermediate reinforced concrete moment Frame R=5

$\therefore S_a= \frac{2}{3} X \frac{ZI}{R} XCs$

$$= \frac{2}{3} X \frac{0.20 X 1.25}{5} X 2.3$$
$$= 0.076$$

Now Check with minimum S_a Value ,

$S_a = 0.67\beta ZIS$
$= 0.67 \times 0.11 \times 0.20 \times 1.25 \times 1.15$
$= 0.021$

$S_a > S_{a\,(Minimum)}$

$\therefore$ Base Shear

$V = S_a X W$

T	k	Interpolate Value
<0.50s	1	1.29
1.08	?	
≥2.5s	2	

The seismic base shear force,

V=SaW
=0.0076x22930.73
=1742.73 kip

Here,
Sa=0.076 (Lateral seismic force coefficient)
W=2250 kip (Total seismic weight of the building defined in Sec 2.5.7.3)

$$\boldsymbol{F_x = V_x\left(\frac{Wxhxk}{\sum_{i=1}^{n} Wihjk}\right)}$$

Etabs Results:

Direction	Period Used (sec)	C_s	W (kip)	V (kip)
X	0.755	0.076159	22930.7032	1746.3781
X + Ecc. Y	0.755	0.076159	22930.7032	1746.3781
X - Ecc. Y	0.755	0.076159	22930.7032	1746.3781

Table5.7:story shears

Story	Elevation ft	X-Dir kip	Y-Dir kip
ROOF	72	536.523	0
6TH	62	318.406	0
5TH	52	267.608	0
4TH	42	219.776	0
3RD	32	167.908	0
2ND	22	127.728	0
1ST	12	79.949	0
GF	0	28.481	0
Base	-8	0	0

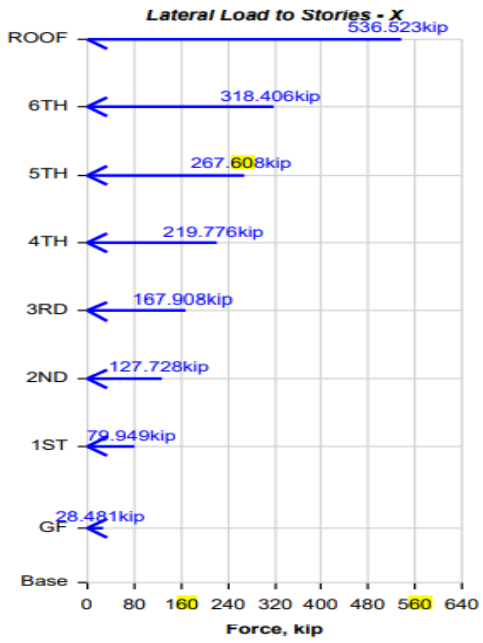


Figure 5.2:lateral load to stories-x

Table5.8:story shears

Story	Elevation ft	X-Dir kip	Y-Dir kip
ROOF	72	0	192.476
6TH	62	0	101.665
5TH	52	0	74.693
4TH	42	0	52.321
3RD	32	0	32.901
2ND	22	0	19.472
1ST	12	0	8.557
GF	0	0	1.37
Base	-8	0	0

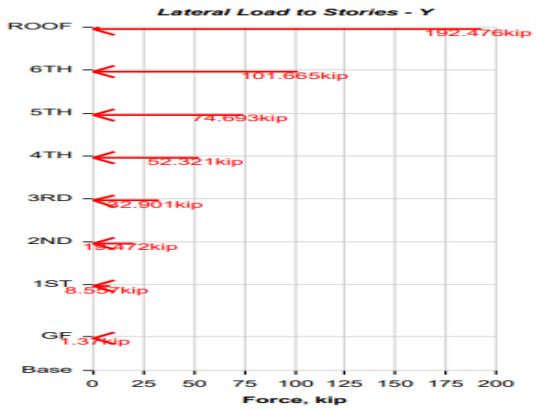


Figure5.3:lateral load to stories-y

5.3:Wind Load Calculation:

Location:	Dhaka
Building Type :	Factory Building
Number Of Story :	G+6 (7 Storied)
Building Type :	Partially Enclosed Building
Exposure Category :	B
Floor Height :	3.048 meter
Total Height :	21.95m (Floor ground to Roof)

Velocity pressure

Velocity pressure, q_z evaluated at height z shall be calculated by the following

equation:

$q_z = 0.000613\ K_z\ K_{zt}\ K_d V^2 I$; ($kN/m^2$), V in m/s BNBC Eq(6.2.17)

Where

K_d =wind directionality factor = 0.85 BNBC Table 6.2.12

K_z = velocity pressure exposure coefficient defined BNBC Table 6.2.11

$K_{z\,t}$= topographic factor = 1 BNBC Sec 2.4.7.2

I = Importance factor =1.25 BNBC Table 6.2.9

V = Velocity Pressure =65.7m/s BNBC Table 6.2.8

$q_z = 0.000613 * K_z * 1 * 0.85 * 65.7 * 1.25\ (kN/m^2)$

$= 2.58\ k_z kN/m^2$

$P = qGC_p - q_i(GC_{pi})\ (kN/m^2)$ BNBC Eq 6.2.17

Where

$q = q_z = 2.85\ K_z\ (kN/m^2)$ for windward walls evaluated at height $q = q_h$ for leeward walls, side walls, and roofs, evaluated at height h

$q_i = q_h$ for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings

$q_i = q_z$ for positive internal pressure evaluation in partially enclosed buildings where height is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, glazing that is not impact resistant or protected with an impact resistant covering, shall be treated as an opening in accordance with Sec 2.4.9.3. For positive internal pressure evaluation, may conservatively be evaluated at height h = ($q_i = q_h$)

G = gust effect factor from =0.85 BNBC Sec 2.4

C_p = external pressure coefficient =0. 8 BNBC Figure 6.2.6

GC_{pi} = internal pressure coefficient = 0.55 BNBC Figure 6.2.5

P= 2.58kz X 0.85 X 0.3 – 2.58kz X (+0.55)

= 0.65kz – 1.41 kz

=0.65 X 0.89 – 1.41 X .89

= 0.58-1.25 = 0.67

So Pressure at leeward = 0.67 kN/m^2

$K_z = 2.01 \left(\frac{21.95}{365.76}\right)^{\frac{2}{7}}$

$= 0.89$

Now Pressure at windward

P= qGCP-qi(GCpi)

= 2.58kz X 0.85 X 0.8 – 2.58kz X (-.55)

= 1.754kz + 1.419 kz

= 3.173 kzkN/m^2

Pressure at 1stFloor = 3.173 X 0.54

= 1.71 kN/m^2

Pressure at 2nd Floor = 3.173X0.64

=2.03 kN/m^2

Pressure at 3rdFloor = 2.26 kN/m^2

Pressure at 4thFloor = 2.45 kN/m^2

Pressure at 5th Floor = 2.60 kN/m^2

Pressure at 6th Floor = 2.74 kN/m^2

Pressure content upto 4.75 m Height

Net Wind Pressure at different Level

Net Pressure = (windward + Leeward Pressure)

1st Floor = (1.71+0.67) = 2.38 kN/m^2

2nd Floor = (2.03+0.67) = 2.70 kN/m^2

3rd Floor = (2.26+0.67) = 2.93 kN/m^2

4th Floor = (2.45+0.67) = 3.12 kN/m^2

5th Floor = (2.60+0.67) = 3.27 kN/m^2

6th Floor = (2.70+0.67)= 3.41kN/m^2

Pressure at Each Floor or Vertical distribution at each floor

Area = 29.78X3 = 89.34

1st Floor = 89.34 X 2.38 = 212.73

2nd Floor = 89.34 X 2.70 = 241.63

3rd Floor = 89.34 X 2.93 = 262.17

4th Floor = 89.34 X 3.12 = 278.51

5th Floor = 89.34 X 3.27 = 292.27

6th Floor = 89.34X3.41= 304.25

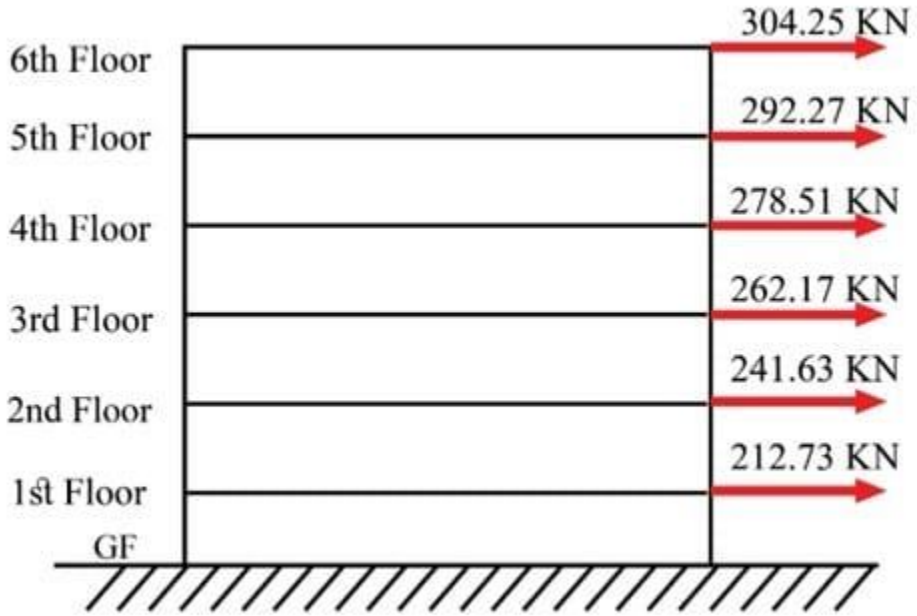


Figure5.4:Pressure at each floor

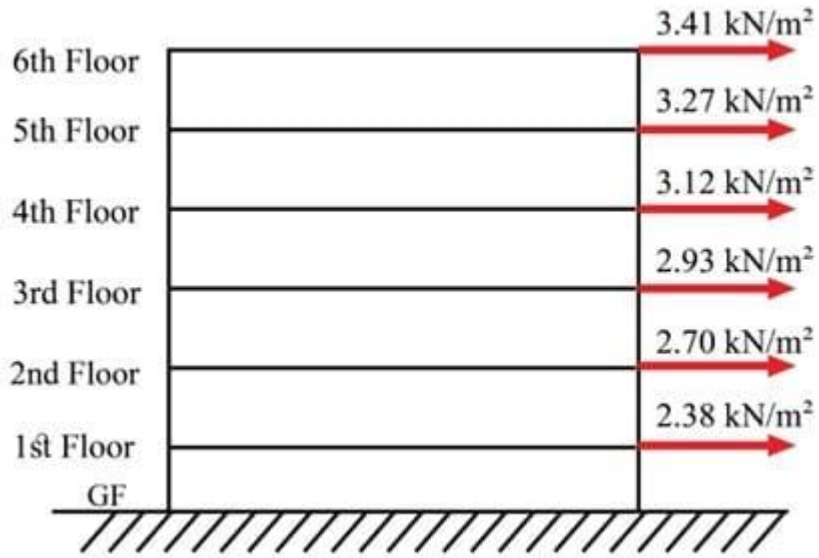


Figure5.5:Net pressure

Etabs result:

Story	Elevation ft	X-Dir kip	Y-Dir kip
ROOF	72	0	15.364
6TH	62	0	30.366
5TH	52	0	29.774
4TH	42	0	29.098
3RD	32	0	28.302
2ND	22	0	27.327
1ST	12	0	28.607
GF	0	0	25.282
Base	-8	0	0

Story	Elevation ft	X-Dir kip	Y-Dir kip
ROOF	72	-12.742	0
6TH	62	-25.183	0
5TH	52	-24.687	0
4TH	42	-24.121	0
3RD	32	-23.456	0
2ND	22	-22.64	0
1ST	12	-23.689	0
GF	0	-23.689	0
Base	-8	0	0

5.4:BEAM DESIGN: T Beam & Doubly Beam Check,

T Beam:

First determining the effective flange

width b_f ,

$16h_f + b_w = 16X6+12= 108$ in

$2\frac{ln}{8} + b_w = 2\ X\frac{18.91\ X\ 12}{8} + 12 = 68$ in

Centerline beam spacing = 9 X 12 = 108 in

$d-\frac{a}{2}= 18-\frac{6}{2}= 15$ in

Trail :

$A_s= \frac{Mu}{\emptyset\ fy(d-\frac{a}{2})} = \frac{4.41X60}{0.85X4X108}=0.72$

Another ,

Checking the assumed value for a

$a= \frac{Asfy}{0.85f'cXb} = \frac{Asfy4.41X60}{0.85X4X12} = 6.48$

Since,

a is greater than h_f , a T Beam design is required and Ø = 0.90 is assumed

$A_sf = \frac{0.85\ f'cbhf}{fy} = \frac{0.85X4X12X6}{60} = 4.08\ in^2$

$øM_{n1} =ØA_{sf}fy\ (d-\frac{hf}{2}) = 0.90\ X\ 4.08\ X\ 60(\ 18--\frac{6}{2})$

= 3304.8 kip-in

$øM_{n2} =Mu- øM_{n1}= (300X2)-3304.8$

= 295.2 kip-in

Assume, a = 7 in,

$A_s-A_{sf}= \frac{\emptyset Mn2}{øfy(d-\frac{a}{2})} = \frac{295.2}{.90X60X(18-\frac{7}{2})} = 0.33\ in^2$

$= \frac{(As-Asf)fy}{.85f'cbw} = \frac{0.33X60}{0.85X4X12}$

=0.48 in

$A_s= A_{sf} + (A_s-A_{sf}) = 6.48 + .33 = 6.15$

C= $\frac{a}{B1}$ = $\frac{.48}{.85}$ = .56

$\frac{c}{d}$= $\frac{0.56}{18}$ = 0.031

Doubly Beam:

Assume ,

d= 14 , d'= 2.5

f'$_c$= 4000, f_y=60,000

ø=90

A_s= 12X14X0.01814

= 3.04 in^2

a= $\frac{Asfy}{0.85f'cXb}$ =$\frac{3.04X60}{0.85X4X12}$ = 4.47 in

c= $\frac{a}{\beta 1}$ = $\frac{4.47}{0.85}$=5.25 in

And the maximum nomined moment that can

Be developed is ,

M_n= 3.04X60(14-$\frac{4.47}{0.85}$) = 2145.93 in-kips

M_n= 144 kip-ft

=1728 in-kip

M_1= $\frac{2145.93}{.90}$-1728

=656.36 in-kips

$\sum' s$= 0.003 $\frac{6.02-2.5}{6.02}$=0.0017

f'$_s$= 0.00175 X 29000 = 50.9 ksi

Try ,

A's= $\frac{656.36}{50(14-2.5)}$ = 1.14 in^2

The total area of tensile reinforcement at 60 ksi

As = 3.04 + 1.14 ($\frac{50}{60}$)

= 3.99 in^2

A_s-A'$_s$ ($\frac{f's}{fy}$) = 40-0.88 ($\frac{50}{60}$) = 3.27 in^2

Which is greater than 3.04 for

$\sum' t$= 0.005

So , ø < 0.90

a= $\frac{3.27X60}{0.85X4X12}$ = 4.80in

$c=\frac{4.80}{.85}$= 5.64 in

$\sum 's$= $0.003\frac{5.64-2.5}{5.64}$

=0.0017

f'$_s$ =29000X 0.017

= 49.3 ksi

Which is greater than assumed check ø used d_t=15.5in

From the strain distribution ---------- M_u,

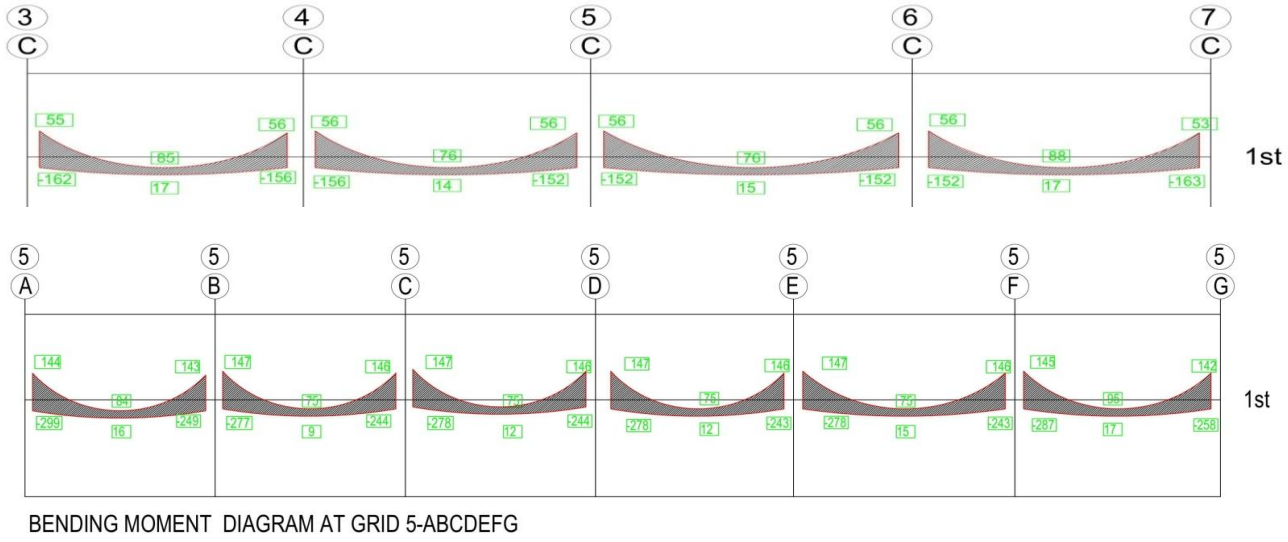
$\sum 't$= $0.003\ \frac{15.5-5.64}{5.64}$= 0.0052

For Which = 0.87 [3.27X60(14-$\frac{4.80}{2}$)+ 0.88X49.3X(14-2.5)]

=2414>M_u

∴øM_n >M_u.

Data From ETABS:



BENDING MOMENT DIAGRAM AT GRID 5-ABCDEFG

Figure5.6:data from ETABS

PROPERTY | GEOMETRY | REINFORCEMENT | LOADINGS

Project:	Factory Building				
Location:	Savar				
Client:	VUB				
Job No.:	1	Calc. by:	RCF	Checked by:	RCF
Subject:	Beam	Date:	22-Nov-22	Revision:	-

ANALYSIS & DESIGN OF RC BEAM AS PER ACI 318-08

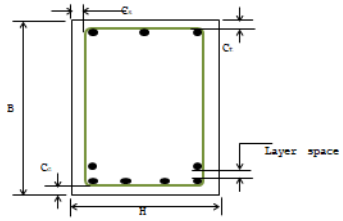
INPUT DATA

MATERIAL PROPERTIES

f'_c	=	27	Mpa	CONCRETE STRENGTH
f_y	=	414	Mpa	REBAR YIELD STRESS
f_{yt}	=	414	Mpa	LINKS YIELD STRESS
E_s	=	200,000	Mpa	Steel Modulus of Elasticity
E_c	=	24,422	Mpa	E_c = 4700*SQRT(f'c)

DIMENSIONS

L	=	5.969	m	BEAM LENGTH
B	=	300	mm	BEAM WIDTH
H	=	450	mm	BEAM HEIGHT
C_t	=	40	mm	COVER TO TOP REINFORCEMENT
C_b	=	40	mm	COVER TO BOTTOM REINFORCEMENT
C_s	=	40	mm	COVER TO SIDE BARS



STEEL REINFORCEMENT

LEFT SUPPORT

TOP REINFORCEMENT

layer 1		layer 2		layer space
no.	mm	no.	mm	
3	Ø 20	2	Ø 20	25

BOTTOM REINFORCEMENT

layer 1		layer 2		layer space
no.	mm	no.	mm	
4	Ø 20	2	Ø 20	25

HOOPS

no.	spacing	no.	spacing
1	50	1	75

MIDSPAN

TOP REINFORCEMENT

layer 1		layer 2		layer space
no.	mm	no.	mm	
3	Ø 20	2	Ø 20	25

BOTTOM REINFORCEMENT

layer 1		layer 2		layer space
no.	mm	no.	mm	
4	Ø 20	2	Ø 20	25

STIRRUPS

legs	mm	spacing
2	Ø 10	150

RIGHT SUPPORT

TOP REINFORCEMENT

layer 1		layer 2		layer space
no.	mm	no.	mm	
3	Ø 20	2	Ø 20	25

BOTTOM REINFORCEMENT

layer 1		layer 2		layer space
no.	mm	no.	mm	
4	Ø 20	2	Ø 20	25

ADDITIONAL LONGITUDINAL BARS

no.	mm
0	Ø 10

APPLIED FORCES

figure5.7:Beam Reinforcement From ETABS.

Checks:

Moment Capacity and Crack Control Checks

Left Support:

ϕMn	=	236.66	kN-m	> Mu = 74 kN-m, O.K.
Spacing	=	66.67	mm	s(min) <= s(act) <= s(max), O.K.!

Midspan:

ϕMn	=	236.66	kN-m	> Mu = 23 kN-m, O.K.
Spacing	=	50.00	mm	s(min) <= s(act) <= s(max), O.K.!

Right Support:

ϕMn	=	236.66	kN-m	> Mu = 75 kN-m, O.K.
Spacing	=	66.67	mm	s(min) <= s(act) <= s(max), O.K.!

Shear Capacity and Torsional Capacity Checks

Spacing of hoops for seismic provisions

S(max)	=	96.25	mm.o.c.	> 75mm, O.K.
@2H	=	900	mm	> 75mm hoops must be provided @ distance 2H from face of support.
ϕVn	=	201.70	kN	>=Vu = 3.20 kN, O.K.
Av_{min}	=	38.04	mm^2	<= Av(prov) = 157.08 mm^2, O.K.
s_{max}	=	192.50	mm.o.c.	mm

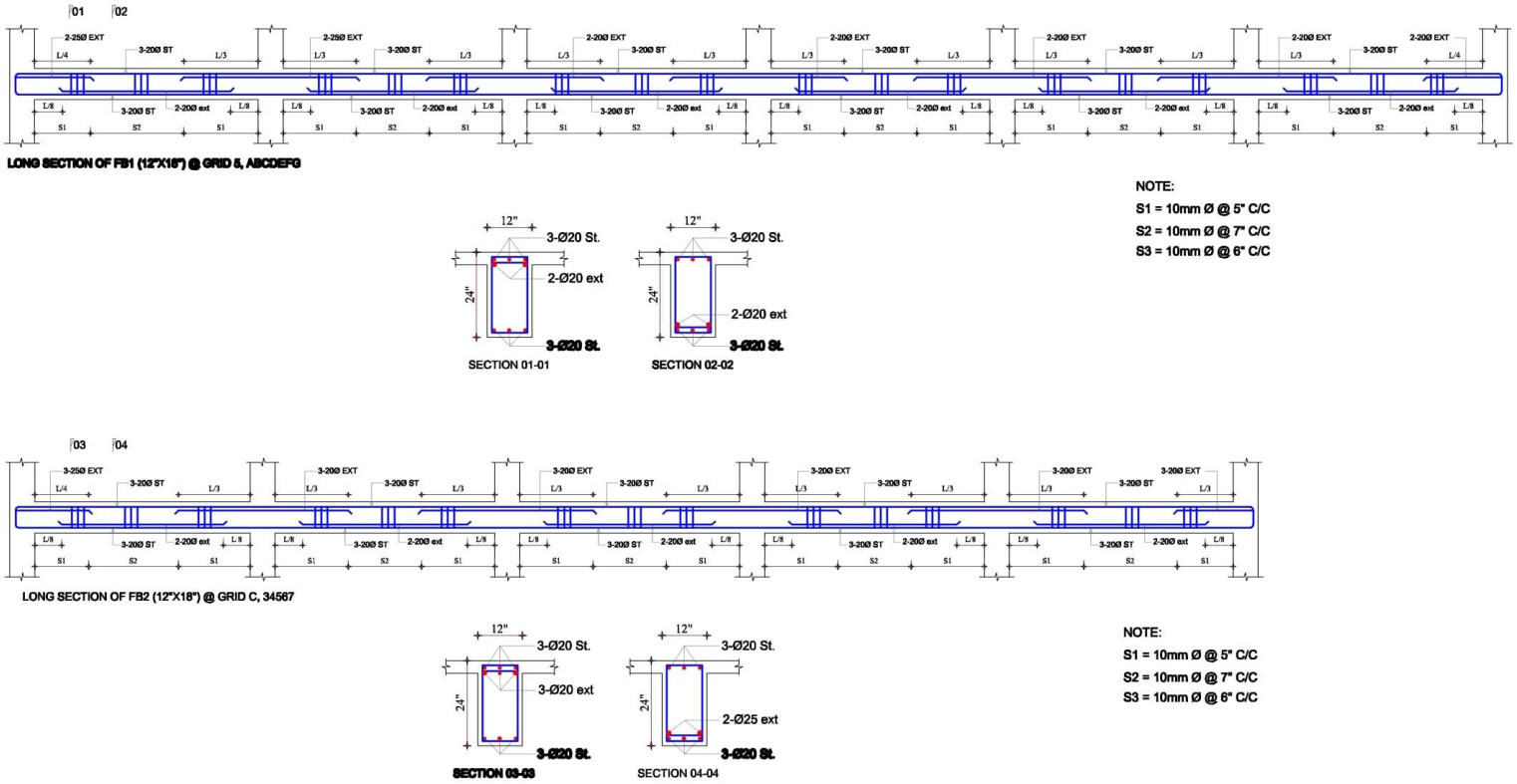


Figure 5.8:long section of beam

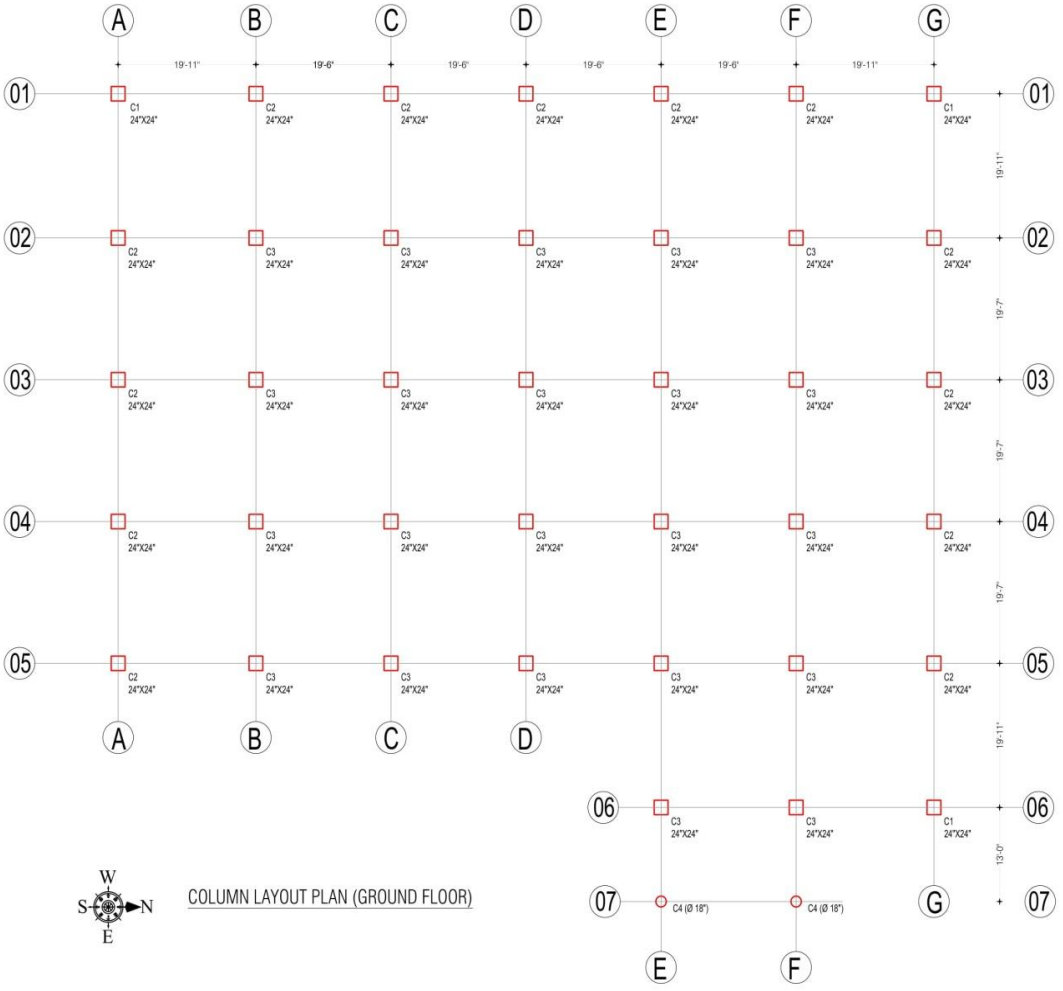


Figure5.9:Column Layout Plan for Ground Floor

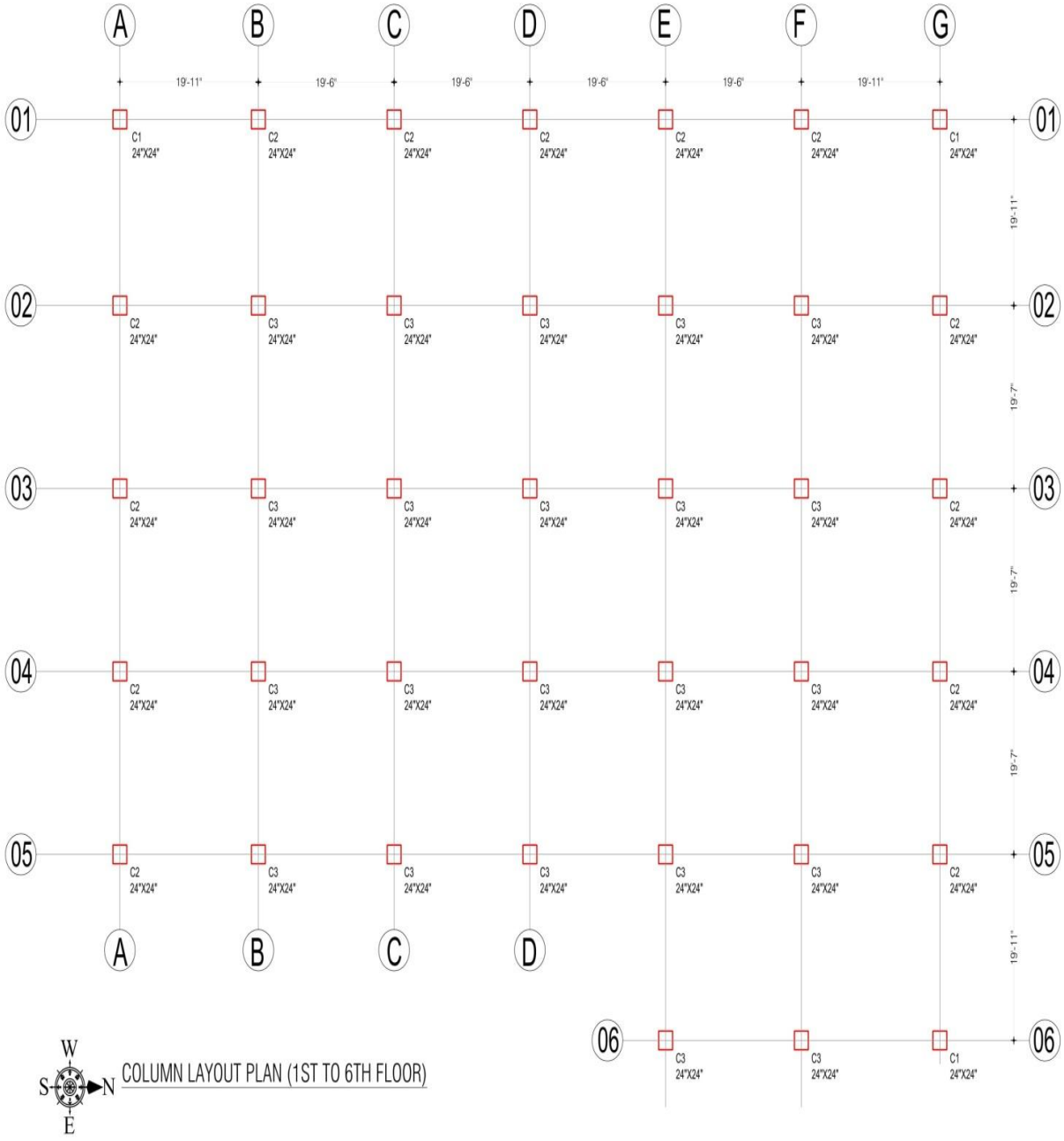


Figure5.10:Column Layout Plan 1st to 6th

5.5:Column Design:

Rectangular column.

Data From Etabs:

Column size: 24''x24''

P_u = *643 kip*, M_x = *226* and

M_y = *547 k-ft, fy = 60 ksi & f'c = 4 ksi*

$$K_n = \frac{P_u}{\varphi f'_c A_g} = \frac{643}{0.65x4x(576)} = 0.42$$

Eccentricity, *e= M/P= (354x12)/643=6.60*

$$R_n = \frac{M_u}{\varphi f'_c A_g h} = \frac{474x12}{0.65x4x576x25}$$

=0.15

With 2.5 in cover the parameter, $\gamma = \frac{24-5}{24} = 0.79$ for this column

The required reinforcement is

steel ratio $\rho = 0.014$

$A_s = \rho A_g = 0.014\ x\ (24x24) = 8\ in^2$

So use 14#7 bar

Tie bar design

a) 48x Diameter of tie bar = 48 x 3/8=18 in. [S ≤ 16 d_b]

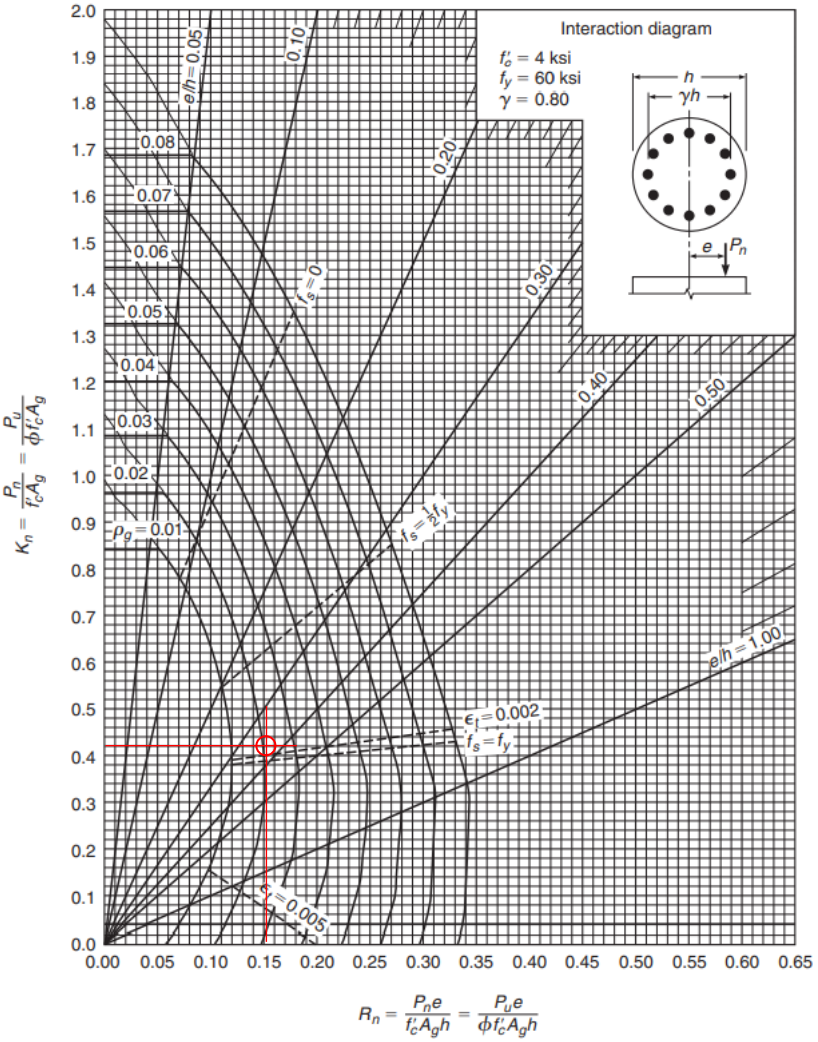
b) 16 × Max. dia among main bar of column = 16 × 7/8 = 14 in (governs) [S ≤ 48×d_t]

c) Least dimension of column section = 24 in

[S ≤ Least diameter of the column]

So, use #3 @14" c/c

1. Geometry and material strength, Graph A6 the Calculate values as kn, Rn, ρg = 0.014(reference:Nilson, Arthur.H, Darwin David and Dolan Charles W. (2010-2011). "Design of Concrete Structures" (14th Ed.) McGraw-Hill book Co. Singapore. P.P 755 (graphA).



For this column geometry and materials strength, Graph A7 of Appendix A applies with thecalculated values of ***Kn*** and ***Rn.***

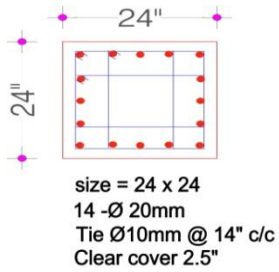


Figure5.11: Reinforcement detailing of Column

Boring no.-03									
Cum depth	Depth	Cum depth	S.P.T.	Corrected S.P.T.Value N-(N-150)/2	Average S.P.T	Allowable Skine Friction Qa=02N/F.S.	Total Allowable Skin Friction QA=.02N/F.S	Allowable End Bearing Capacity Qa=4N/F.S.	ALLOWABLE WORKING LOAD For Cast-in-situ
meter	ft	ft	no	No	no	ton	ton	tons	tons
1.52	5	5.00	9	9	7.5	1.18	3.69	21.21	16.60
3.05	5	10.00	14	14	11.5	1.81	5.66	33.00	25.77
4.57	5	15.00	17	16	14	2.20	6.44	37.71	29.44
6.10	5	20.00	20	17.5	16.25	2.55	10.18	41.25	34.28
7.622	5	25.00	10	10	9	1.41	3.77	23.57	18.23
9.14	5	30.00	14	14	11.5	1.81	5.66	33.00	25.77
10.67	5	35.00	18	16.5	15.75	2.48	6.64	38.89	30.35
12.19	5	40.00	22	18.5	17.75	2.79	12.81	43.61	37.48
13.72	5	45.00	26	20.5	19.25	3.03	13.00	48.32	40.88
15.24	5	50.00	32	23.5	22.5	3.54	19.29	55.39	49.79
Boring no.-04									
1.52	5	5.00	8	8	7.5	1.18	2.36	18.86	14.14
3.05	5	10.00	11	11	10	1,57	5.58	25.93	21.oo
4.57	5	15.00	13	13	12	1,89	7.46	30.64	25.41
6.10	5	20.00	20	17.5	16.25	2.55	10.18	41.25	34.28
7.622	5	25.00	7	7	5	0.79	1.18	16.50	11.79
9.14	5	30.00	14	14	11.5	1.81	5.66	33.00	25.77
10.67	5	35.00	18	16.5	15.75	2.48	6.64	38.89	30.35
12.19	5	40.00	22	18.5	17.75	2.79	12.81	43.61	37.48
13.72	5	45.00	24	19.5	19	2.99	12.38	45.96	38.89
15.24	5	50.00	31	23	21.25	3.34	15.72	54.21	46.62

Note:

a. fast = Allowable value of the skin friction.
b. qpa = Allowable value of the pile end bearing capacity.
c. SPT (N) values are corrected within calculation.
d. The values of fsa and qpa have been halved in making preliminary estimate about the carrying capacity of a Bored R.C.C. pile.
e. In the case of plastic soil, the values of the cohesion have been obtained from the SPT values.
f. The drive values are approximate. Structural/foundation engineer can refine/ recalculate, taking the soil parameters obtained from field & lab tests.
g. The theories on the ultimate bearing capacity presented in this report are based on idealized conditions of soil profiles. This is, in most field conditions, not true. Soil profilesare not alwys homogenous and isotropic.
h. Hence, experience and judgment are always necessary in adopting proper soil parameters to use the calculation of ultimate bearing capacity. 1Tsf = 2ksf = 1.09 Kg/cm2, 1Ton = 1000kg = 2000 Lbs, E.G.L. = Existing Ground Leve

Table 5.9:The skin friction & the end bearing capacity of piles as shown in table(F.S=3.00).
(Reference: Das Braja.M (1984), "Principles of Foundation Engineering" (Sixth Edition).)

Assume

pile dia, D_p = 18”
Pile depth = 60 ft

Bearing Capacity for Skin friction, Qs = Asfs

= π*D*h*fs
= π*1.5*60*0.09
=25.44 ton =50.88 kip

Bearing Capacity for Skin friction:

$$Q_b = \frac{\pi D^2}{4} x f_b$$

$$= \frac{\pi x (1.5^2}{4} x 23$$

=40.64Ton =81.28 kip

Now, Allowable load capacity of pile,

Q_{all} = (Qs+Qb)

= (50.88+81.28) kip

=132.16 kip. = 130 kip

So, Pile dia is 18” and pile length-60ft.

5.6:Pile Design Calculation:

Self-weight of Pile $= \frac{\pi}{4} x 1.5)^2\ x60x0.15 = 15.90\ kip$

Max^{m} load on Pile $P_u = 156.75\ kip$

Total load on Pile $= 156.75 + 19.7 = 177\ kip$

Yield Strength of steel, $f_y = 60\ ksi$

Compressive Strength of concrete, $f'_c = 3.5\ ksi$

Diameter of pile D_p = 20 in and length of a pile = 60 ft

Therefore, Gross cross sectional area of pile

Ac = $\frac{\pi}{4}$xDp^2 = $\frac{\pi}{4}$x18^2 = 254.46 in^2

According to BNBC sec 3.10.4.9, the longitudinal reinforcement shall be of high yield steel bar (Minimum f_y= *60 ksi*) shall not be less than 0.5 % of A_c [For $A_c \leq 0.5\ m^2$]

According to BNBC Table 6.3.11, for 18 in dia pile; minimum dia. of bar $\frac{5}{8}$ (16 mm) and minimum no. of bar is 6 nos for main reinforcement.

Provide $\frac{5}{8}$" dia (16mm) bar (area = 0.31 in^2)

Therefore, Required Number of bar

$=\frac{1.57}{0.31} = 5 < 6$ Nos

So, provide $\frac{5}{8}$" dia (16mm) bar as longitudinal reinforcement considering 3" or 75mm clear cover.

Check for design load

For spiral column Ø= 0.75 α= 0.85

Area of steel, A_s = 0.31 ×6 = 1.87 in^2

$\rho_g = \frac{A_s}{A_c} = \frac{1.87}{254.46} = 0.007$

We know,

$P_u = \text{Ø}\alpha A_g[0.85f'_c(1-\rho_g) + \rho_g f_y]$

$= 0.75\text{x}0.85\text{x}254.46\text{x}[0.85x3.5(1-0.007) + 0.007x60]$

$= 547.35\ \text{kip} > 172\ \text{kip}$ (ok)

Lateral Reinforcement

According to BNBC; for 18 in dia. pile the lateral reinforcement should be of #2 bar (8 mm)

a) Provide #2 (8 mm) dia. bar @ 3 in (75mm) c/c as lateral reinforcement at the top of the pile for at least 5 D_p but not less than 5.9 ft. (1.8 m) below the bottom of the pile cap.

b) Provide #2 (8 mm) dia. bar @ 6 in (150mm) c/c as lateral reinforcement

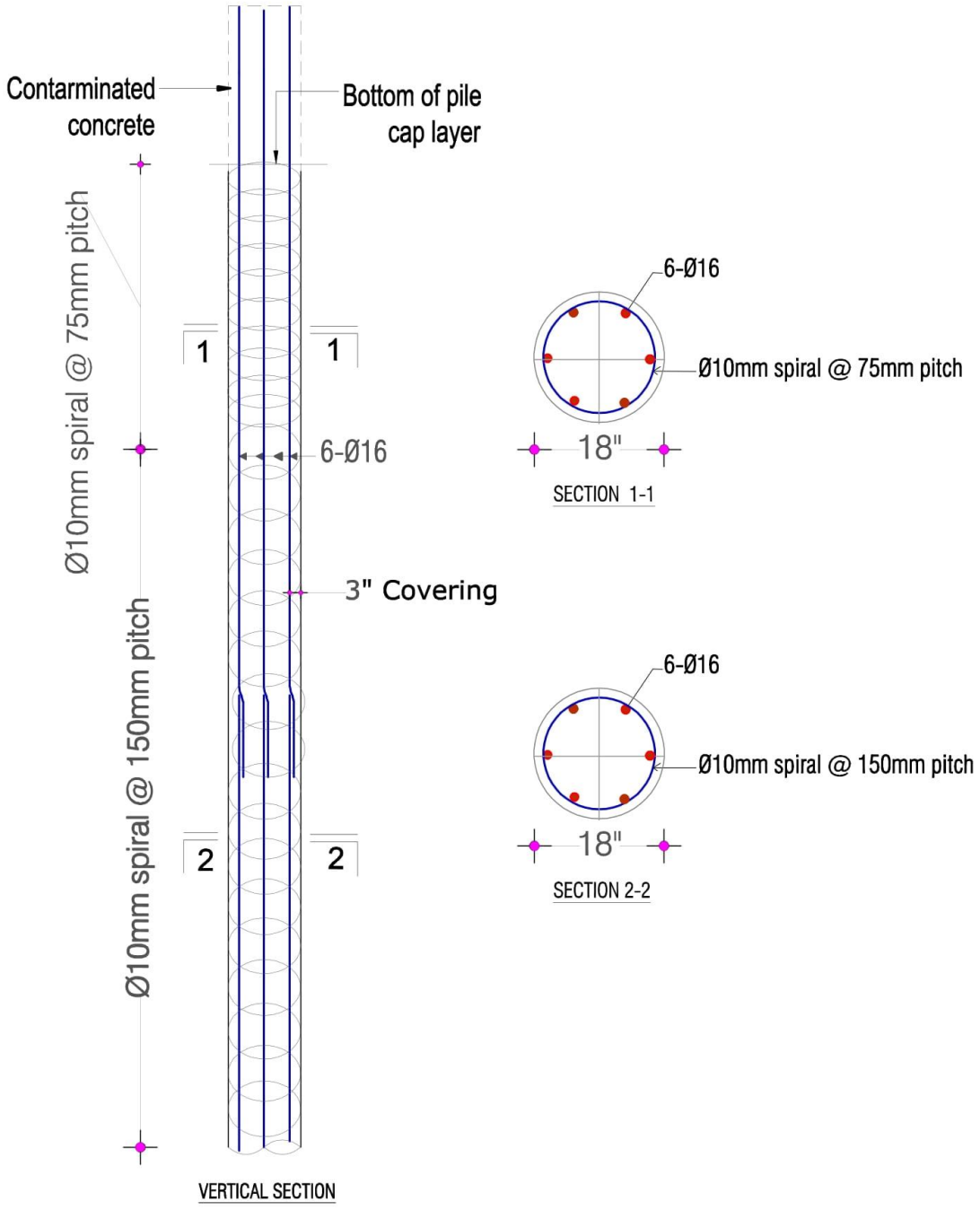
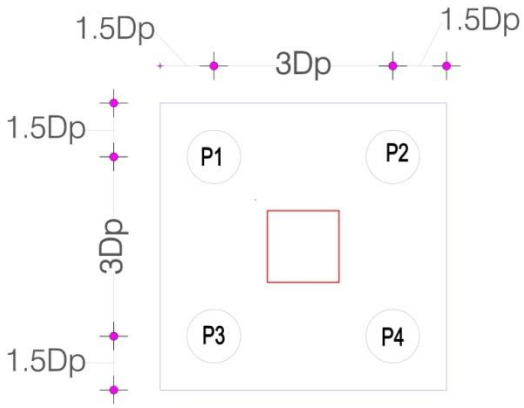


Figure 5.12: Spiral details of cast in situ pile

5.7:Pile Cap Design:

For Column-C_{10} (24"x24") :



Pile load-

Un-factored load=450 kip

Factored load=540 kip

Pile Capacity=130 kip

$$\text{No. of pile} = \frac{450}{130} = 3.46 \text{ nos} = 4\ nos$$

$$\text{Pile reaction} = \frac{540}{4} = 135\ kip$$

Assume, Pile cap depth= 21"

Ultimate Shear,

Vu=4*135=540 kip

b_o= 2*(24+21)+2(24+21)= 180 in.

ΦVc=0.75*4*√f'c*b_od

=0.75x4x$\frac{\sqrt{4000}}{100}$x180x21

=717 kip > Vu (ok)

M=2x135x2=540 kip-ft.

Long Direction moment:

$$A_s = \frac{M_u}{\varphi\, f_y (d - \frac{a}{2})}$$

$$= \frac{540x12}{0.90x60x(21 - \frac{2.1}{2})}$$

= 6.01 in^2

a= 0.1d = 0.1x21 = 2.1

Check , a= $\frac{Asfy}{.85\,f'cb} = \frac{6.01X60}{0.85X4X12X12} = 0.73$

$$A_s = \frac{M_u}{\varphi\, f_y(d - \frac{a}{2})}$$

$$= \frac{540x12}{0.90x60x(21-\frac{.73}{2}}$$

$=5.81\ in^2$

Minimum Steel area,

Minimum Steel area ,

As min = ma X ($\frac{3\sqrt{f'c}}{fy}$bd, $\frac{200}{fy}$bd)

= ma X ($\frac{3\sqrt{4000}}{60000}$x12x12x21, $\frac{200}{fy}$x12x12x21)

= (9.56, 10.08)

Calculate development length

Available l_d= 51”

d_b= 10.08/8 in , ψ_t= 1.0;

$\psi\in$= 1 ; λ= 1

l_d=($\frac{fy\psi t\psi\in}{20\lambda\sqrt{f'c}}$) d_b

= (60000X1X1) /(20X1X$\sqrt{4000}$) X 7/8

= 42 in < 51 in (ok)

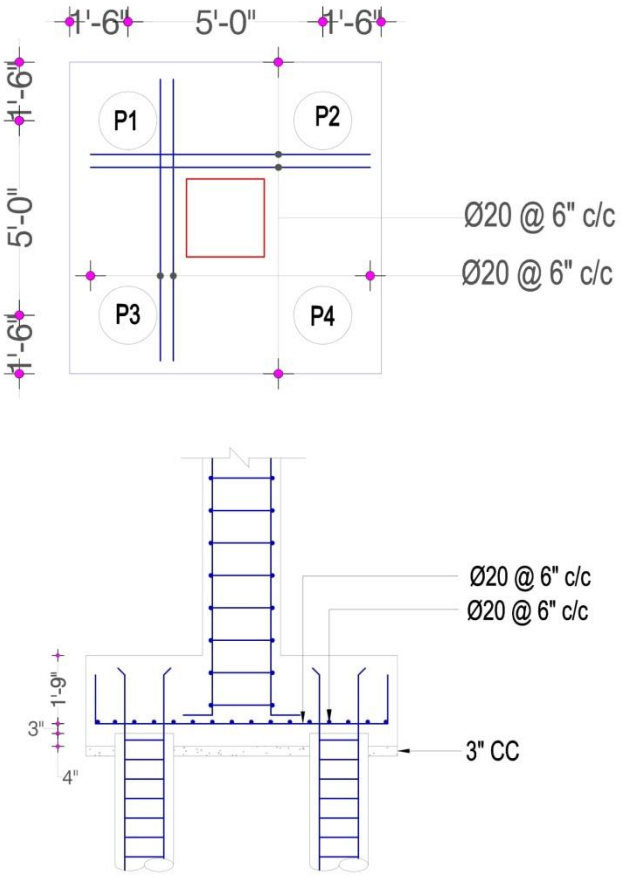


Figure5.13: Reinforcement Details of Pile Cap

5.8:conclusion and recommendations:

CONCLUSION**:**

Amount of moment at a given section increased multiple times under earthquake loading. Overall cost of the building doubled after considering the earthquake loading during design process.

RECOMMENDATIONS:

Further study of different types of building to verify the cost ratio of ordinary structure to earthquake resistance structure. Further study required to verify the cost effectiveness of BNBC against other international codes like Euro Code, Indian Code etc. for designing earthquake resistance building.

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