What happens to the design when two more stories are added on top of an existing six storied residential structure in Khulna?

CE-400 (project/thesis)

Supervised by:

Mr. Rayhan Md. Faysal

Assistant Professor

Department of Civil engineering, Daffodil International University

Daffodil International University

102, Sukrabad, Dhanmondi, Dhaka 1207

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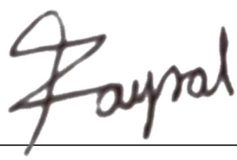
SUBMITTED BY

Name	ID
Nazibullah hossain Shourov	171-47-341
Mohamed abdirisak said	173-47-606
Md. Kabirul hasan	173-47-537
Tarakur Rahman	182-47-738
Yasir Hassan Mohamud	172-47-381

APPROVAL

This is to certify that the thesis entitled "What happens to the design when two more stories are added on top of an existing six storied residential structure in Khulna?" submitted by **Nazibullah Hossain Shourov** (171-47-341, Session Spring 2017) **Mohamed Abdirisak Said** (173-47-606, Session Fall 2017), **Md. Kabirul Hasan** (173-47-537, Session Fall 2017), **Tarakur Rahman** (182-47-738 Session Summer 2018), and **Yasir Hassan Mohamud** (172-47-381, Session Summer 2017) has been accepted as satisfactory in partial fulfillment of the requirement for the degree of **Bachelor of Science in Civil Engineering** on June 2021.



Mr. Rayhan Md. Faysal

Assistant Professor

Department of Civil engineering,

Daffodil International University

DECLARATION

It is stated that the work "What happens to the design when two more stories are added on top of an existing six storied residential structure in Khulna?" reported in this thesis has been performed under the supervision of **Mr. Rayhan Md. Faysal** Assistant Professor Department of Civil engineering, Daffodil International University. The thesis contains no material previously published or written by another person, to the best of our knowledge and belief except where due to reference were made in the thesis itself.

Mr. Rayhan Md. Faysal

Assistant Professor

Department of Civil engineering,

Daffodil International University

Nazibullah Hossain Shourov

ID: 171-47-341

Mohamed Abdirisak Said

ID: 173-47-606

Md. Kabirul Hasan

ID: 173-47-537

Tarakur Rahman

ID: 182-47-738

Yasir Hassan Mohamud

ID: 172-47-381

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DEDICATED

TO

ALMIGHTY ALLAH

&

OUR PARENTS

ABSTRACT

The study has been conducted in the Department of Civil Engineering of Daffodil International University, Bangladesh with the objective to prepare a project and thesis with a view to partial fulfillment of the requirements for the degree of Bachelor of Science (B.Sc.) in Civil Engineering. The specific objective of the study is the analysis of residential building considering lateral forces of a six storied structure then compare the deflection caused due to wind loads and earthquake loads and then turning the set structure into eight storied and comparing the differences design and compare the change in deflection in the eight storied structure due to wind load. It was observed that the design of the structure changed in the eight storied structure design and the per floor avg. rebar cost for beams increased by 39% for beams and for columns increased by 75%. Also, the dominating force in both structure was wind due to high exposure and the deflection due to wind was more in 8 storied structure, to be exact 21% in X direction and 134% in Y-direction. The basic methodology adopted here is the sequential presentation of analysis & design of all forces of six storied buildings for earthquake and wind effect by UBC 1994, BNBC 1993, ACI, and code 318R-05 code proposed places in Khulna, Bangladesh. Data & figures are presented wherever felt necessary in a reader-friendly way. Analysis and design have been performed with ETABS. The designs of the beam, column, and slab sections were done using AutoCAD.

LIST OF ACRONYMS & ABBREVIATION

P = Axial Load of Column

V = Shear Stress

h = Slab Thickness

b = Width of Beam

d = Effective Depth of Beam

a = Equivalent Depth of Beam

Pcf = Pound per Cubic feet

Psi = Pound per Square Inch

ASTM = American Standard for Testing Material

ACI = American Concrete Institute

BNBC = Bangladesh National Building Code

DDM = Direct Design Method

PWD = Public Work Department

RCC = Reinforcement Cement Concrete

USD = Ultimate Strength Design

UBC = Uniform Building Code

LL = live load

DL = Dead load

PW = partition wall

FF = Floor finish

WL = Wind load

WX= Wind load X-direction

WY= Wind load Y-direction

EQ= Earthquake

EQX= Earth quake X-direction

EQY= Earth quake Y-direction

EQL = Earth quake load

Ag= Gross area

Ast= Area of steel

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CHAPTER ONE: INTRODUCTION

1.1 INTRODUCTION:

It is well known that we as students are only taught only the basics of designing and load calculations during our study. But as the world is constantly evolving so do the design procedures and methods of calculations. In order to achieve a successful professional practice one must be well trained in special designing skills and codified procedures. To understand and keep knowledge of this rapid development and engage safely in innovative design, engineers need basic understanding of the new procedures through grounding in basic performance of concrete and steel structures and how they can be designed safely, economically and efficiently.

This knowledge creates a base foundation on new design procedures and familiarizes engineers with current design procedures.

1.2 BACKGROUND OF THE STUDY:

The world is rapidly evolving and the population of it is increasing day by day. To accommodate most people in one economic structure, maintaining the safety regulations is the greatest challenge a civil engineer has to face in this modern world.

Most of the modern economic structures are made of R.C.C (Reinforced cement concrete) to simplify design procedures. The main goal of a structural engineer is to design the structures in a way that the strength of the structure must be safe against collapse and serviceable in use. Safety requirements indicate that the structure must be adequate to support all loads that may act on it during the design period.

There are mostly two types of load a structural system has to carry:

- Vertical loads due to gravity (consists of dead loads and live loads).
- Lateral loads due to wind action on building sides and earthquake loads.

The wind forces differ a considerable amount with the location in a maximum of 100-year intervals. In Bangladesh, the annual average wind speed at 30 m height along the coastal belt is above 5 m/s. Wind speed in northeastern parts is above 4.5 m/s while inland wind speed is around

3.5 m/s for the most part. But during tropical cyclones (like sidr) the wind speed in the coastal area increases a significant amount. Sidr had an average wind speed of 215 km/h.

Wind loading competes with seismic loading as the dominant environmental loading for structures. Both of these forces produce an equal amount of damages to the structures in a long time period. Although large damaging earthquakes happen less often than severe wind storms. But many wind storms on earth are small and localized. In tropical oceans, most severe wind events occur (like cyclones, hurricanes, and typhoons). When these wind events make landfills in coastal areas the effects can be devastating. Also, earthquakes are responsible for the damages of a lot of structures. In Dhaka city, the effects of a huge earthquake will be devastating mostly in old towns as the structures there are old and closely erected to other structures. Damage to one structure has potential to damage the neighboring structures. Also in recent years, there is a trend of making skyscrapers in Dhaka city which if not designed properly can be a victim of a large disaster.

Fig1.2.1: Building collapse due to typhoon.

Fig1.2.2: Building collapse due to Earthquake

This study is an attempt to compare the effect of lateral forces on a residential building.

Structural engineering has been in use for ages, and one of the best ancient structures was the Pyramid of Giza that was constructed within the 26th century BC. The main structures during the medieval period were the pyramids since the form of the pyramids is essentially stable. Theoretical knowledge about the structures was limited, and construction techniques were supported by experience only. As a result, the produced structures were inefficient and expensive. But thanks to the rapid development of society and increase in population humans needed to adopt advanced technologies and construction methods to satisfy their got to be accommodated and therefore the consideration of advanced, economic, and safe structure came into existence.

Generally, a concrete structure is made of a set of frames consisting of several vertical and horizontal members. That's why it is also known as a frame structure. According to the definition of BNBC, there are 3 types of frame structures:

- Any building having the height less than 65 feet is called as low rise building.
- Any building having the height of 70-75 feet is called a medium rise building (typically 8 storied buildings).
- Any building having a height more than 75 feet is called a high rise building (typically more than 8 stories).

The structural design of a building is usually carried out considering the earthquake load and wind load.

1.3 OBJECTIVE:

The objectives of this project are:

- To analyze the effect of lateral forces on a residential building.
- To compare design between a similar structure with 6 floors and 8 floors.
- To find out dominating lateral force (i.e. weather wind force or earthquake force)
- To compare changes in deflection due to wind of similar structures after adding extra floors.
- To get basic knowledge in designing concrete structures using advanced ETABS software.

1.4 SCOPE OF STUDY:

The general scope of this study is to gain general knowledge about designing structures using advanced software such as ETABS. This study will also help to learn about the effects of lateral forces affecting the structures and compare between those forces.

We will also understand how extra floor loads affect the structure in design and understand the design codes better.

1.5 LIMITATIONS:

A detailed overall analysis of the structure could not be done due to the shortage of time. Only two beam sections from each floor were designed (one on the x-axis and one on the y-axis), only 3 columns were designed (the columns bearing the most load), only slab of GF was designed (the slab bearing the most moment). The design of stairs and footings was completely skipped.

The BNBC 1993 was used so the study may be outdated and old software was used (i.e., ETABS V.9.6.0) to comply with the recommendation of the BNBC 1993 code.

This study was conducted for a theoretical scenario so the dimensions of the structure were mostly unrealistic.

This study was conducted for residential buildings. A study of other structures was not conducted because it would create a large variation and cost time.

In this study, no cost estimation was done.

CHAPTER TWO: REVIEW OF LITERATURE

2.1 INTRODUCTION

There are sufficient studies presented by scholars, during which they performed detailed studies on wind and earthquake loads working on the structures and explained various sorts of methods, and used different software for the analysis of such buildings. The detailed description of such studies are presented below.

2.2 LITERATURE REVIEWS

Dr. K. R. C. Reddy and Sandip A. Tupat (2014) did a study on a comparative of wind and earthquake loads to decide the design loads of a multistoried building. The significance of the research is to estimate the design loads of a structure when subjected to wind and earthquake loads in every earthquake zone. The research design made use of an equivalent lateral load method for the calculation of the forces on the structures. The research considered the wind load as stochastic and time-dependent. It estimated wind load based on the design wind speed of that zone with a variation of 20%. He made the analysis on the low, medium, and high rise buildings. The wind forces are constant up to the third floor and have increased beyond the Third floor at a constant rate. The wind pressure increased as the height of the building increased. As the zone factor increases the earthquake forces also increase gradually. He concluded that wind loads are more critical than the earthquake loads. [1]

Khaled M. Heiza (2012) made a study on the effects of lateral Loads induced from wind and earthquakes in the design of reinforced concrete structures, especially for high–rise buildings. They made use of the Egyptian code of practice for calculating loads and forces on the High Rise building. They developed a computer program to analyze the structural behavior under wind and earthquake lateral loads acting on the structure. For performing analysis he took different heights of structures with different Floor weights, boundary conditions and also considered different seismic Zone factors and also different wind zones. A computer program has been developed to analyze the reinforced concrete buildings under the effect of wind and earthquake loads. The program calculates the flexibility of vertical members that resist wind and earthquake loads. He considered a twelve-story building for the analysis. It also calculates the center of mass and the center of rigidity of the building. Moments, lateral shear forces and the additional shear forces due to torsion on each vertical element resisting lateral load at each floor are also calculated. The wind is more effective than an earthquake for tall buildings when minimum design factors are considered, while an earthquake is found to be more effective for short Buildings. The wind effect increases rapidly when the height of the building Increases. The total shear force and the moment at the base result from seismic analysis when loads acting normal to the short side can be greater than the other direction. [2]

Hemil M .Chauhan, Manish Pomal, and Gyayak Bhuta (2013), presented a study on the comparative study of wind forces on high-rise buildings. For analysis, they used ETABS software with four terrain categories and six different wind Speeds. They performed both the analysis on 60m and 120m buildings. In static analysis, both buildings give almost the same values of shear forces & bending Moments. IS present code gives increased values of base shear compared to IS Draft code. IS Draft code gives more accurate and more direct than present code for estimating response parameters such as acceleration and Force. [3]

M.D. Kevadkar and P. B. Kodag (2013) have done studies of the susceptibility of the lateral Load forces on the structure. For this analysis, an R.C.C structure is modeled using the computer-aided program E-TABS to find out the effective lateral Load system during an earthquake. They modeled a 13 storied building for Carrying out the analysis using pushover analysis. They concluded that steel bracings reduce flexure and shear demands on beams and columns and Transfer the lateral load through axial load mechanism. The lateral displacement of the building is reduced by 40 to 60 % by the use of a shear Wall Type-III and X Type steel bracing system. Steel bracings can be used as an alternative to the other strengthening techniques available as the total Weight of the structure changes significantly. Shear wall has more story shear as compared to steel bracing but there is a 10 to 15% difference in lateral Displacement between shear wall and steel bracing. Shear wall and steel bracing increase the level of safety since the demand curve intersects near the elastic domain. The capacity of the steel braced structure is extra as compared to the shear wall structure. Steel bracing has more margin of Safety against collapse as compared with shear wall. According to M.D. Kevadkar and P. B. Kodag, the structure in high seismic areas may be admitted to the severe damage. Along with load due to gravity, the structure has to withstand lateral load which can develop high stresses. Nowadays, shear walls in R.C structure and steel bracings in steel structure are the most popular systems to resist lateral load due to earthquakes, wind, and blast. [4]

According to the researcher **Erdal, Irtem, Kaan Turker, and Umut Hasgul (2007)**, to make a realistic evaluation related to heavy damage and collapse reasons of reinforced concrete buildings during the severe earthquake using the Turkish Earthquake Code the SAP2000 Structural Analysis program is used Pushover analyses are carried out to determine the nonlinear behavior of the buildings under earthquake loads. To determine the performance of the building, displacements rather than forces are used. [5]

According to **Zahra, Tatheer, and Yasmeen Zehra (2012),** the existing building in Karachi and the susceptibility of high-rise buildings to severe earthquakes that may occur in the near future. As the earthquake zone has increased after the Bhuj earthquake the high-rise buildings are analyzed using the computer-aided software E-TABS. For carrying out the analysis of a building with a moment-resisting frame with a shear wall and a beam column moment resisting frame is considered for comparison of the high rise building. He clearly observed that the buildings in the higher seismic zone require a greater amount of steel to increase the stiffness of the members when subjected to higher seismic forces. The interior columns demand more steel reinforcement to support the higher seismic forces. The buildings in absence of a shear wall are more critical in resisting seismic forces during higher seismic activity and require quite a higher amount of reinforcement. [6]

Hirde, Suchita, and Vinay Magadum (2014) presented a study on the strictness of earthquake forces against wind forces for multistory RCC structure. The main purpose is to dissect the multistory structure set in wind zone VI and compare its performance to the structures set in zone V. The analysis is carried out using the software ETABS. They observed that the effect of both earthquake forces and wind forces on multistory structures increases with an increase in height of the structure. It's observed that base shear and story drift is less in the case of wind analysis for G 5 and G 10 structures whereas for G 15 and G 20 rearing it's other in the case of wind analysis. Effect of earthquake forces compared with the effect of wind forces on the performance of multistory structures set in seismic zone V and wind zone VI, earthquake is less effective than wind effect for towering structures. Since towering structures are more flexible and for short structures, earthquake is plant to be more effective. Hence the strictness of wind forces increases from medium- rise to high- rise structures than that of earthquake forces. The structure should be design for more severe weight to achieve safer design [7]

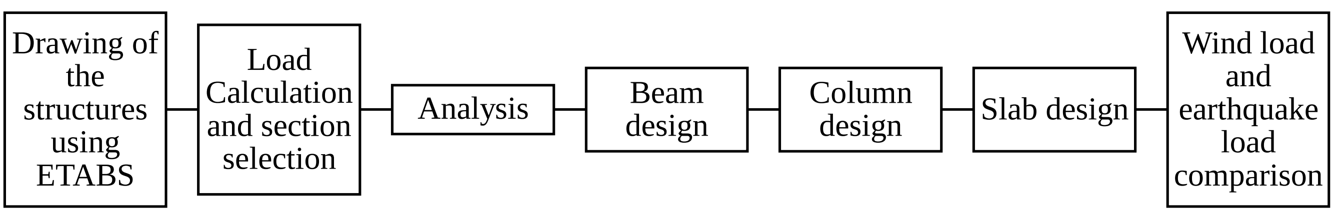
So, as there are no studies conducted for design a six storied structure in Khulna, Bangladesh using BNBC 1993 considering lateral loads in mind and comparing the deflection caused due to earthquake and wind load on the structure and comparing the effect of adding two extra stories in a structure we decided we will conduct this study to understand about the outcomes.

CHAPTER THREE: METHODOLOGY

3.1 METHODOLOGY:

A fictional six-storied edge-supported slab structure building was provided. Its plan and elevation were drawn. ETABS software was used for the analysis and design of the project. The calculation was done as per ACI, BNBC, and UBC codes and the portal frame method has been used for analysis using ETABS software and sections were selected. The wind load calculation on the structure was done as per BNBC 1993 and UBC 1994 (co-efficient, wind speed, etc.). The calculation was first done manually to get accurate loads on the building using ETABS the analysis was done and matched with the manually calculated wind loads. Earthquake load on the structure was calculated as per BNBC 1993 and UBC 1994 (co-efficient, seismic zone, etc.). The calculation was done mostly by ETABS. Then the beam, column and slab were designed. And at last the wind-load and earthquake-load effects were compared.

3.2 WORK FLOW CHART:



CHAPTER FOUR: ETABS MODEL AND SECTION DETAILS

4.1 ETABS MODEL DETAILS:

The structures were designed for "Khulna" where wind speed is 238km/h and Earthquake zone is Zone 1 with Z=0.075

Below the model for our 6 storied building is shown:

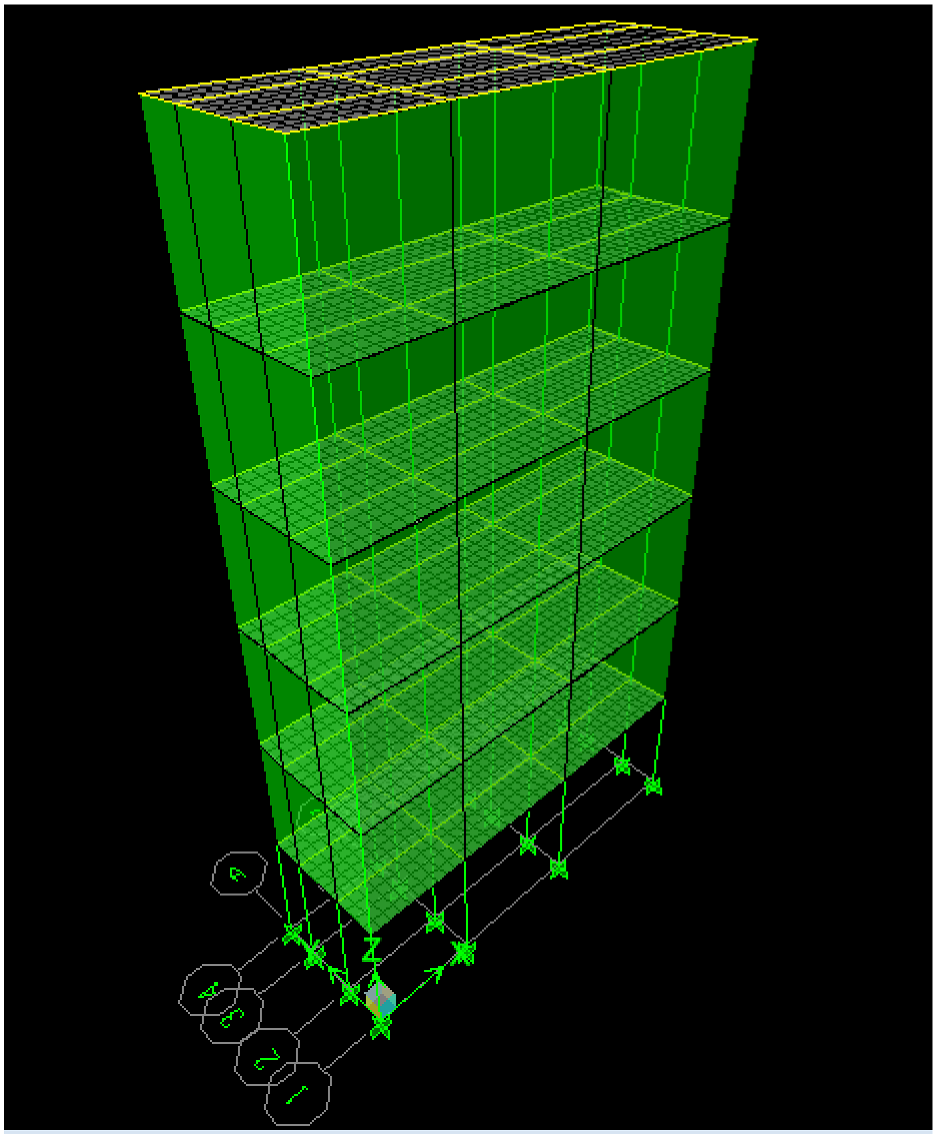


Fig4.1.1: ETABS model of 6 storied building

In the figure the distance between sections in Y-axis are:

1-2 is 1.25m, 2-3 is 1.5m and 3-4 is 1m

In the figure the distance between sections in X-axis are:

A-B is 3m, B-C is 3.75m and C-D is 4.5m

Below the 6 storied model with longitudinal rebar percentage is shown:

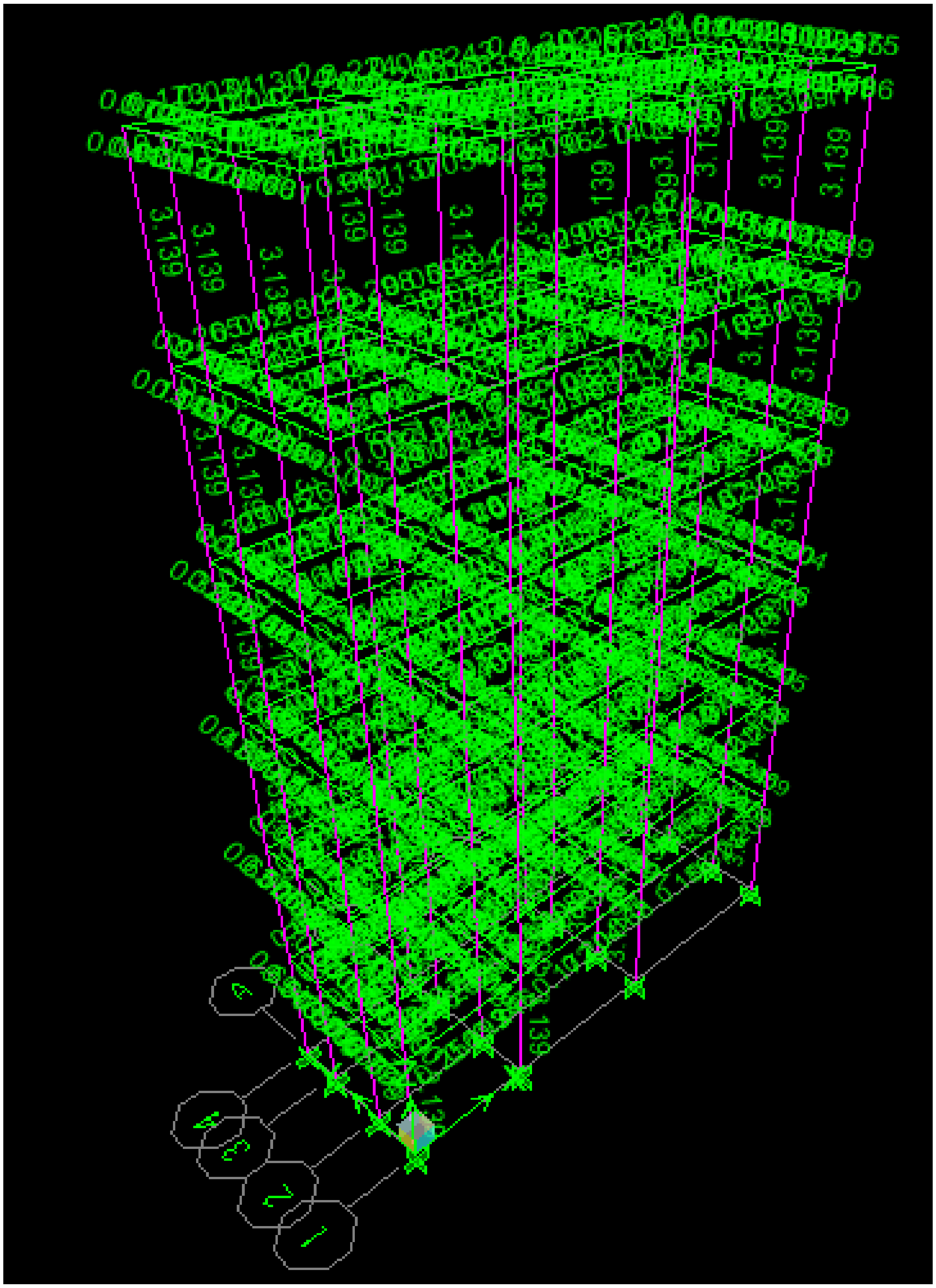


Fig4.1.2: ETABS model of 6 storied building with longitudinal reinforcement

Below the model for our 8 storied building which was designed by adding two extra floors on our old 6 storied building is shown:

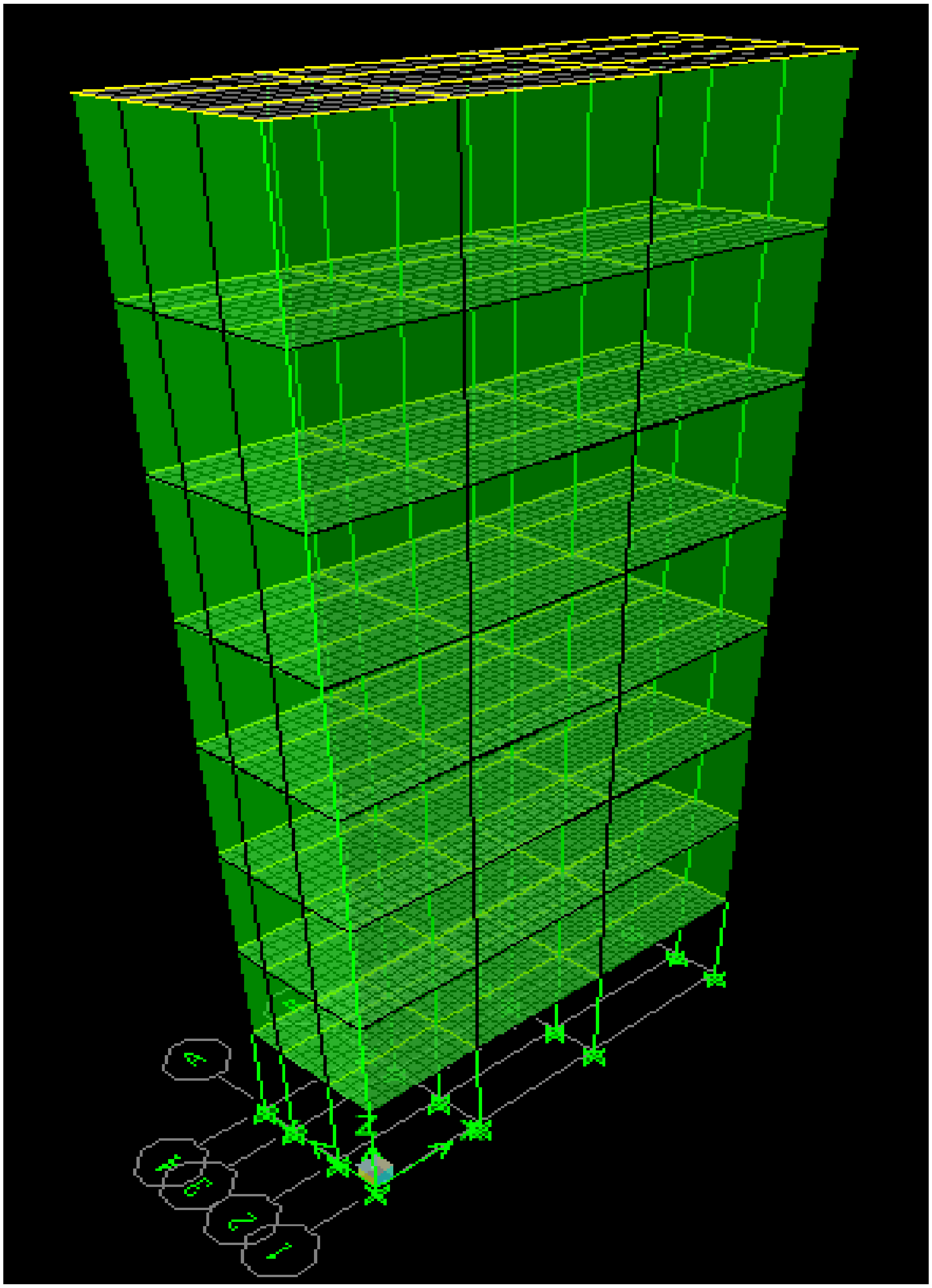


Fig4.1.3: ETABS model of 8 storied building

Below the 8 storied model with longitudinal rebar percentage is shown:

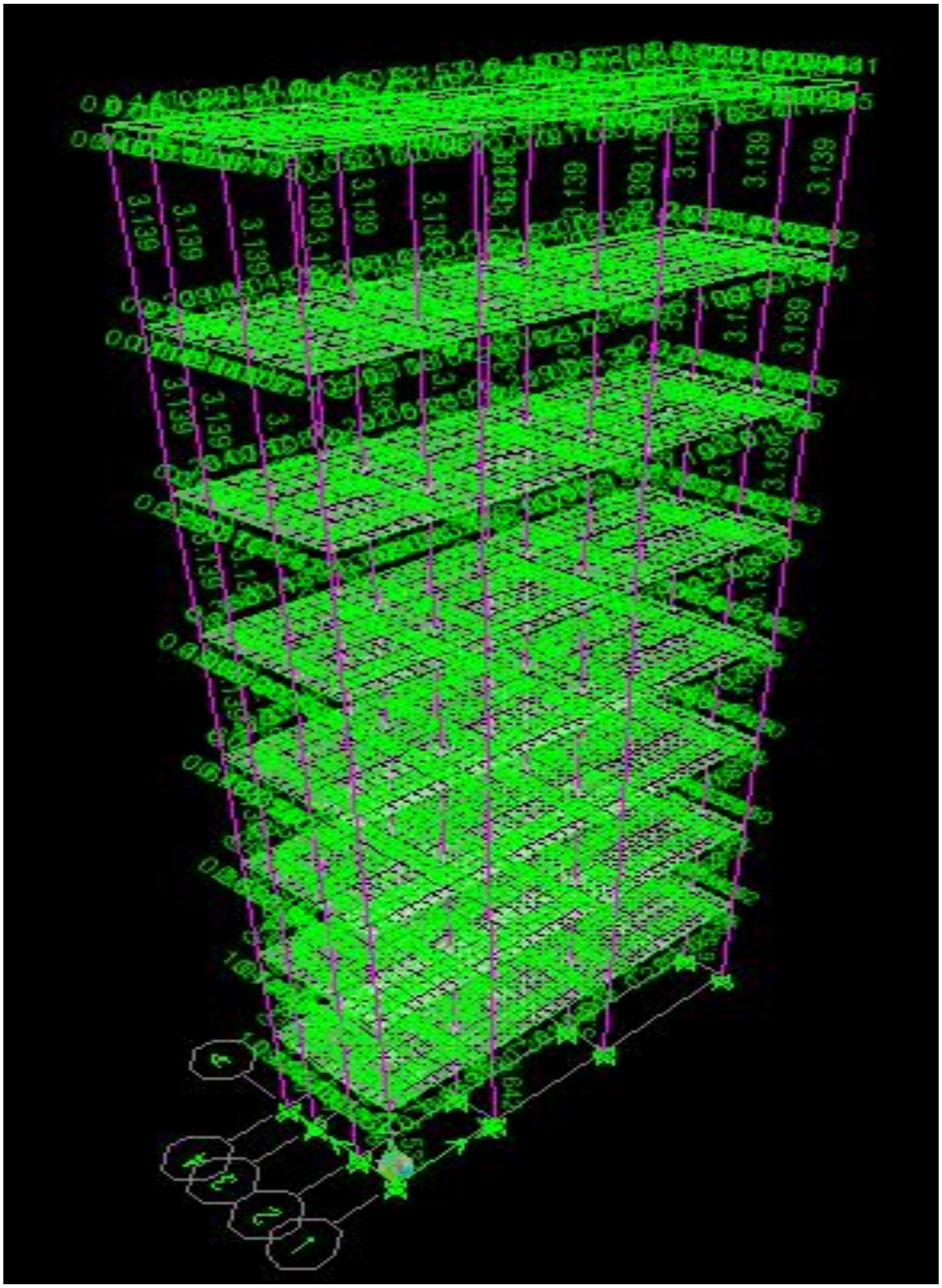


Fig4.1.4: ETABS model of 8 storied building with longitudinal reinforcement

4.2 LOADS APPLIED ON THE MODEL

4.2.1 NORMAL LOADS:

Here are the loads that were applied to the structure

Loads	
LL	3 KN/m^2
PW	2.5 KN/m^2
FF	1.5 KN/m^2

4.2.2 WIND LOADS:

Here are the calculations for wind load at our 6 storied structure:

From BNBC 1993 we get,

Exposure condition	Basic wind speed, V_b	Velocity to pressure co-efficient, C_c	Structural importance co-efficient C_I	Sustained wind pressure q_z	Gust co-efficient C_g	Pressure, P_{z-x}	Pressure, P_{z-y}
A	238 km/h	$47.2x10^{-6}$	1	2.6736 C_z KN/m^2	1.363	4.0968 C_z KN/m^2	5.3072 C_z KN/m^2
	147.918 mph						

Wind direction-X	Wind direction-Y
B= 3.75m	B= 11.25m
L= 11.25m	L= 3.75m
h= 17.5m	h= 17.5m
L/B= 3	L/B= 0.3
h/B= 4.6	h/B= 1.5
C_p= 1.10	C_p= 1.425

WL calculation for 6 storied structure: the calculation was done using BNBC 1993 (Section 2.4)

Table 4.2.2.1: WL calculation for 6 storied structure										
Wind pressure calculation			**X-dir**				**Y-dir**			
Floor	Height (m) from GL	Cz	P_{z-x} (KN/m²)	Area (m²)	Floor level force (KN)	Floor level force (kip)	P_{z-y} (KN/m²)	Area (m²)	Floor level force (KN)	Floor level force (kip)
1F	3.5	0.368	1.508	13.125	19.788	4.448	1.953	39.375	76.901	17.287
2F	7	0.442	1.811	13.125	23.767	5.343	2.346	39.375	92.365	20.764
3F	10.5	0.531	2.175	13.125	28.552	6.419	2.818	39.375	110.964	24.945
4F	14	0.604	2.474	13.125	32.477	7.301	3.206	39.375	126.218	28.374
Roof	17.5	0.668	2.737	6.563	17.959	4.037	3.545	19.688	69.796	15.690
			Total force along X direction			**27.548**	Total force along Y direction			**107.060**

Maximum allowable deflection for 6 floors:

h/500	**0.035 m**	**1.378 in**

Here are the calculations for wind load at our 8 storied structure:

From BNBC 1993 we get,

Exposure condition	Basic wind speed, V_b	Velocity to pressure co-efficient, C_c	Structural importance co-efficient C_I	Sustained wind pressure q_z	Gust co-efficient C_g	Pressure, P_{z-x}	Pressure, P_{z-y}
A	238 km/h	$47.2x10^{-6}$	1	2.6736 C_z KN/m^2	1.333	3.9844 C_z KN/m^2	5.0786 C_z KN/m^2
	147.918 mph						

Wind direction-X	Wind direction-Y
B= 3.75m	B= 11.25m
L= 11.25m	L= 3.75m
h= 25.5m	h= 25.5m
L/B= 3	L/B= 0.3
h/B= 6.8	h/B= 2.26
C_p= 1.118	C_p= 1.425

WL calculation for 8 storied structure: the calculation was done using BNBC 1993 (Section 2.4)

Table 4.2.2.2: WL calculation for 8 storied structure										
Wind pressure calculation			X-dir				Y-dir			
Floor	Height (m) from GL	Cz	P_{z-x} (KN/m^2)	Area (m^2)	Floor level force (KN)	Floor level force (kip)	P_{z-y} (KN/m^2)	Area (m^2)	Floor level force (KN)	Floor level force (kip)
1F	3.5	0.368	1.466	13.125	19.245	4.326	1.869	39.375	73.589	16.543
2F	7	0.442	1.761	13.125	23.115	5.196	2.245	39.375	88.387	19.869
3F	10.5	0.531	2.116	13.125	27.769	6.242	2.697	39.375	106.184	23.870
4F	14	0.604	2.407	13.125	31.586	7.101	3.067	39.375	120.782	27.152
5F	17.5	0.668	2.662	13.125	34.933	7.853	3.393	39.375	133.580	30.029
6F	21	0.725	2.889	13.125	37.914	8.523	3.682	39.375	144.978	32.591
Roof	25.5	0.783	3.120	6.563	20.474	4.602	3.977	19.688	78.288	17.599
			Total force along X direction			**43.844**	Total force along Y direction			**167.653**

Maximum allowable deflection for 8 floors:

h/500	**0.051 m**	**2.001 in**

4.2.3 EARTHQUAKE LOAD CALCULATIONS

Both of the structure was designed for Khulna. So, according to BNBC 1993:

Earth quake load		
Soil type	S3	S=1.5
Zone	Zone 1	Z=0.075
Importance factor	1	

EQL calculation: No manual calculation was done. ETABS was used to apply the load on the structure. The application process was similar to BNBC 1993 (Section 2.5).

4.3 SECTIONS USED:

Below the sections used in the 6 storied structure to support the loads are shown:

Column selections		
Name	Sign	Dimensions (W X D)
Corner columns	Cc	450mm X 450mm
Internal columns	Ci	450mm X 450mm
External columns (X-dir)	Cex	450mm X 450mm
External columns (Y-dir)	Cey	450mm X 450mm

Beam selections		
Name	Sign	Dimensions (W X D)
Internal beams (X-dir)	Bix	300mm X 350mm
Internal beams (Y-dir)	Biy	300mm X 300mm
External beams (X-dir)	Bex	300mm X 300mm
External beams (Y-dir)	Bey	300mm X 300mm

Material properties	**Slab thickness**
Fc= 3.5ksi	Membrane= 150mm
Fy= 60ksi	Bending= 125mm

After the addition of two extra floors at our 6 storied structure it was noticed that many of our beams at X-direction failed. Below are the figures of the floors on which the beams failed:

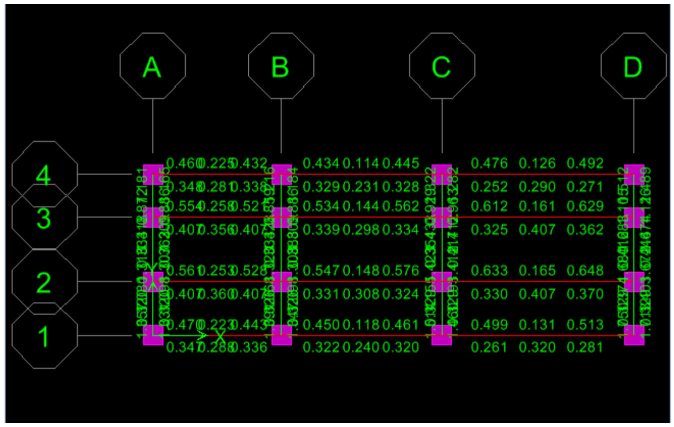


Fig4.3.1: Beam failures of GF

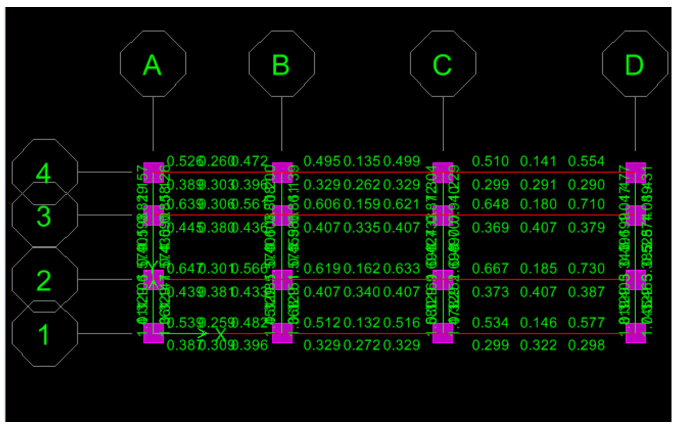


Fig4.3.2: Beam failures of 1F

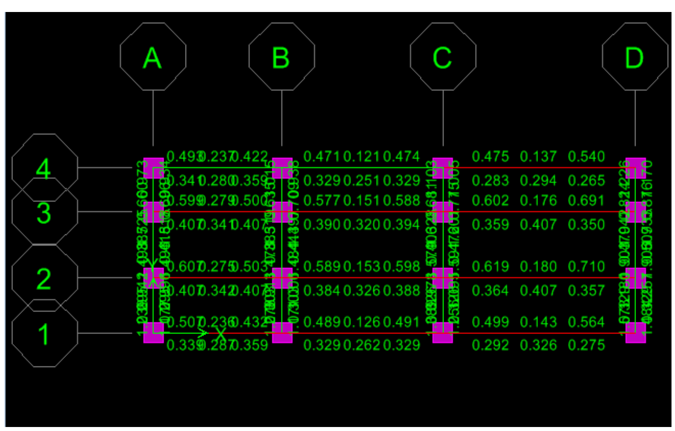


Fig4.3.3: Beam failures of 2F

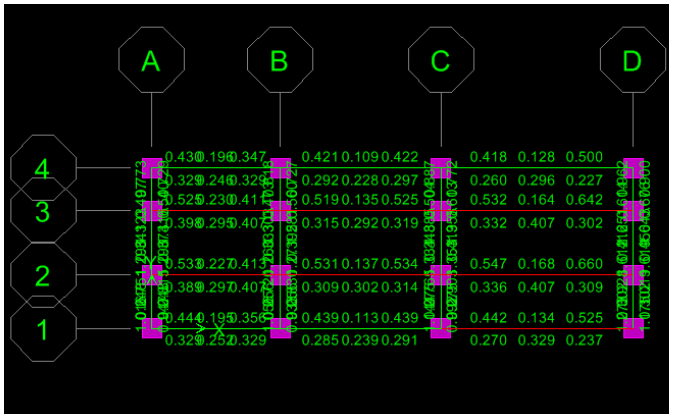


Fig4.3.4: Beam failures of 3F

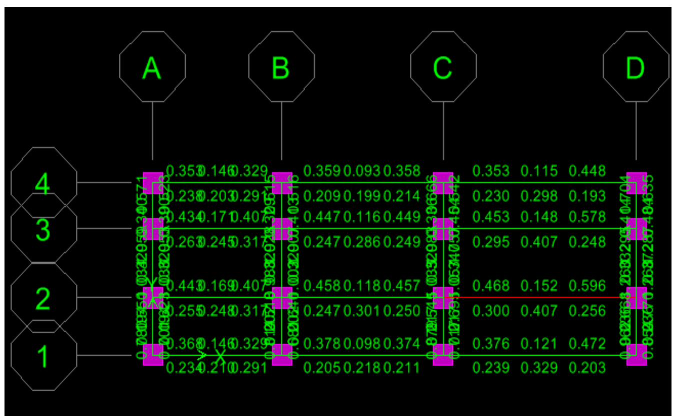


Fig4.3.5: Beam failures of 4F

This occurred due to O/S #45 Shear stress due to shear force and torsion together exceeds maximum allowed. There are many ways to solve this, but we increased the sections of internal and external Beams at X-direction.

Below the new sections used in the 8 storied structure to support the loads are shown:

Column selections		
Name	Sign	Dimentions (W X D)
Corner columns	Cc	450mm X 450mm
Internal columns	Ci	450mm X 450mm
External columns (X-dir)	Cex	450mm X 450mm
External columns (Y-dir)	Cey	450mm X 450mm

Beam selections		
Name	Sign	Dimentions (W X D)
Internal beams (X-dir)	Bix	400mm X 450mm
Internal beams (Y-dir)	Biy	300mm X 300mm
External beams (X-dir)	Bex	300mm X 450mm
External beams (Y-dir)	Bey	300mm X 300mm

Material properties	Slab thickness
Fc= 3.5ksi	Membrane= 150mm
Fy= 60ksi	Bending= 125mm

NOTE: Clear cover to rebar Centre in both 6 floor and 8 floor structure for beam was 87.5mm and for column was 62.5mm.

CHAPTER FIVE: DESIGN OF BEAM SECTIONS

5.1 CALCULATIONS:

For time constraints only two beam sections from every floor of both structures were designed in this study. One in the X-axis and One in the Y-axis.

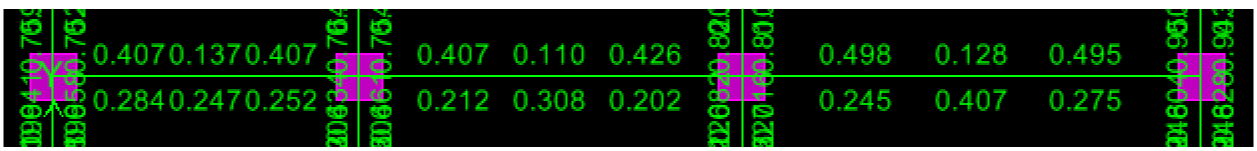
The beams were selected based on the highest reinforcement requirement.

Below the Calculations for 6storied structure are shown:

(**Note:** For design of beam sections Bending m11 Modifier, Bending m22 Modifier, Bending m12 Modifier was taken as 1 instead of 0.00000001)

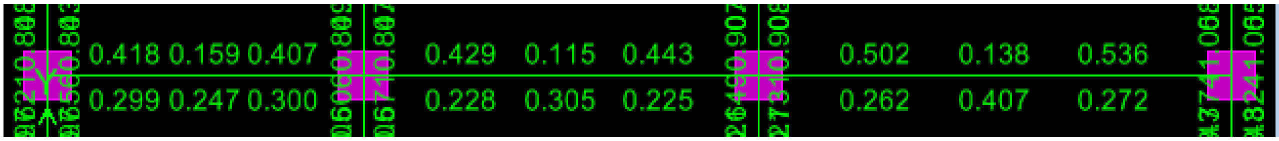
CALCULATION FOR X-AXIS OF 6 STORIED STRUCTURE:

Table 5.1.1: X-axis GF #2 beam:



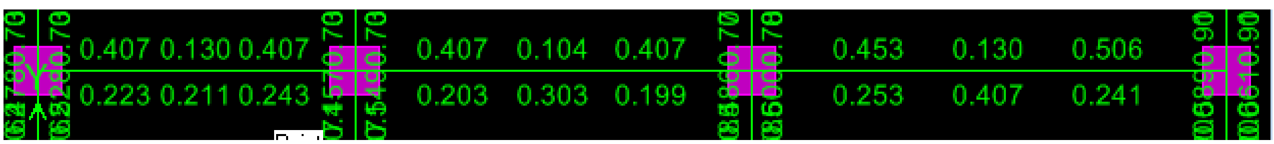
X-axis GF #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.498	4	0.2	3	Left	0.284	4	0.2	2
Middle	0.137	4	0.2	2	Middle	0.407	4	0.2	3
Right	0.495	4	0.2	3	Right	0.275	4	0.2	2

Table 5.1.2 X-axis 1F #2 beam:



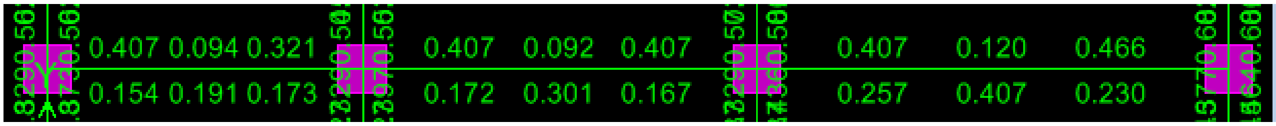
X-axis 1F #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.502	4	0.2	3	Left	0.299	4	0.2	2
Middle	0.159	4	0.2	2	Middle	0.407	4	0.2	3
Right	0.536	4	0.2	3	Right	0.3	4	0.2	2

Table 5.1.3 X-axis 2F #2 beam:



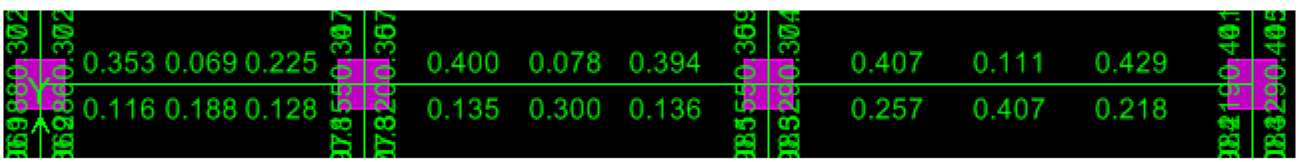
X-axis 2F #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.453	4	0.2	3	Left	0.253	4	0.2	2
Middle	0.13	4	0.2	2	Middle	0.407	4	0.2	3
Right	0.506	4	0.2	3	Right	0.243	4	0.2	2

Table 5.1.4 X-axis 3F #2 beam:



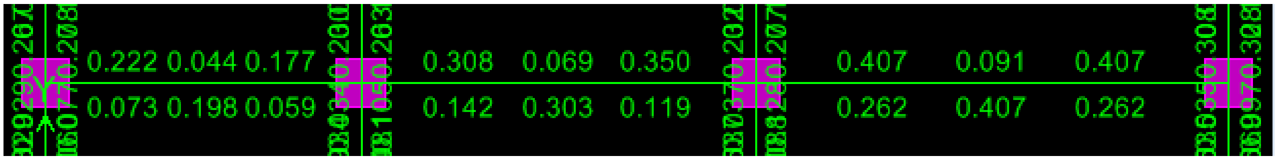
X-axis 3F #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.407	4	0.2	3	Left	0.257	4	0.2	2
Middle	0.12	4	0.2	2	Middle	0.407	4	0.2	3
Right	0.466	4	0.2	3	Right	0.23	4	0.2	2

Table 5.1.5 X-axis 4F #2 beam:



X-axis 4F #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.407	4	0.2	3	Left	0.257	4	0.2	2
Middle	0.111	4	0.2	2	Middle	0.407	4	0.2	3
Right	0.429	4	0.2	3	Right	0.218	4	0.2	2

Table 5.1.6 X-axis ROOF #2 beam:



X-axis ROOF #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.407	4	0.2	3	Left	0.262	4	0.2	2
Middle	0.091	4	0.2	2	Middle	0.407	4	0.2	3
Right	0.407	4	0.2	3	Right	0.262	4	0.2	2

Stirrups design:

(Note: value of shear reinforcement was very low so minimum shear reinforcement was provided)

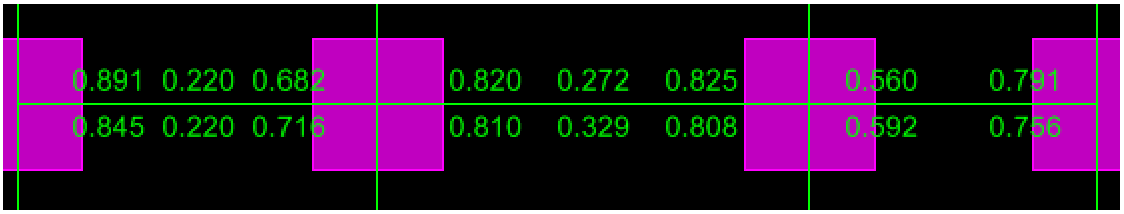
We use #3 bars as stirrups. According to SMRF:

Spacing of stirrups for first and last L/3 is d/4= (13.8-3.48)/4= 2.5in c/c

For middle part spacing of stirrups are d/2= 5in c/c

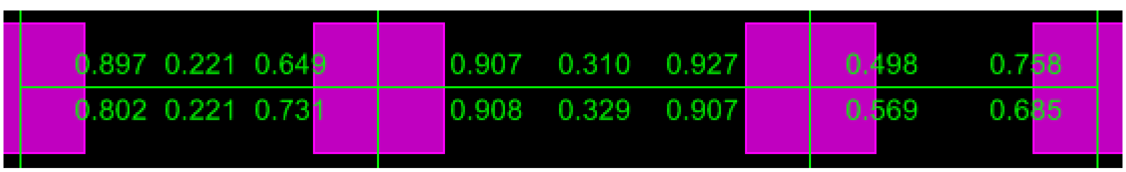
CALCULATION FOR Y-AXIS OF 6 STORIED STRUCTURE:

Table 5.1.7 Y-axis GF #C beam:



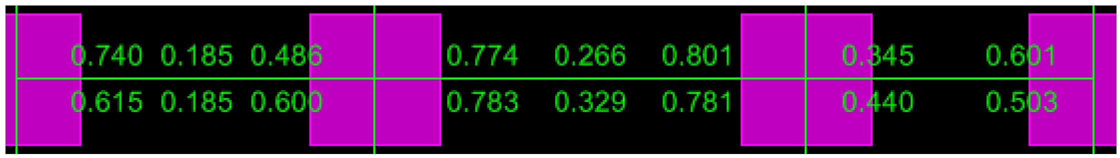
Y-axis GF #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.891	5	0.31	3	Left	0.845	5	0.31	3
Middle	0.272	5	0.31	2	Middle	0.329	5	0.31	3
Right	0.825	5	0.31	3	Right	0.808	5	0.31	3

Table 5.1.8 Y-axis 1F #C beam:



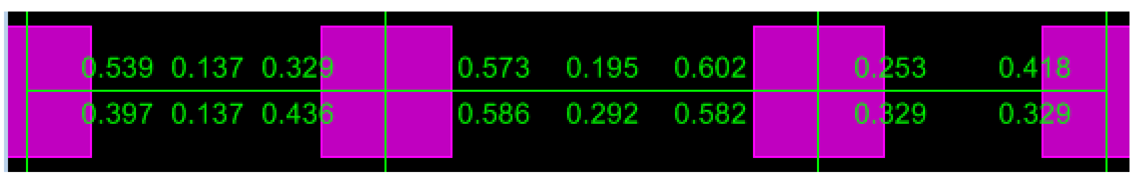
Y-axis 1F #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.907	5	0.31	3	Left	0.908	5	0.31	3
Middle	0.31	5	0.31	2	Middle	0.329	5	0.31	3
Right	0.927	5	0.31	3	Right	0.907	5	0.31	3

Table 5.1.9 Y-axis 2F #C beam:



Y-axis 2F #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.774	5	0.31	3	Left	0.783	5	0.31	3
Middle	0.266	5	0.31	2	Middle	0.329	5	0.31	3
Right	0.801	5	0.31	3	Right	0.781	5	0.31	3

Table 5.1.10 Y-axis 3F #C beam:

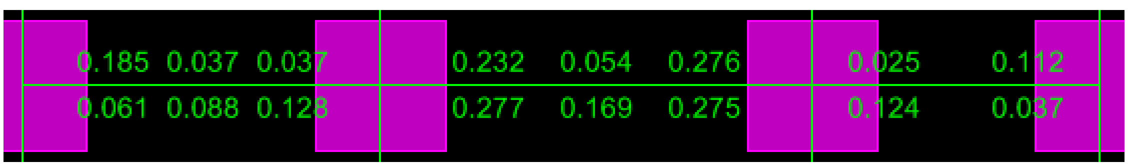


Y-axis 3F #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.573	5	0.31	2	Left	0.586	5	0.31	2
Middle	0.195	5	0.31	2	Middle	0.292	5	0.31	2
Right	0.602	5	0.31	2	Right	0.582	5	0.31	2

Table 5.1.11 Y-axis 4F #C beam:

Y-axis 4F #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.359	4	0.2	2	Left	0.374	4	0.2	2
Middle	0.116	4	0.2	2	Middle	0.211	4	0.2	2
Right	0.387	4	0.2	2	Right	0.365	4	0.2	2

Table 5.1.12 Y-axis ROOF #C beam:



Y-axis ROOF #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.232	4	0.2	2	Left	0.277	4	0.2	2
Middle	0.054	4	0.2	2	Middle	0.169	4	0.2	2
Right	0.276	4	0.2	2	Right	0.275	4	0.2	2

Stirrups design:

(Note: value of shear reinforcement was very low so minimum shear reinforcement was provided)

We use #3 bars as stirrups. According to SMRF:

Spacing of stirrups for first and last L/3 is d/4= (11.81-3.48)/4= 2in c/c

For middle part spacing of stirrups are d/2= 4in c/c

CALCULATION FOR X-AXIS OF 8 STORIED STRUCTURE:

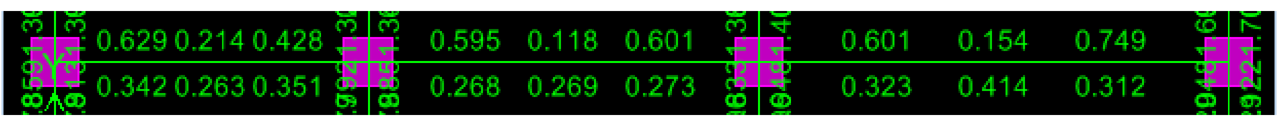
Table 5.1.13 X-axis GF #2 beam:

X-axis GF-2.0 #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.713	5	0.31	3	Left	0.456	5	0.31	2
Middle	0.227	5	0.31	2	Middle	0.425	5	0.31	2
Right	0.742	5	0.31	3	Right	0.379	5	0.31	2

Table 5.1.14 X-axis 1F #2 beam:

X-axis 1F-2.0 #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.683	5	0.31	3	Left	0.43	5	0.31	2
Middle	0.242	5	0.31	2	Middle	0.414	5	0.31	2
Right	0.749	5	0.31	3	Right	0.401	5	0.31	2

Table 5.1.15 X-axis 2F #2 beam:

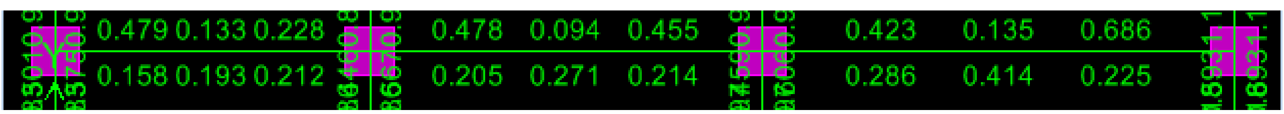


X-axis 2F-2.0 #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.629	5	0.31	3	Left	0.342	4	0.2	2
Middle	0.214	5	0.31	2	Middle	0.414	4	0.2	3
Right	0.749	5	0.31	3	Right	0.351	4	0.2	2

Table 5.1.16 X-axis 3F #2 beam:

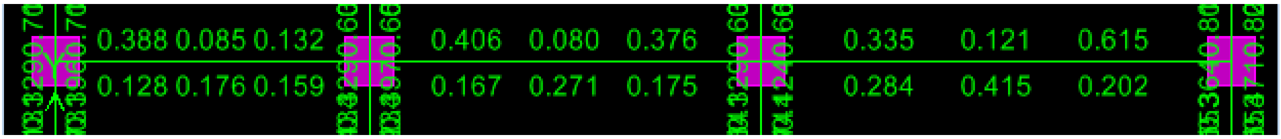
X-axis 3F-2.0 #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.561	5	0.31	3	Left	0.308	4	0.2	2
Middle	0.176	5	0.31	2	Middle	0.414	4	0.2	3
Right	0.744	5	0.31	3	Right	0.286	4	0.2	2

Table 5.1.17 X-axis 4F #2 beam:



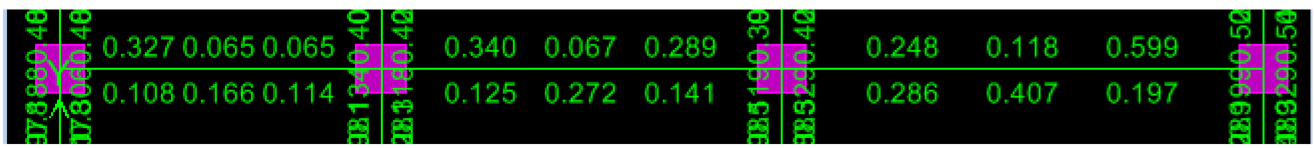
X-axis 4F-2.0 #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.479	5	0.31	3	Left	0.286	4	0.2	2
Middle	0.135	5	0.31	2	Middle	0.414	4	0.2	3
Right	0.686	5	0.31	3	Right	0.225	4	0.2	2

Table 5.1.18 X-axis 5F #2 beam:



X-axis 5F-2.0 #2 beam										
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.406	5	0.31	2	Left	0.284	4	0.2	2
Middle	0.121	5	0.31	2	Middle	0.415	4	0.2	3
Right	0.615	5	0.31	2	Right	0.202	4	0.2	2

Table 5.1.19 X-axis 6F #2 beam:



X-axis 6F-2.0 #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.34	5	0.31	2	Left	0.286	4	0.2	2
Middle	0.118	5	0.31	2	Middle	0.407	4	0.2	3
Right	0.599	5	0.31	2	Right	0.197	4	0.2	2

Table 5.1.20 X-axis ROOF #2 beam:

X-axis ROOF-2.0 #2 beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.264	4	0.2	3	Left	0.287	4	0.2	2
Middle	0.082	4	0.2	2	Middle	0.447	4	0.2	3
Right	0.416	4	0.2	3	Right	0.231	4	0.2	2

Stirrups design:

(Note: value of shear reinforcement was very low so minimum shear reinforcement was provided)

We use #3 bars as stirrups. According to SMRF:

Spacing of stirrups for first and last L/3 is d/4= (11.81-3.48)/4= 4in c/c

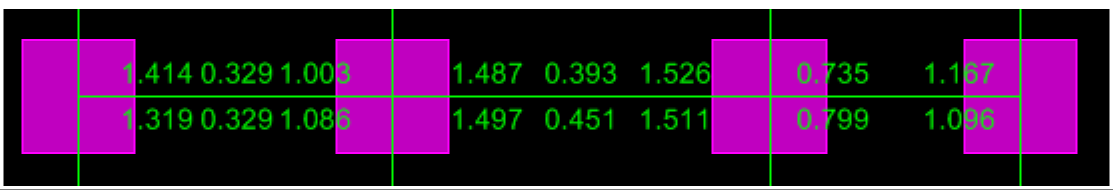
For middle part spacing of stirrups are d/2= 7in c/c

CALCULATION FOR Y-AXIS OF 8 STORIED STRUCTURE:

Table 5.1.21 Y-axis GF #C beam:

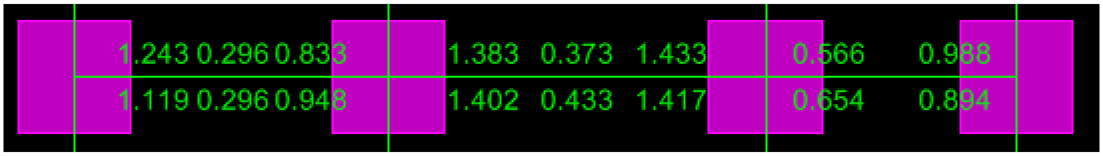
Y-axis GF-2.0 #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	1.374	5	0.31	5	Left	1.325	6	0.44	4
Middle	0.33	5	0.31	2	Middle	0.384	6	0.44	4
Right	1.273	5	0.31	5	Right	1.261	6	0.44	4

Table 5.1.22 Y-axis 1F #C beam:



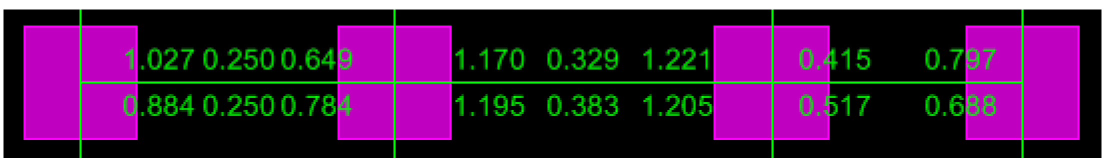
Y-axis 1F-2.0 #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	1.487	5	0.31	5	Left	1.497	6	0.44	4
Middle	0.393	5	0.31	2	Middle	0.451	6	0.44	4
Right	1.526	5	0.31	5	Right	1.511	6	0.44	4

TABLE 5.1.23 Y-AXIS 2F #C BEAM:



Y-axis 2F-2.0 #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	1.383	5	0.31	5	Left	1.402	6	0.44	4
Middle	0.373	5	0.31	2	Middle	0.433	6	0.44	4
Right	1.433	5	0.31	5	Right	1.417	6	0.44	4

TABLE 5.1.24 Y-AXIS 3F #C BEAM:

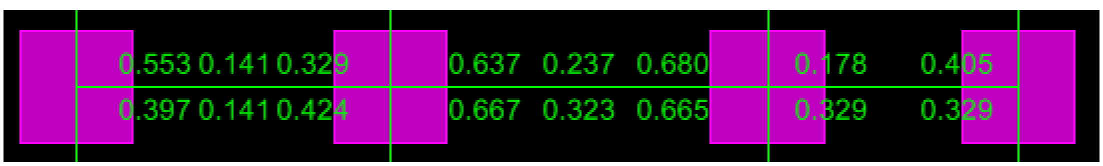


Y-axis 3F-2.0 #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	1.17	5	0.31	4	Left	1.195	5	0.31	4
Middle	0.329	5	0.31	2	Middle	0.383	5	0.31	4
Right	1.221	5	0.31	4	Right	1.205	5	0.31	4

Table 5.1.25 Y-axis 4F #C beam:

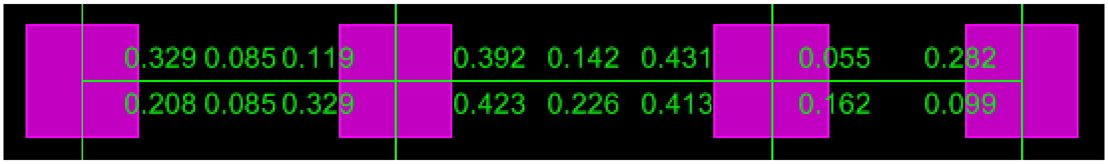
Y-axis 4F-2.0 #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.908	5	0.31	4	Left	0.936	4	0.31	4
Middle	0.329	5	0.31	2	Middle	0.329	4	0.31	4
Right	0.956	5	0.31	4	Right	0.94	4	0.31	4

TABLE 5.1.26 Y-AXIS 5F #C BEAM:



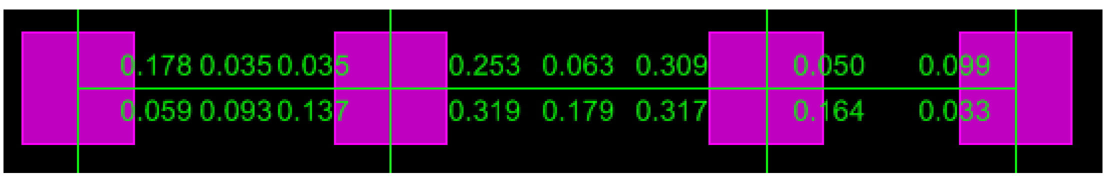
Y-axis 5F-2.0 #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.637	4	0.2	4	Left	0.667	4	0.2	4
Middle	0.237	4	0.2	2	Middle	0.323	4	0.2	4
Right	0.68	4	0.2	4	Right	0.665	4	0.2	4

TABLE 5.1.27 Y-AXIS 6F #C BEAM:



Y-axis 6F-2.0 #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.392	4	0.2	3	Left	0.423	4	0.2	3
Middle	0.142	4	0.2	2	Middle	0.226	4	0.2	3
Right	0.431	4	0.2	3	Right	0.413	4	0.2	3

TABLE 5.1.28 Y-AXIS ROOF #C BEAM:



Y-axis ROOF-2.0 #C beam									
Maximum reinforcement needed		Bar selected to be used		Number of Bar provided	Maximum reinforcement needed		Bar selected to be used		Number of Bar provided
Negative side(in^2)		No.	Area(in^2)		Positive side(in^2)		No.	Area(in^2)	
Left	0.253	4	0.2	2	Left	0.319	4	0.2	2
Middle	0.063	4	0.2	2	Middle	0.179	4	0.2	2
Right	0.309	4	0.2	2	Right	0.317	4	0.2	2

Stirrups design:

(Note: value of shear reinforcement was very low so minimum shear reinforcement was provided)

We use #3 bars as stirrups. According to SMRF:

Spacing of stirrups for first and last L/3 is d/4= (11.81-3.48)/4= 3in c/c

For middle part spacing of stirrups are d/2= 6in c/c

5.2 FIGURES OF THE BEAM SECTIONS: BEAM FIGURES FOR 6 STORIED STRUCTURE.

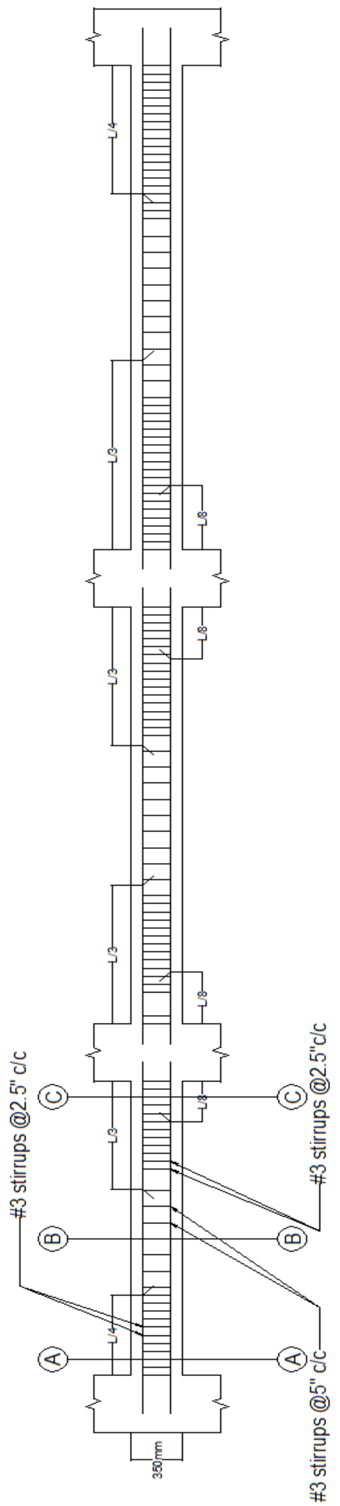


Fig 5.2.1: Beam rod detailing X-axis GF-4F

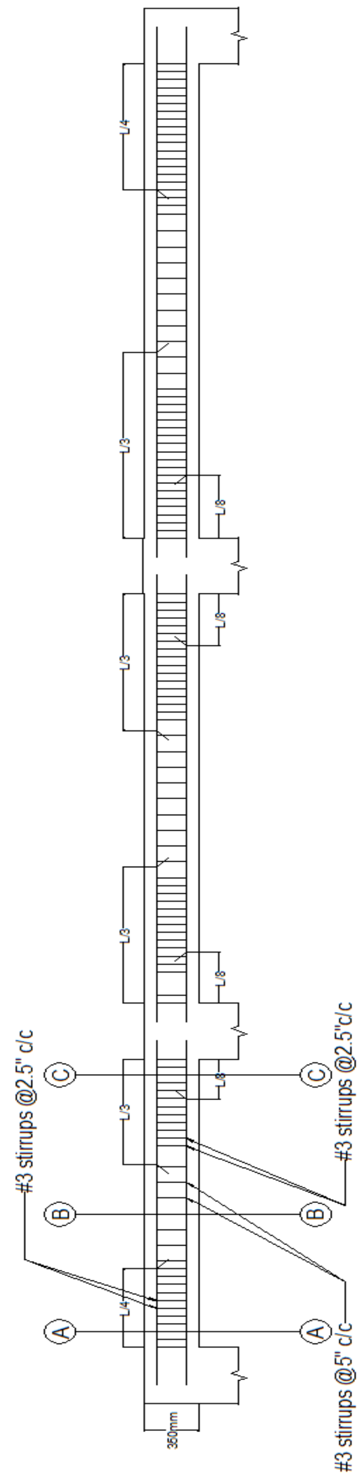


Fig 5.2.2: Beam rod detailing for X-axis ROOF

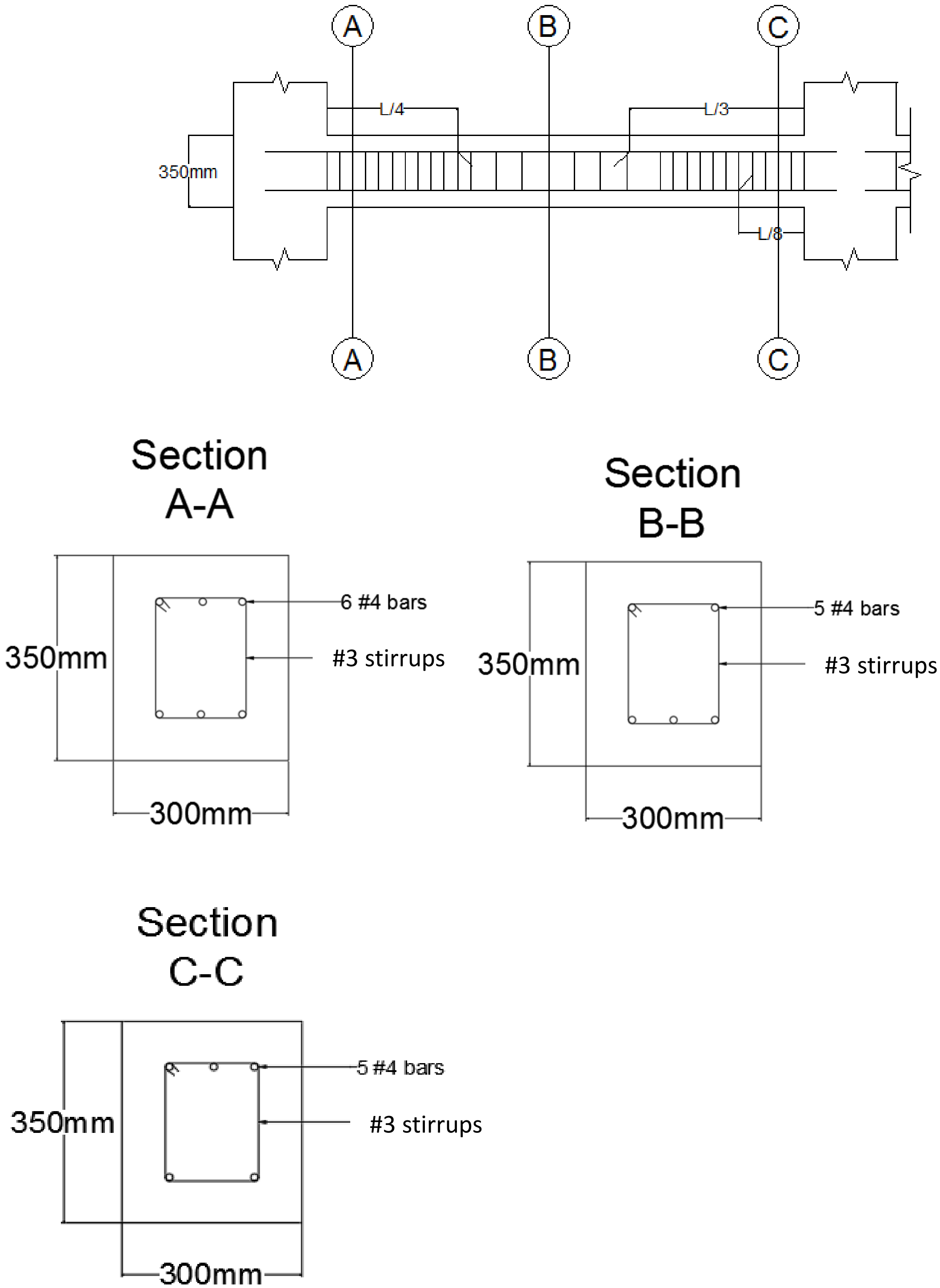


Fig 5.2.3: Beam Cross section for X-axis GF-ROOF

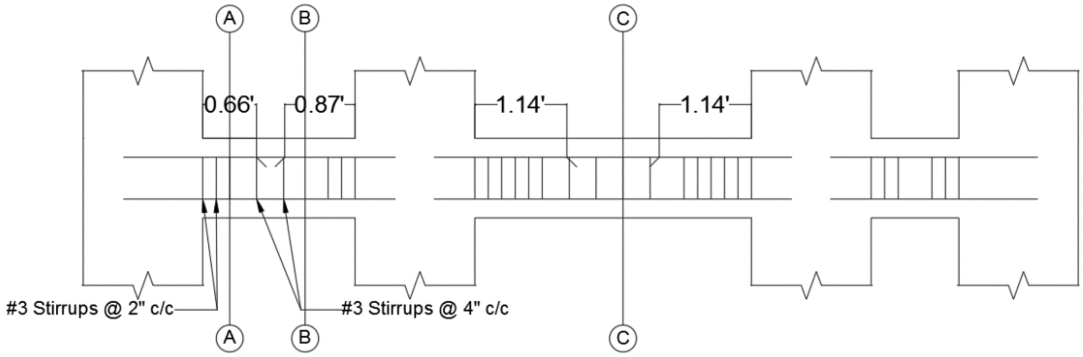
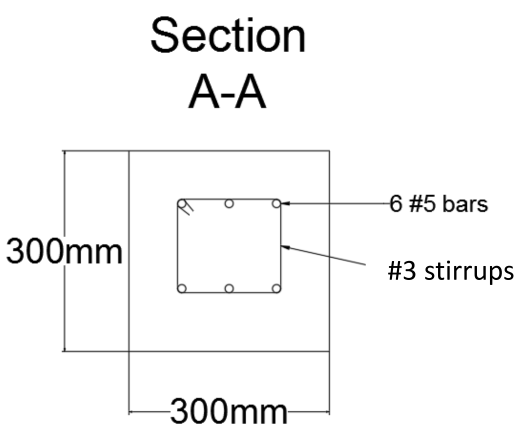
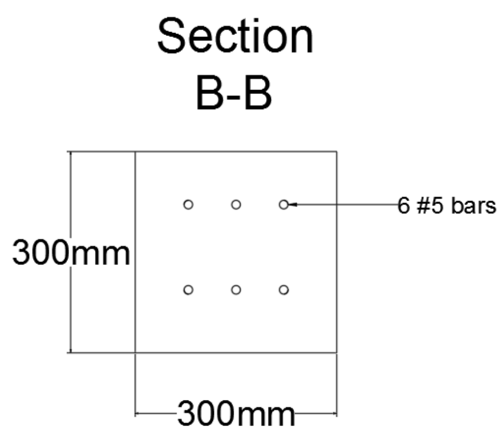


Fig 5.2.4: Beam rod detailing for Y-axis GF-2F





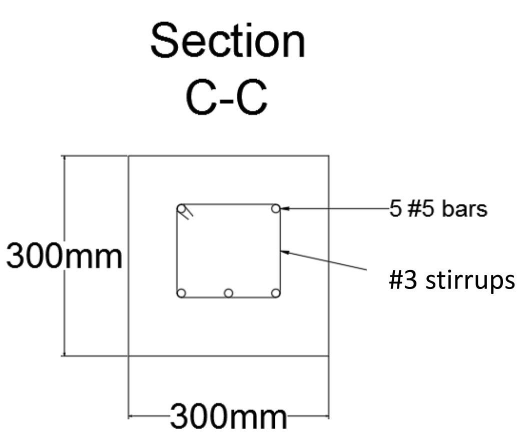


Fig 5.2.5: Beam Cross-sections for Y-axis GF-2F

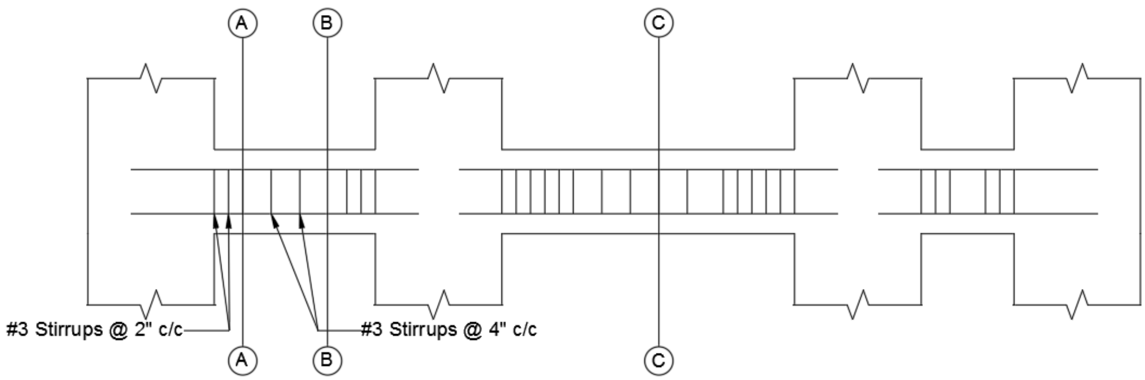


Fig 5.2.6: Beam rod detailing for Y-axis 3F

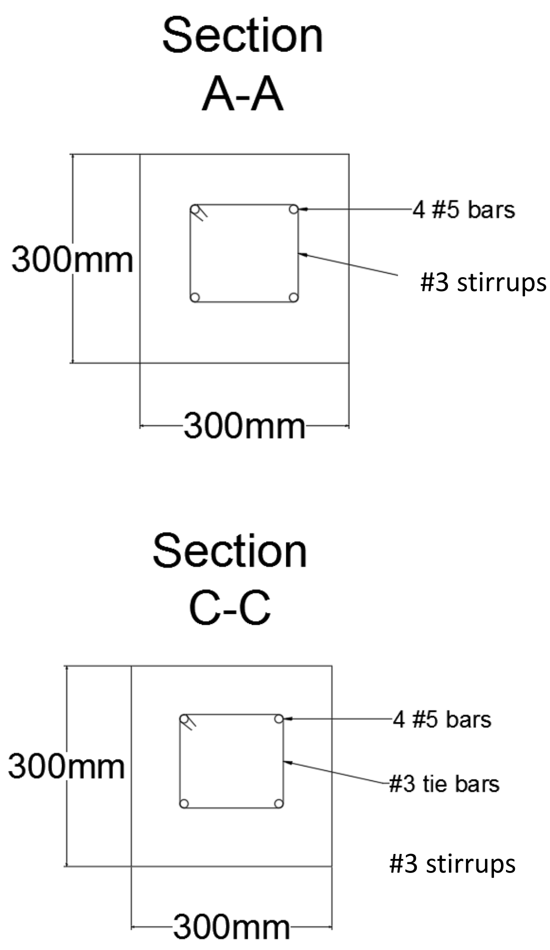


Fig 5.2.7: Beam Cross-sections for Y-axis 3F

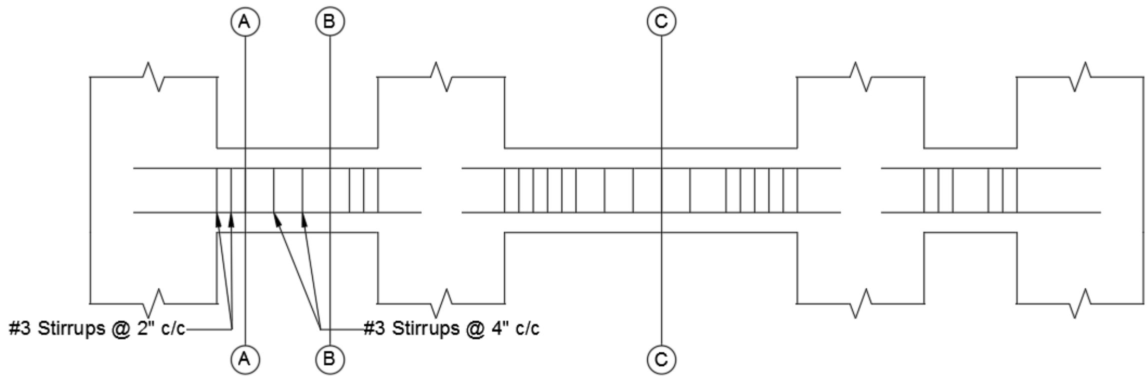


Fig 5.2.8: Beam rod detailing for Y-axis 4F

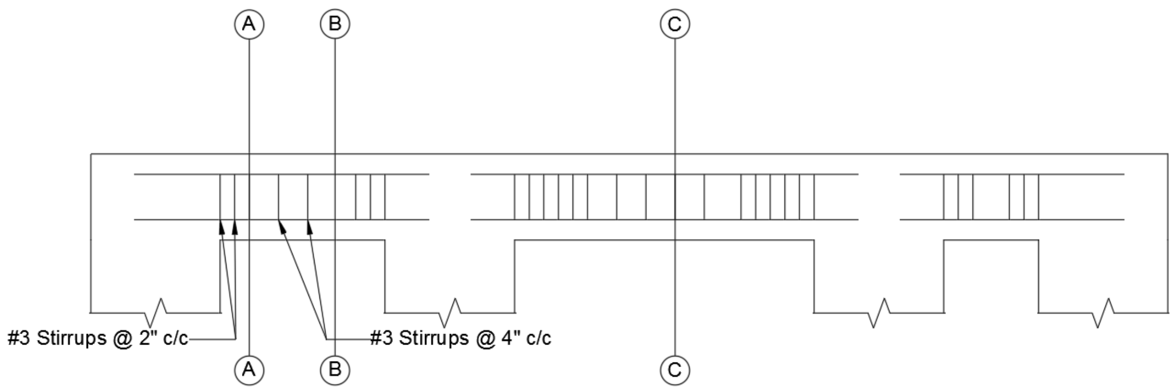


Fig 5.2.9: Beam rod detailing for Y-axis ROOF

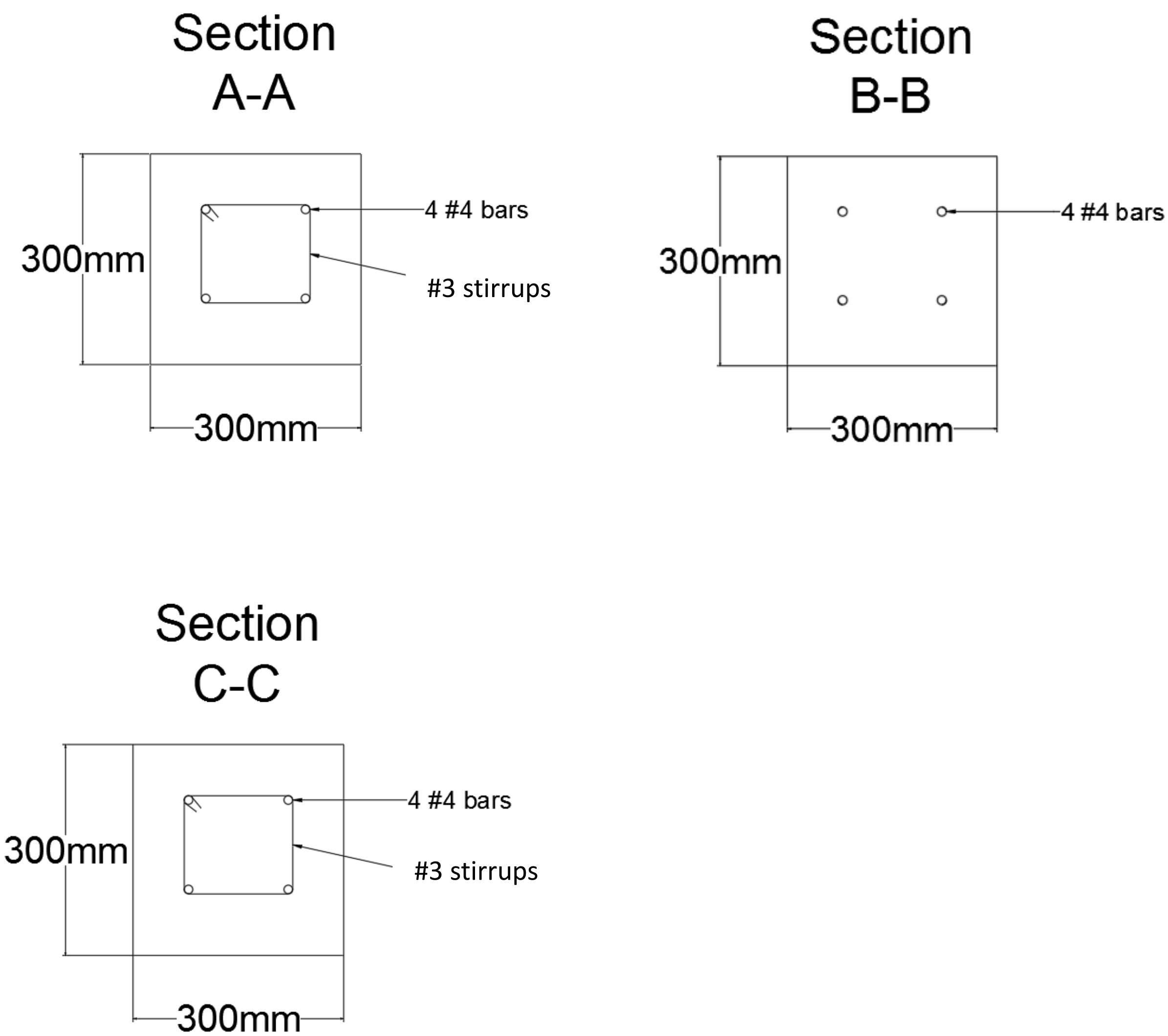


Fig 5.2.10: Beam Cross-sections for Y-axis 4F-ROOF

BEAM FIGURES FOR 8 STORIED STRUCTURE.

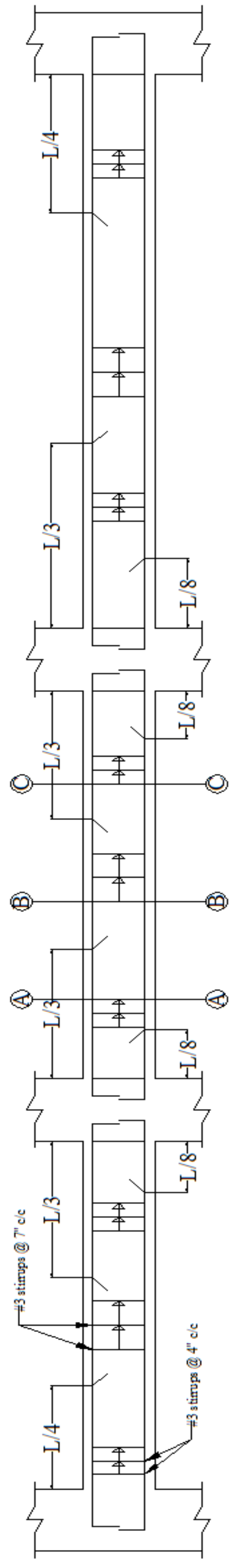


Fig 5.2.11: Beam rod detailing X-axis

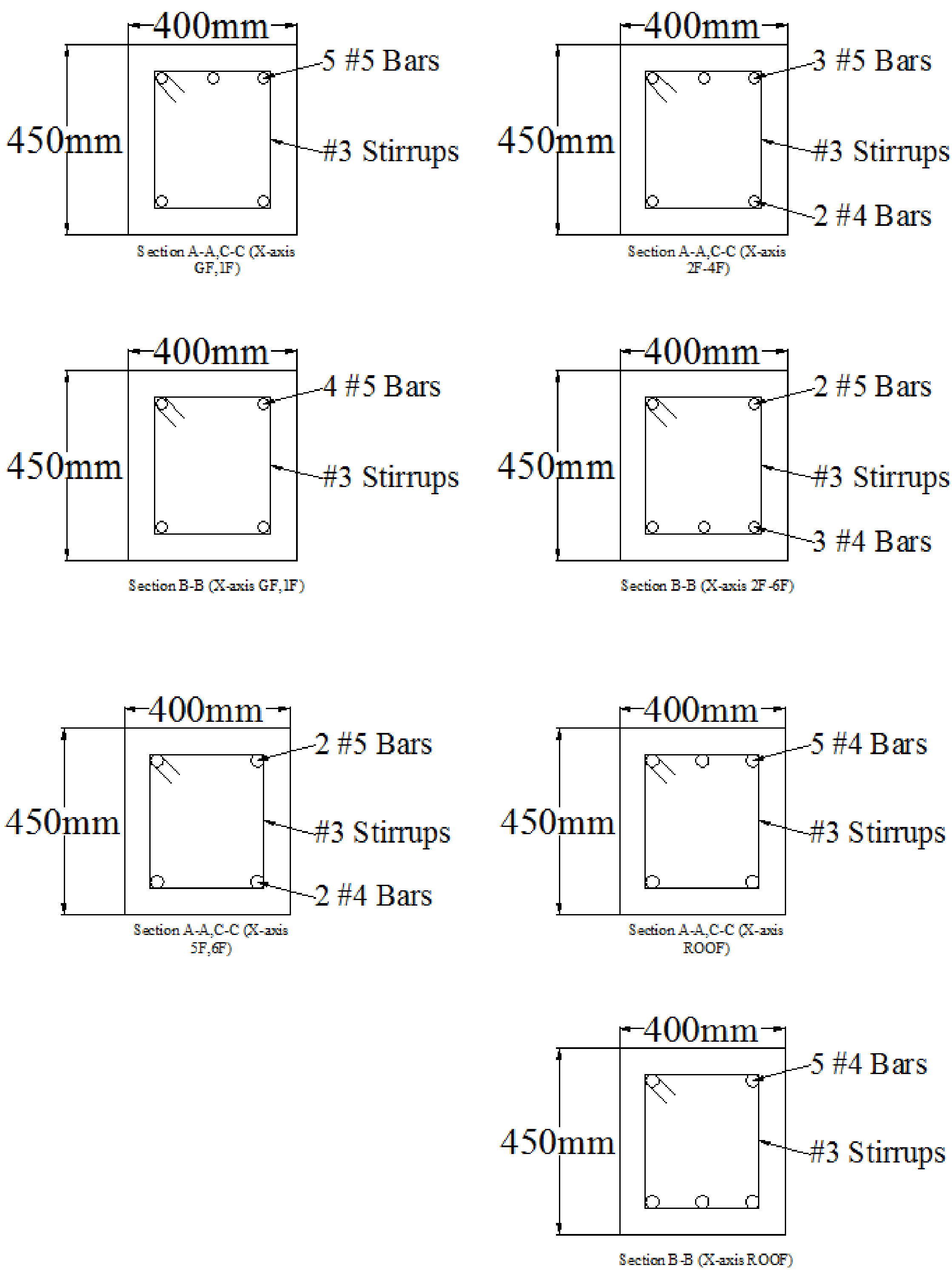


Fig 5.2.12: Beam Cross-sections for X-axis

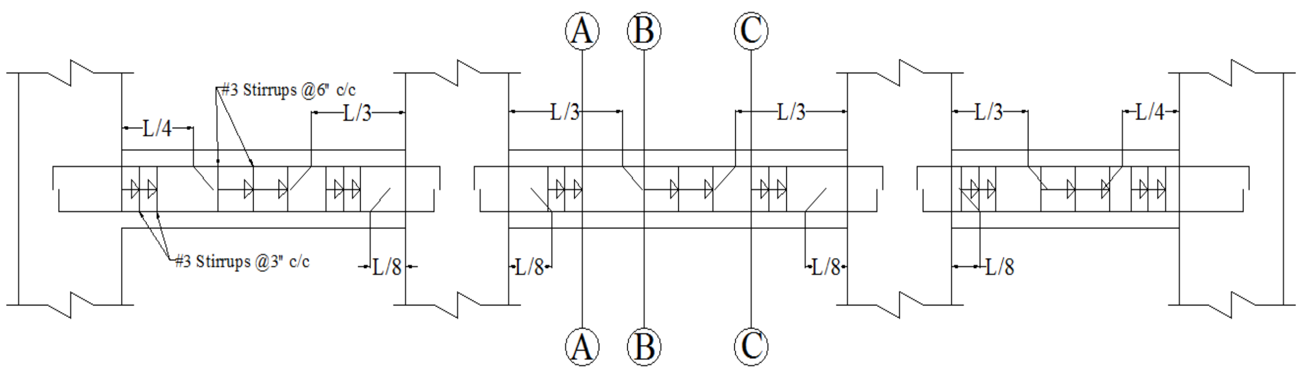


Fig 5.2.13: Beam rod detailing Y-axis

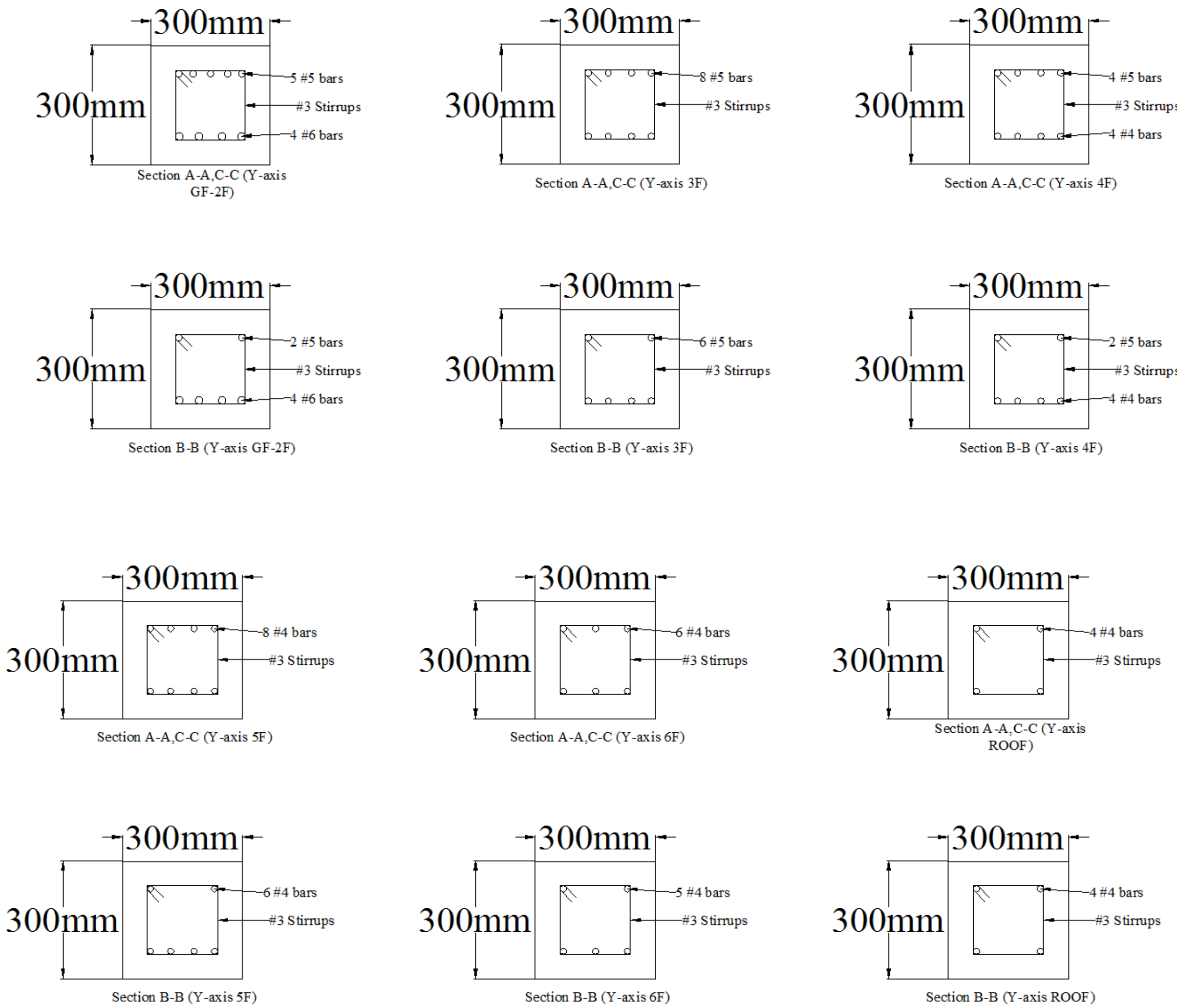


Fig 5.2.14: Beam Cross-sections for Y-axis

5.3 COMPARISON OF 6 FLOOR AND 8 FLOOR DESIGN:

While comparing the design of 6floor structure and 8-floor structure it was noticed that to compensate for the increase in reinforcement requirement the beams, higher no bars had to be used in designing 8-floor structure where lower no bars satisfied loads of 6-floor structure. Below is a graph showing the difference between the bars used per floor of the 6-floor and 8-floor structures:

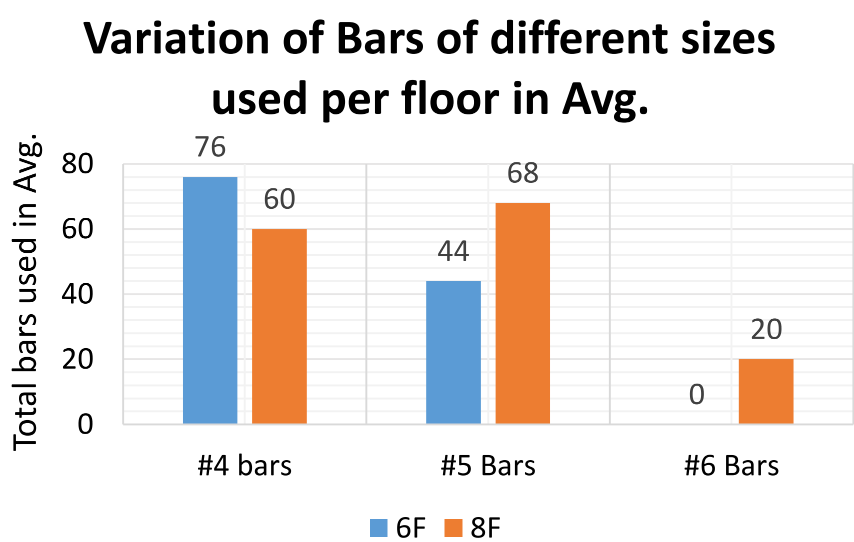


Fig 5.3.1: Chart of different size bars used per floor in the structures

The use of #4 bars decreased by 21%, #5 bars increased by 55% And We had to use #6 bars also.

According to www.homeadvisor.com :

Rebar Sizes and Prices Per Foot

You'll pay **$0.18 to $6.20 per foot** depending on the length you purchase and diameter. Most hardware stores identify rebar by its diameter. However, some will use industry sizing. The smaller the number, the smaller the diameter:

Standard size (metric)	Diameter	Cost per foot
#3 (#10)	3/8"	$0.18 - $1.25
#4 (#13)	1/2"	$0.30 - $2.00
#5 (#16)	5/8"	$0.45 - $2.55
#6 (#19)	3/4"	$0.75 - $3.70
#7 (#22)	7/8"	$1.30 - $5.05
#8 (#25)	1"	$3.25 - $6.20

Fig 5.3.2: Beam cost per linear foot

Using these prices of rebar we can calculate total increase in price per floor in 6 storied and 8 storied designs

Below is the graph indicating the avg. increase in price per floors reinforcement design.

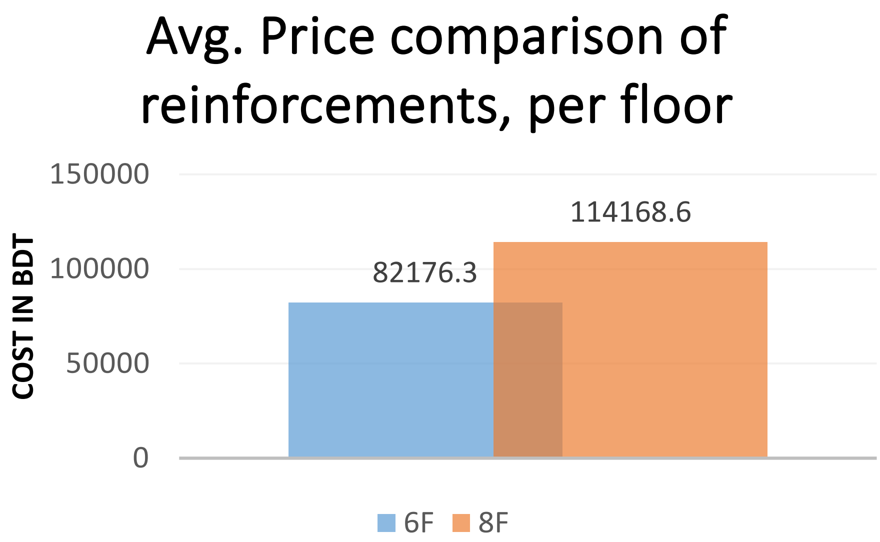


Fig 5.3.3: Chart of Prices to design per floor of 6Floor and 8Floor designs

It was observed per floor reinforcement design cost of the 8 storied structure increased by almost 39% than the design of the 6 storied structure.

6.1 CALCULATIONS:

For time constraints we were able to design only 3 columns (with the highest reinforcement requirements) from both structures.

(**Note:** For design of Column sections Bending m11 Modifier, Bending m22 Modifier, Bending m12 Modifier was taken as 1 instead of 0.00000001)

CALCULATION FOR 6 STORIED STRUCTURE:

Calculations of CC:

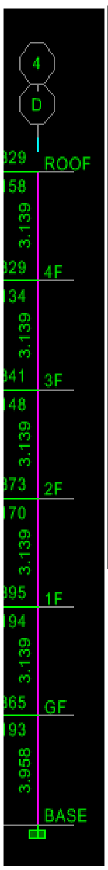


Table 6.1.1: Calculation of CC

Max bar area requirement (in^2)		Bar used	Bar area (in^2)	No of bars provided	Area of column section, Ag (in^2)	0.04 X Ag	Total area of steel provided, Ast (in^2)	Comments
For bottom 3 floors	3.958	#9	1	4	324	12.96	4	Ast<12.96 (ok)
For top 3 floors	3.139	#9	1	4	324	12.96	4	Ast<12.96 (ok)

Calculations of CEX:

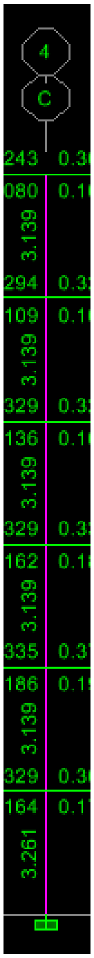


Table 6.1.2: Calculation of CEX								
Max bar area requirement (in^2)		Bar used	Bar area (in^2)	No of bars provided	Area of column section, Ag (in^2)	0.04 X Ag	Total area of steel provided, Ast (in^2)	Comments
For bottom 3 floors	3.261	#9	1	4	324	12.96	4	Ast<12.96 (ok)
For top 3 floors	3.139	#9	1	4	324	12.96	4	Ast<12.96 (ok)

Calculations of CI:

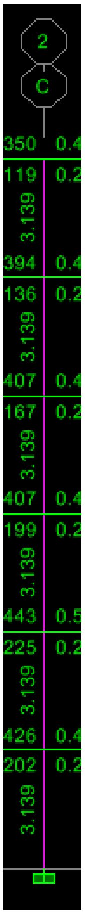


Table 6.1.3: Calculation of CI								
Max bar area requirement (in^2)		Bar used	Bar area (in^2)	No of bars provided	Area of column section, Ag (in^2)	0.04 X Ag	Total area of steel provided, Ast (in^2)	Comments
For bottom 3 floors	3.139	#9	1	4	324	12.96	4	Ast<12.96 (ok)
For top 3 floors	3.139	#9	1	4	324	12.96	4	Ast<12.96 (ok)

Design of tie bars:

We are going to use #3 bars as tie.

For top and bottom 23in tie spacing will be smaller of 4in or ¼ of least column dimension= 4.5in

So tie spacing will be #3 bars @4in c/c for top and bottom 23in

Tie spacing for middle part will be 2x4= 8in

So tie spacing will be #3 bars @8in c/c for middle portion

Calculation for 8 storied structure:

Calculations of CC:

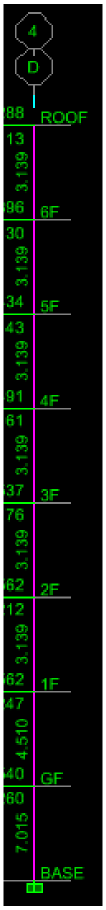


Table 6.1.4: Calculation of CC

Max bar area requirement (in^2)		Bar used	Bar area (in^2)	No of bars provided	Area of column section, Ag (in^2)	0.04 X Ag	Total area of steel provided, Ast (in^2)	Comments
For bottom 4 floors	7.015	#9	1	8	324	12.96	8	Ast<12.96 (ok)
For top 4 floors	3.139	#9	1	4	324	12.96	4	Ast<12.96 (ok)

Calculations of CEX:

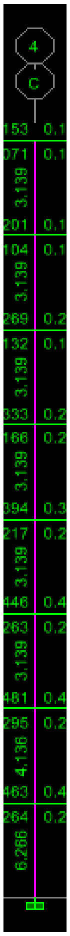


Table 6.1.5: Calculation of CEX									
Max bar area requirement (in^2)		Bar used	Bar area (in^2)	No of bars provided	Area of column section, Ag (in^2)	0.04 X Ag	Total area of steel provided, Ast (in^2)	Comments
For bottom 4 floors	6.266	#9	1	8	324	12.96	8	Ast<12.96 (ok)
For top 4 floors	3.139	#9	1	4	324	12.96	4	Ast<12.96 (ok)

Calculations of CI:

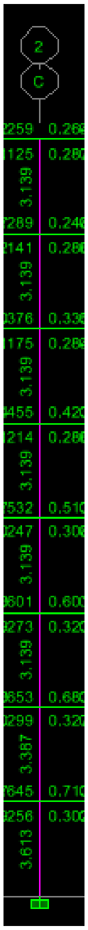


Table 6.1.6: Calculation of CI

Max bar area requirement (in^2)		Bar used	Bar area (in^2)	No of bars provided	Area of column section, Ag (in^2)	0.04 X Ag	Total area of steel provided, Ast (in^2)	Comments
For bottom 4 floors	3.613	#9	1	4	324	12.96	4	Ast<12.96 (ok)
For top 4 floors	3.139	#9	1	4	324	12.96	4	Ast<12.96 (ok)

Design of tie bars:

We are going to use #3 bars as tie.

For top and bottom 23in tie spacing will be smaller of 4in or ¼ of least column dimension= 4.5in

So tie spacing will be #3 bars @4in c/c for top and bottom 23in

Tie spacing for middle part will be 2x4= 8in

So tie spacing will be #3 bars @8in c/c for middle portion

6.2 FIGURES OF COLUMN SECTIONS: COLUMN FIGURES FOR 6 STORIED STRUCTURE:

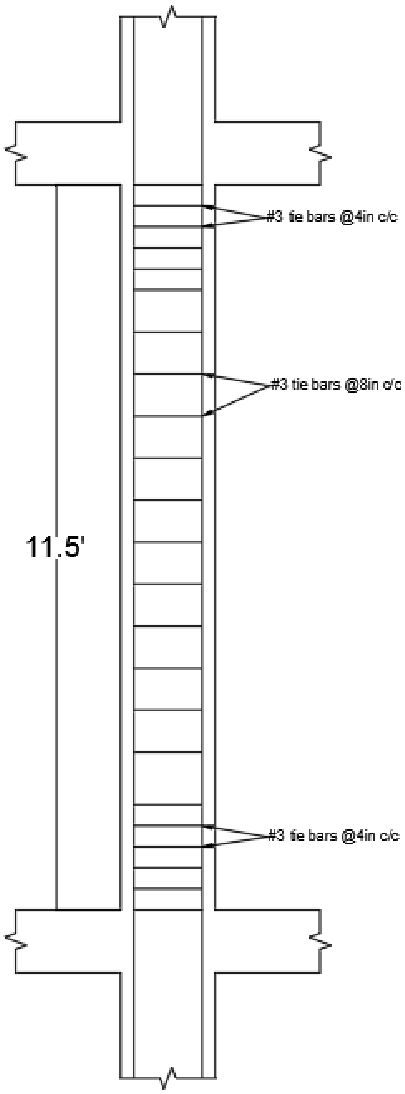


Fig 6.2.1: Column rod detailing of CC, CEX and CI for all floors

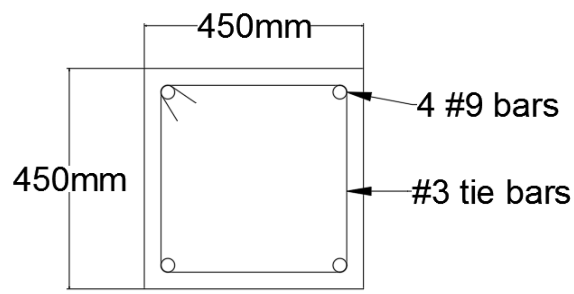


Fig 6.2.2: Column Cross-sections of CC, CEX and CI for all floors

Column figures for 8 storied structure:

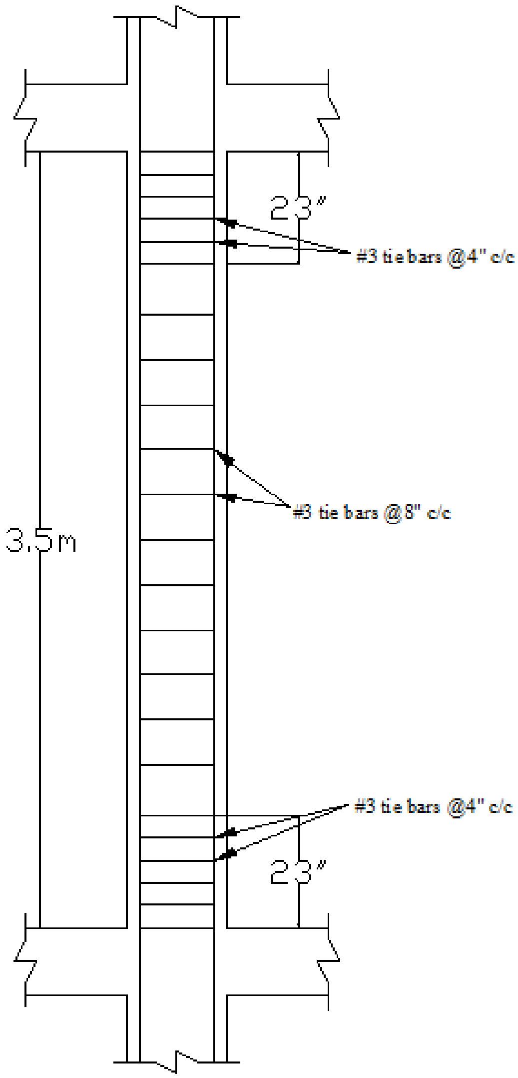


Fig 6.2.3: Column rod detailing of CC and CEX for top 4 floors and CI for all floors

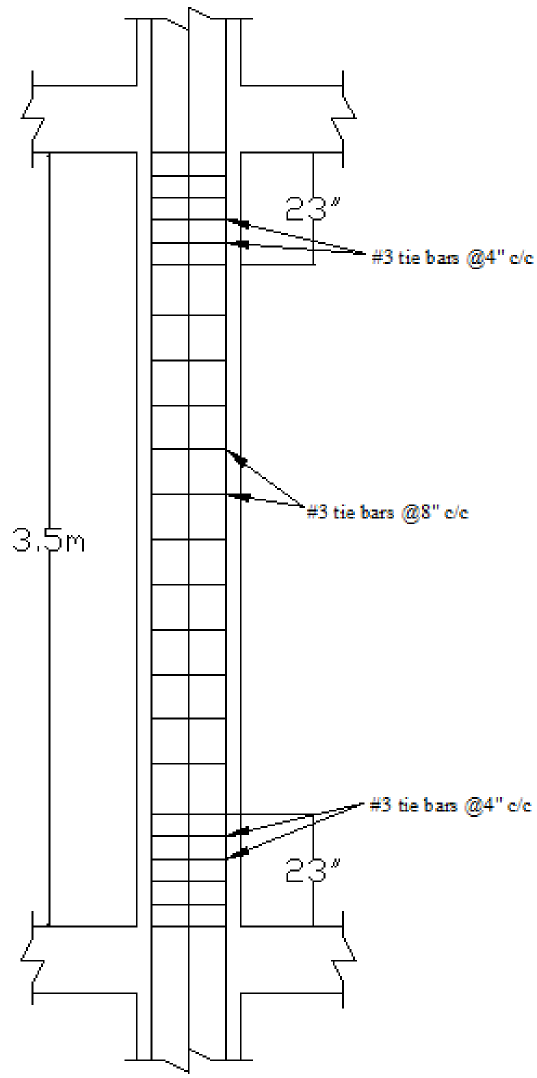


Fig 6.2.4: Column rod detailing of CC and CEX for bottom 4 floors

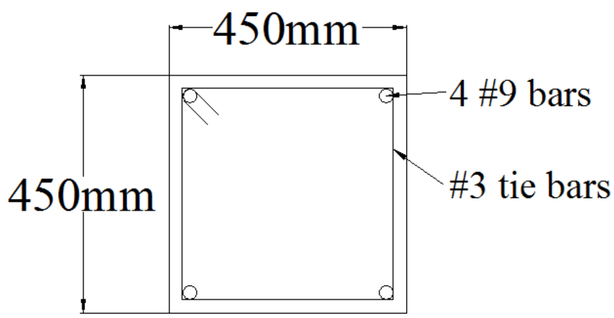


Fig 6.2.5: Column Cross-sections of CC and CEX for top 4 floors and CI for all floors

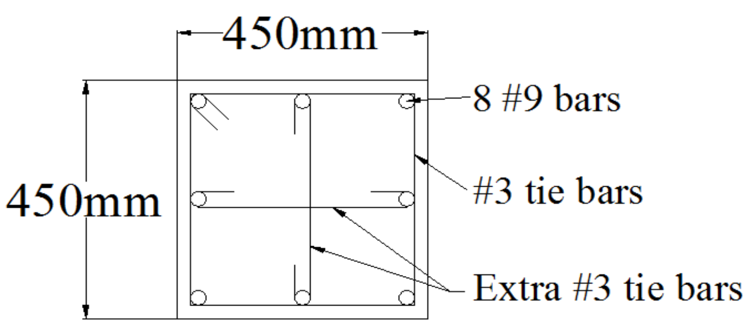


Fig 6.2.6: Column Cross-sections of CC and CEX bottom 4 floors

6.3 COMPARISON OF 6 FLOOR AND 8 FLOOR DESIGN:

While comparing the columns of the 6 storied and 8 storied structure it was noticed that the longitudinal reinforcement requirement in the base of the corner and external columns almost doubled in the 8 storied structure due to added load. So, to compensate for this the design of the bottom 4 floors had to be changed and extra 4 #9 bars were provided. But the requirements gradually decreased as we went up the floors with the constant being 3.139 in2 so for the remaining 4 floors the same design as 6 storied structures were provided.

The longitudinal reinforcement requirement for internal columns in 8-floor structure increased but not by much so the same column design could be used.

Below is a chart showing the percentage of increase in longitudinal reinforcement requirement for the base of 8 storied structure from the base of 6 storied structure:

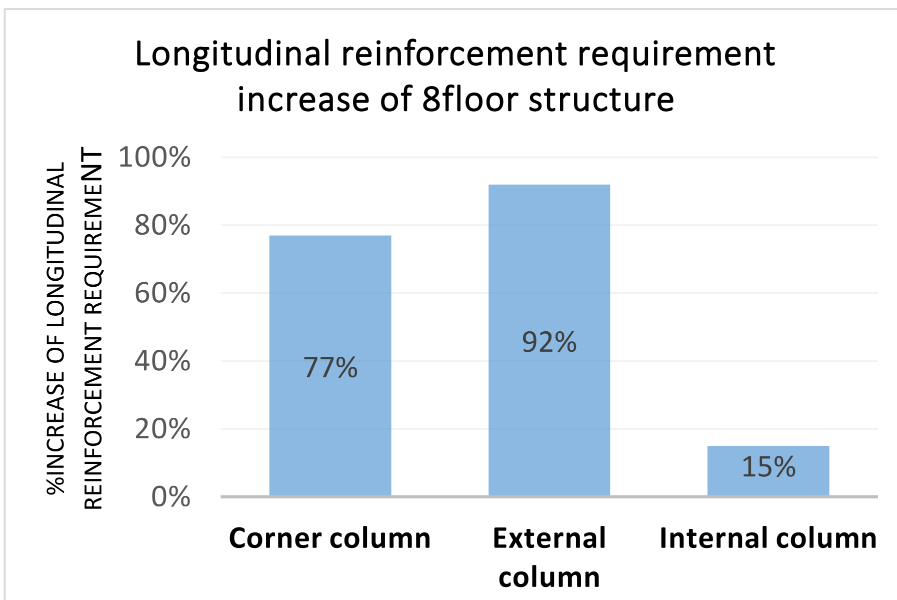


Fig 6.3.1: Chart of Longitudinal reinforcement requirement increase of 8floor structure

According to www.wirelessestimator.com :

Reinforcement Steel Pricing - Per Linear Foot			
Diameter	**Type**	**Pound Per LF**	**LF Price**
1/4"	#2	0.170	**$0.21**
3/8"	#3	0.376	**$0.34**
1/2"	#4	0.668	**$0.47**
5/8"	#5	1.043	**$0.66**
3/4"	#6	1.502	**$0.84**
7/8"	#7	2.044	**$1.02**
1"	#8	2.670	**$1.25**
1-1/8"	#9	3.400	**$1.53**
1-1/4"	#10	4.303	**$1.85**
1-3/8"	#11	5.313	**$2.13**

Fig 6.3.2: Column cost per linear foot

From this we can estimate the column Avg. cost difference per floor for six storied and eight storied structures reinforcement design. Below is a chart showing the cost comparison:

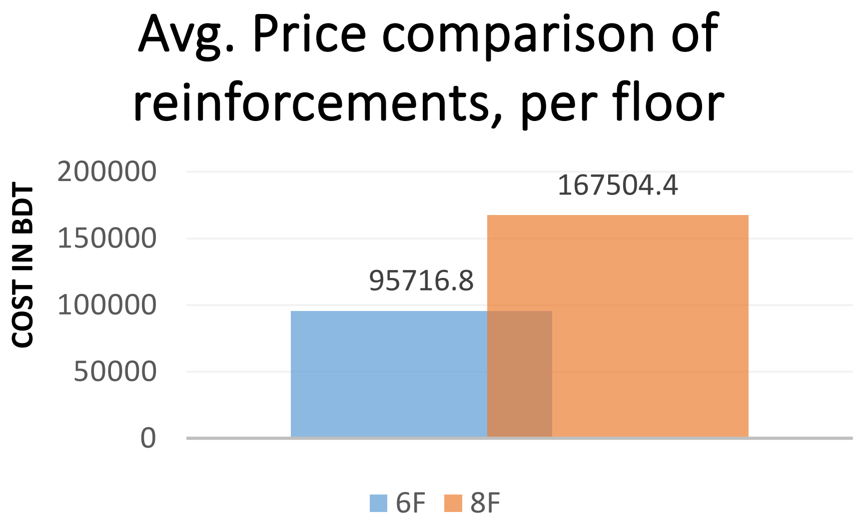


Fig 6.3.3: Chart of Prices to design per floor of 6Floor and 8Floor designs

It was observed per floor reinforcement design cost of the 8 storied structure increased by almost 75% in Avg. than the design of the 6 storied structure.

CHAPTER SEVEN: DESIGN OF SLAB

7.1 CALCULATIONS:

(**Note:** we are only designing the GF slab of both structures in this study for time constraints)

CALCULATION FOR LONG DIRECTION OF SIX STORIED STRUCTURE:

(**Note:** Slab no 1 = Slab 3-4 x A-B, Slab no 2 = Slab 3-4 x B-C, Slab no 3 = Slab 3-4 x C-D, Slab no 4 = Slab 2-3 x A-B, Slab no 5 = Slab 2-3 x B-C, Slab no 6 = Slab 2-3 x C-D, Slab no 7 = Slab 1-2 x A-B, Slab no 8 = Slab 1-2 x B-C, Slab no 9 = Slab 1-2 x C-D)

Slab dimensions:

Slab no.	Length, L (ft)	Base, B (ft)
1	3.3	9.84
2	3.3	12.3
3	3.3	14.76
4	4.9	9.84
5	4.9	12.3
6	4.9	14.76
7	4.1	9.84
8	4.1	12.3
9	4.1	14.76

Given			
Slab thickness (t)	Bending (t-1)	fc'	fy
150 mm	125 mm	3.5 Ksi	60 Ksi
6 in	5 in	3500 Psi	60000 psi

Considering b=1ft=12in

Load Determination:

	Dead load			Live load
	Partition wall	Floor finish	Self-weight	Live load
	2.5 KN/m2	1.5 KN/m2	73.8 lb/ft2	3.0 KN/m2
	0.363 Psi	0.218 Psi	0.513 Psi	0.435 Psi
Total	1.093 Psi			0.435 Psi
Factored load (1.2DL + 1.6LL)				2.007 Psi
				289.057 Psf
				0.289 Ksf

Moment selection: From ETABS model we get,

Table 7.1.1: Data for long direction

Slab no.	Length (ft)	Base (ft)	Left support moment (-M)	Midspan moment (M)	Right support moment (-M)
1	3.3	9.84	0.3202	0.1875	0.4322
2	3.3	12.3	0.5327	0.2718	0.6524
3	3.3	14.76	0.784	0.4214	0.6355
4	4.9	9.84	0.393	0.2633	0.5581
5	4.9	12.3	0.6663	0.3534	0.8014
6	4.9	14.76	0.9258	0.5008	0.7139
7	4.1	9.84	0.3837	0.2349	0.5152
8	4.1	12.3	0.6365	0.3242	0.7596
9	4.1	14.76	0.8956	0.4672	0.7136

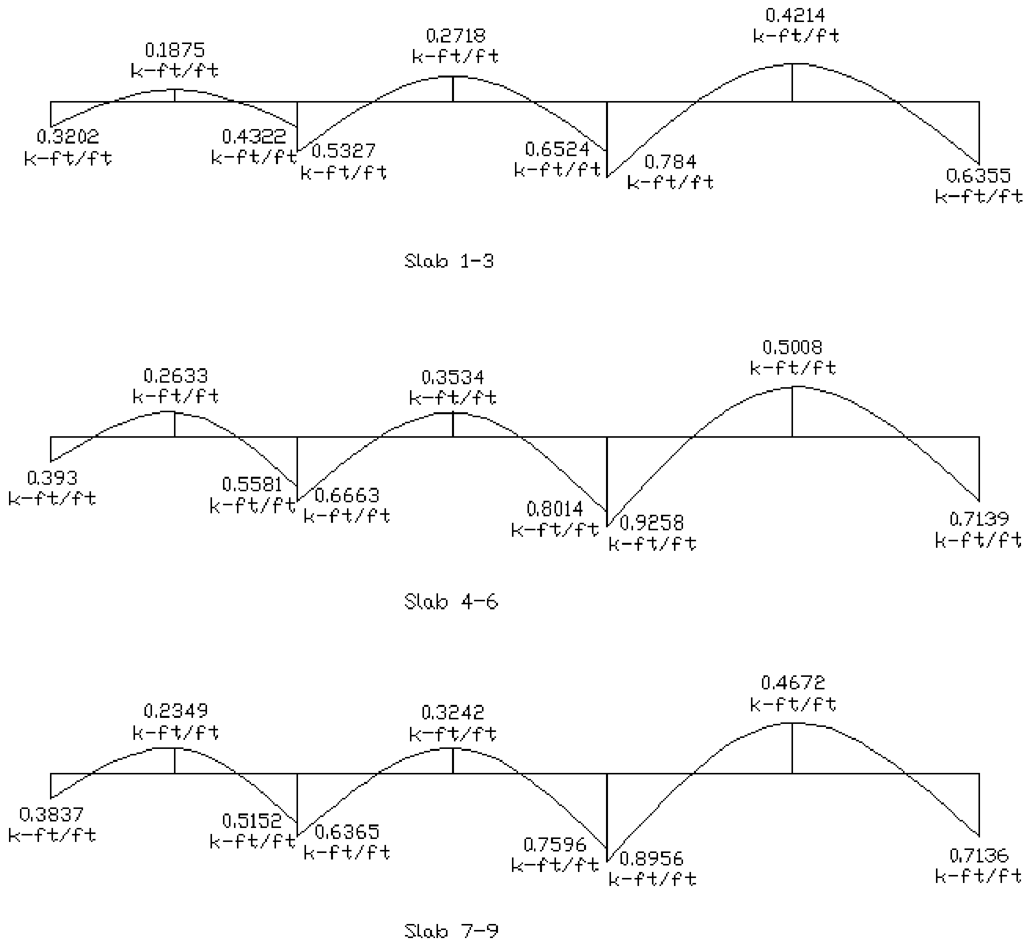


Fig 7.1.1: Moments on slab 1-9 Long direction

Minimum Effective Depth:

ρ = $0.85*\beta_1*(Fc'/Fy)*\xi u/(\xi u+\xi t) = .85*.85*(3.5/60)*0.003/(0.003+0.004) = 0.020$

Taking b=1ft=12in

Now $d_{eff}= (M_{max}/\varphi*\rho*Fy*b*(1-(.59*\rho*Fy/Fc')))^{(1/2)} = (0.3863*1000/.9*.020*60000*(1-(.59*.020*60000/3500)))^{(1/2)}$

= 0.67”

Checking availability of thickness:

As “d” is less than effective depth of (t-1) = (6-1) = 5”

So, t= 6” is ok

Determining minimum Area of steel required:

As_{min}= 0.0018*12*6= 0.13 in2

Table 7.1.2: Slab calculation for long direction

Slab no.	Position	M (k-ft/ft)	As (in2/ft)	As_{min} (in2/ft)	$As_{provided}$ (in2/ft)	Bar provided	Bar Area (in2)	Spacing
1	Left support	0.3202	0.0143	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1875	0.0083	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.4322	0.0193	0.13	0.13	1, #3 extra top	0.11	10 in c/c
2	Left support	0.5327	0.0238	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.2718	0.0203	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.6524	0.0291	0.13	0.13	1, #3 extra top	0.11	10 in c/c
3	Left support	0.784	0.0351	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.4214	0.0188	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.6355	0.0284	0.13	0.13	1, #3 extra top	0.11	10 in c/c
4	Left support	0.393	0.0175	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.2633	0.0117	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.5581	0.0249	0.13	0.13	1, #3 extra top	0.11	10 in c/c

5	Left support	0.6663	0.0298	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.3534	0.0157	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.8014	0.0358	0.13	0.13	1, #3 extra top	0.11	10 in c/c
6	Left support	0.9258	0.0414	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.5008	0.0223	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.7139	0.0319	0.13	0.13	1, #3 extra top	0.11	10 in c/c
7	Left support	0.3837	0.0171	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.2349	0.0105	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.5152	0.023	0.13	0.13	1, #3 extra top	0.11	10 in c/c
8	Left support	0.6365	0.0284	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.3242	0.0144	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.7596	0.034	0.13	0.13	1, #3 extra top	0.11	10 in c/c
9	Left support	0.8956	0.0401	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.4672	0.0208	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.7136	0.0319	0.13	0.13	1, #3 extra top	0.11	10 in c/c

CALCULATION FOR SHORT DIRECTION OF SIX STORIED STRUCTURE:

From ETABS model we get,

Table 7.1.3: Data for short direction

Slab no.	Length,L (ft)	Base,B (ft)	Left support moment (-M)	Mid-span moment (M)	Right support moment (-M)
1	9.84	3.3	0.5976	0.1918	0.4899
2	12.3	3.3	0.762	0.2836	0.8192
3	14.76	3.3	1.0417	0.4365	1.1126
4	9.84	4.9	0.6888	0.2625	0.5963
5	12.3	4.9	0.8918	0.3602	0.9593
6	14.76	4.9	1.1989	0.4896	1.2456
7	9.84	4.1	0.6443	0.2302	0.5754
8	12.3	4.1	0.8554	0.3309	0.9173
9	14.76	4.1	1.1586	0.4838	1.1964

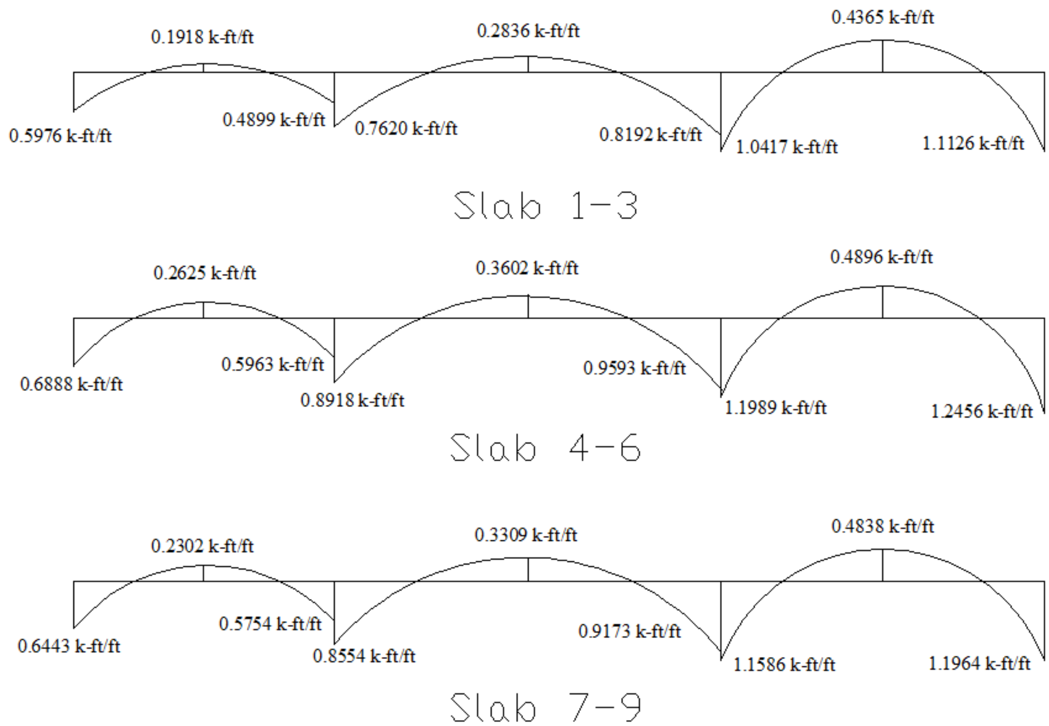


Fig 7.1.2: Moments on slab 1-9 Short direction

Table 7.1.4: Slab calculation for short direction								
Slab no.	Position	M (k-ft/ft)	As (in2/ft)	As_{min} (in2/ft)	$As_{provided}$ (in2/ft)	Bar provided	Bar Area (in2)	Spacing
1	Left support	0.5976	0.0267	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1918	0.0085	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.4899	0.0219	0.13	0.13	1, #3 extra top	0.11	10 in c/c
2	Left support	0.762	0.0341	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.2836	0.0126	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.8192	0.0366	0.13	0.13	1, #3 extra top	0.11	10 in c/c
3	Left support	1.0417	0.0467	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.4365	0.0195	0.13	0.13	#3	0.11	10 in c/c
	Right Support	1.1126	0.0499	0.13	0.13	1, #3 extra top	0.11	10 in c/c
4	Left support	0.6888	0.0308	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.2625	0.0117	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.5963	0.0266	0.13	0.13	1, #3 extra top	0.11	10 in c/c
5	Left support	0.8918	0.0399	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.3602	0.0161	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.9593	0.0429	0.13	0.13	1, #3 extra top	0.11	10 in c/c

6	Left support	1.1989	0.0538	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.4896	0.0218	0.13	0.13	#3	0.11	10 in c/c
	Right Support	1.2456	0.0559	0.13	0.13	1, #3 extra top	0.11	10 in c/c
7	Left support	0.6443	0.0288	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.2302	0.0102	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.5754	0.0257	0.13	0.13	1, #3 extra top	0.11	10 in c/c
8	Left support	0.8554	0.0383	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.3309	0.0147	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.9173	0.0411	0.13	0.13	1, #3 extra top	0.11	10 in c/c
9	Left support	1.1586	0.0519	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.4838	0.0216	0.13	0.13	#3	0.11	10 in c/c
	Right Support	1.1964	0.0537	0.13	0.13	1, #3 extra top	0.11	10 in c/c

CALCULATION FOR LONG DIRECTION OF EIGHT STORIED STRUCTURE:

From ETABS model we get,

Table 7.1.5: Calculation for long direction					
Slab no.	Length (ft)	Base (ft)	Left support moment (-M)	Midspan moment (M)	Right support moment (-M)
1	3.3	9.84	0.1844	0.1021	0.2386
2	3.3	12.3	0.2889	0.1366	0.3466
3	3.3	14.76	0.3912	0.2143	0.3103
4	4.9	9.84	0.2702	0.1594	0.3523
5	4.9	12.3	0.4041	0.1912	0.4757
6	4.9	14.76	0.5234	0.2709	0.4072
7	4.1	9.84	0.2527	0.1335	0.3145
8	4.1	12.3	0.3674	0.1652	0.4324
9	4.1	14.76	0.4843	0.2379	0.4036

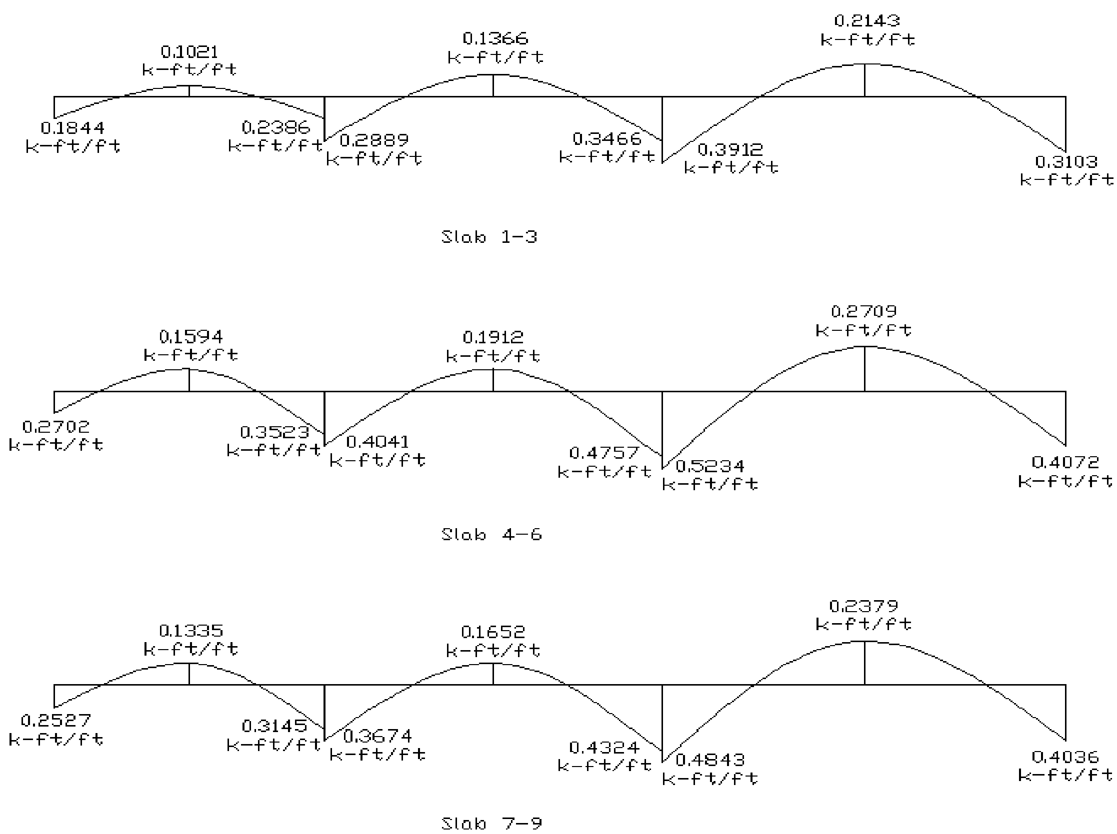


Fig 7.1.3: Moments on slab 1-9 Long direction

Table 7.1.6: Slab calculation for long direction								
Slab no.	Position	M (k-ft/ft)	As (in2/ft)	As_{min} (in2/ft)	$As_{provided}$ (in2/ft)	Bar provided	Bar Area (in2)	Spacing
1	Left support	0.1844	0.0082	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1021	0.0045	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.2386	0.0106	0.13	0.13	1, #3 extra top	0.11	10 in c/c
2	Left support	0.2889	0.0129	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1366	0.0061	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.3466	0.0154	0.13	0.13	1, #3 extra top	0.11	10 in c/c
3	Left support	0.3912	0.0174	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.2143	0.0095	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.3103	0.0138	0.13	0.13	1, #3 extra top	0.11	10 in c/c
4	Left support	0.2702	0.012	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1594	0.0071	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.3523	0.0157	0.13	0.13	1, #3 extra top	0.11	10 in c/c
5	Left support	0.4041	0.018	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1912	0.0085	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.4757	0.0212	0.13	0.13	1, #3 extra top	0.11	10 in c/c
6	Left support	0.5234	0.0234	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.2709	0.0121	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.4072	0.0182	0.13	0.13	1, #3 extra top	0.11	10 in c/c

7	Left support	0.2527	0.0113	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1335	0.0059	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.3145	0.014	0.13	0.13	1, #3 extra top	0.11	10 in c/c
8	Left support	0.3674	0.0164	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1652	0.0074	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.4324	0.0193	0.13	0.13	1, #3 extra top	0.11	10 in c/c
9	Left support	0.4843	0.0216	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.2379	0.0106	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.4036	0.018	0.13	0.13	1, #3 extra top	0.11	10 in c/c

CALCULATION FOR SHORT DIRECTION OF EIGHT STORIED STRUCTURE:

From ETABS model we get,

Table 7.1.7: Calculation for short direction

Slab no.	Length,L (ft)	Base,B (ft)	Left support moment (-M)	Midspan moment (M)	Right support moment (-M)
1	9.84	3.3	0.3479	0.1021	0.3424
2	12.3	3.3	0.4605	0.1366	0.5038
3	14.76	3.3	0.6375	0.2143	0.6185
4	9.84	4.9	0.4022	0.1594	0.4215
5	12.3	4.9	0.5516	0.1912	0.6032
6	14.76	4.9	0.7473	0.2709	0.7
7	9.84	4.1	0.3453	0.1335	0.337
8	12.3	4.1	0.4561	0.1652	0.4822
9	14.76	4.1	0.6139	0.2379	0.5963

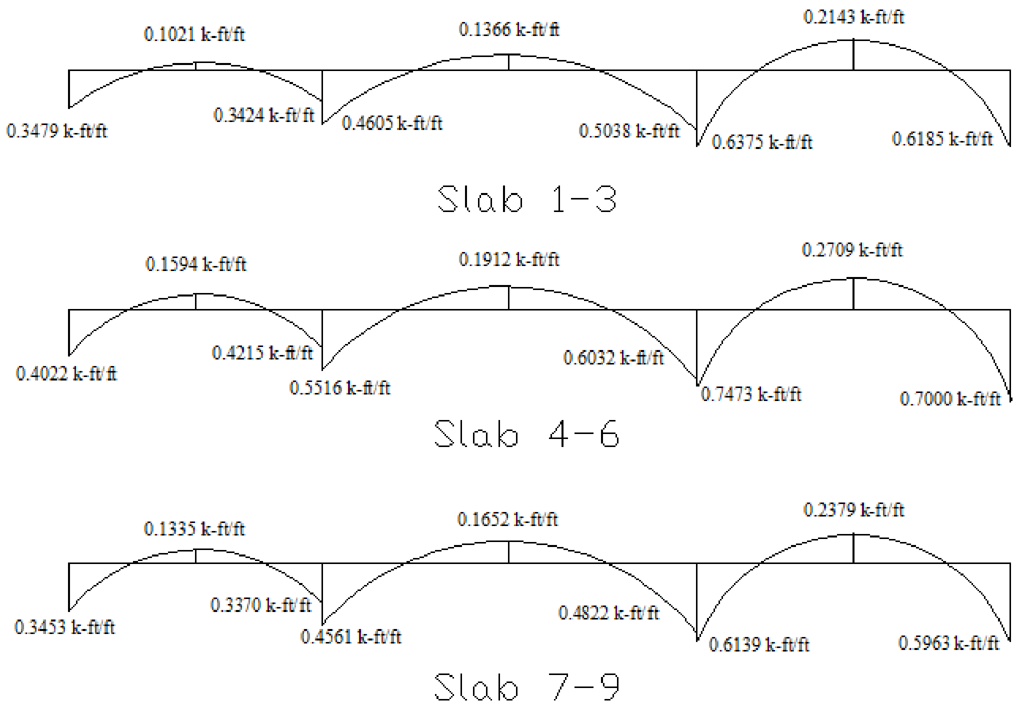


Fig 7.1.4: Moments on slab 1-9 Long direction

Table 7.1.8: Slab calculation for long direction

Slab no.	Position	M (k-ft/ft)	As (in2/ft)	As_{min} (in2/ft)	$As_{provided}$ (in2/ft)	Bar provided	Bar Area (in2)	Spacing
1	Left support	0.3479	0.0155	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1021	0.0045	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.3424	0.0153	0.13	0.13	1, #3 extra top	0.11	10 in c/c
2	Left support	0.4605	0.0205	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1366	0.0061	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.5038	0.0225	0.13	0.13	1, #3 extra top	0.11	10 in c/c

3	Left support	0.6375	0.0285	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.2143	0.0095	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.6185	0.0276	0.13	0.13	1, #3 extra top	0.11	10 in c/c
4	Left support	0.4022	0.0179	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1594	0.0071	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.4215	0.0188	0.13	0.13	1, #3 extra top	0.11	10 in c/c
5	Left support	0.5516	0.0246	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1912	0.0085	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.6032	0.0269	0.13	0.13	1, #3 extra top	0.11	10 in c/c
6	Left support	0.7473	0.0334	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.2709	0.0121	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.7	0.0313	0.13	0.13	1, #3 extra top	0.11	10 in c/c
7	Left support	0.3453	0.0154	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1335	0.0059	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.337	0.015	0.13	0.13	1, #3 extra top	0.11	10 in c/c
8	Left support	0.4561	0.0203	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.1652	0.0074	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.4822	0.0215	0.13	0.13	1, #3 extra top	0.11	10 in c/c
9	Left support	0.6139	0.0274	0.13	0.13	1, #3 extra top	0.11	10 in c/c
	Mid-span	0.2379	0.0106	0.13	0.13	#3	0.11	10 in c/c
	Right Support	0.5963	0.0266	0.13	0.13	1, #3 extra top	0.11	10 in c/c

7.2 FIGURES OF SLAB SECTIONS:

The design for both 6 storied and 8 storied structure's ground floor slab design was same. So, same slab design was used for both structures.

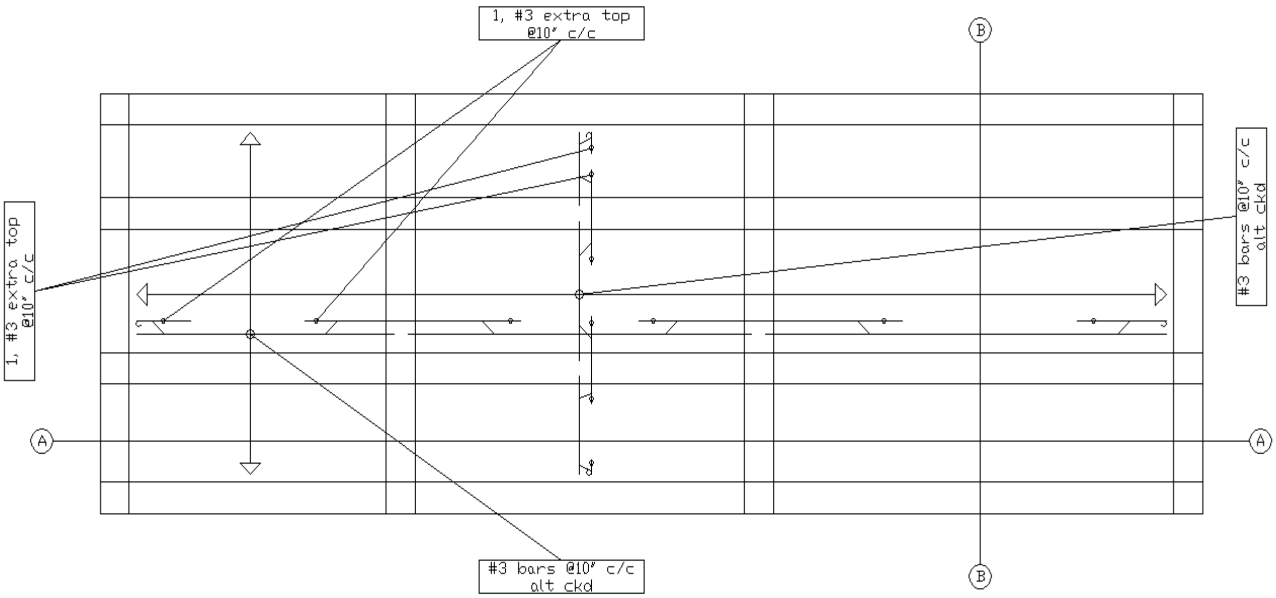


Fig 7.2.1: Slab section

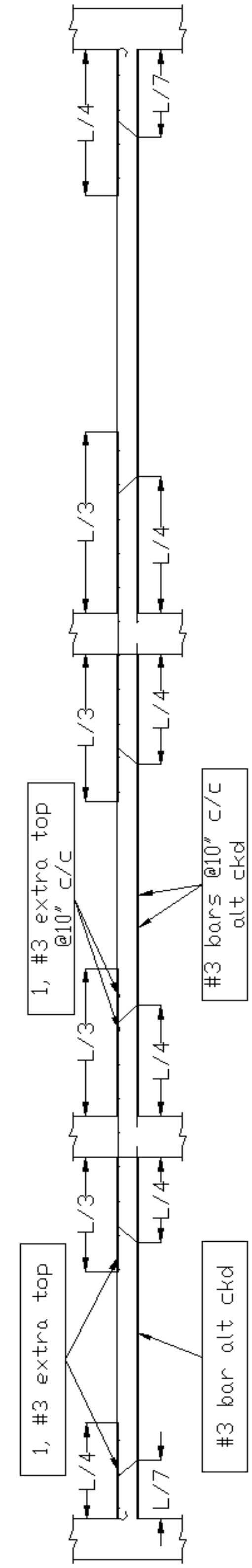


Fig 7.2.2a: Section A-A

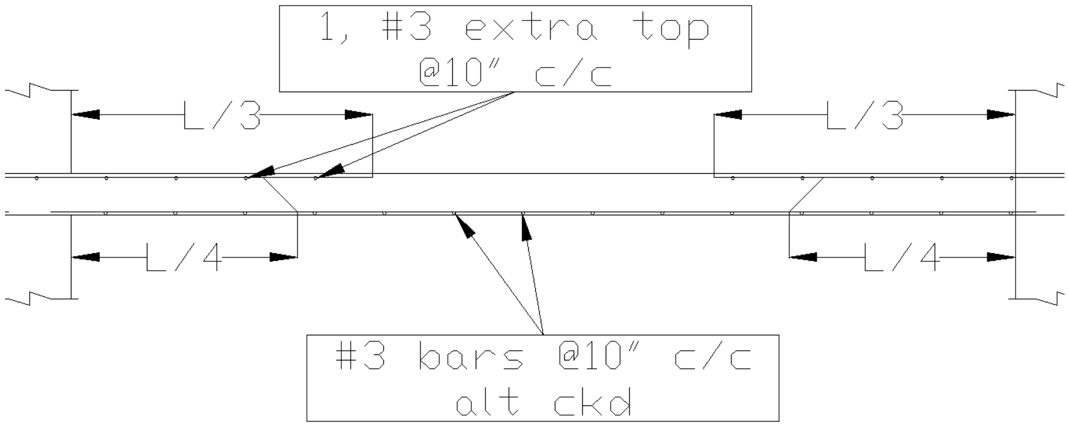


Fig 7.2.2b: Section A-A

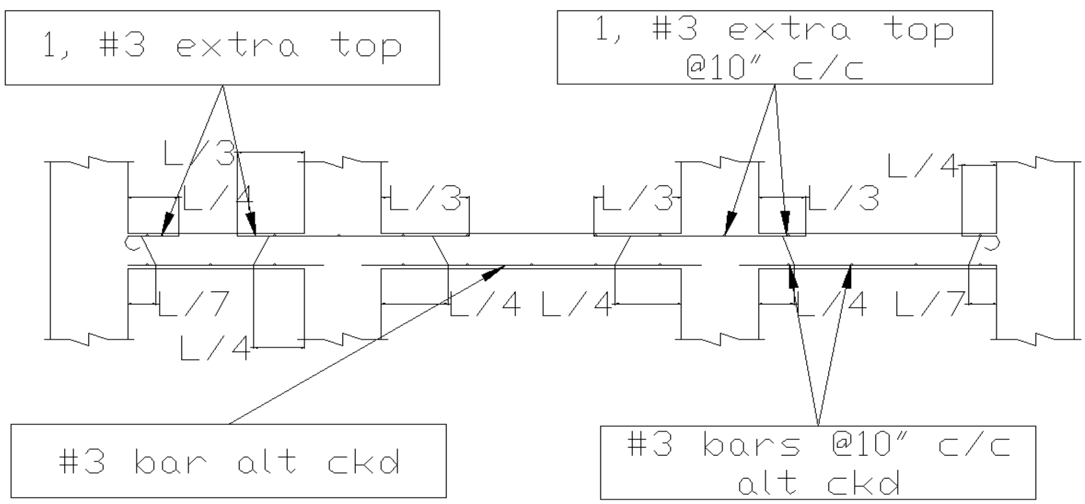


Fig 7.2.3: Section B-B

CHAPTER EIGHT: COMPARISON BETWEEN WIND AND EARTHQUAKE LOADS

8.1 METHOD OF COMPARISON:

From the ETABS model, the maximum simulated deflection due to Earthquake and wind load was selected. Only the data of ROOF story-level was selected as in both structure ROOF deflected the most from its initial point and the point which deflected the most the data of that was collected for each load (i.e. WX, WY, EQX and EQY). After that the deflections for EQ and WL in the 6 floor structure were compared and the same was done for the 8 floor structure. Then the deflection due to wind load was compared between 8floor and 6floor structures.

8.2 COMPARISON OF EQ AND WL IN 6 STORIED STRUCTURE:

Below are the figures of the structure deflected due to wind load and the maximum deflection point:

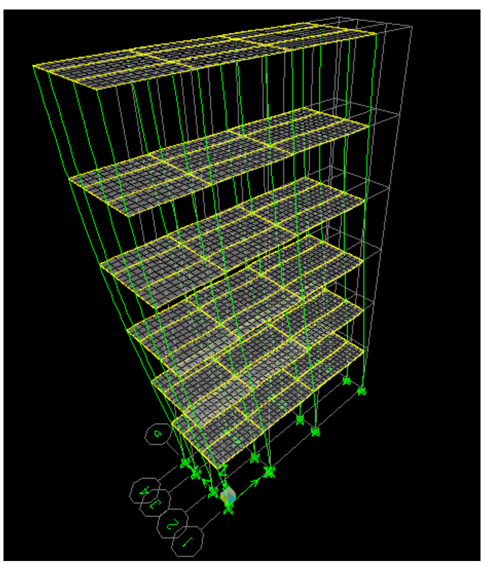


Fig 8.2.1: Deflection of 6 storied structure due to Wind-WX load

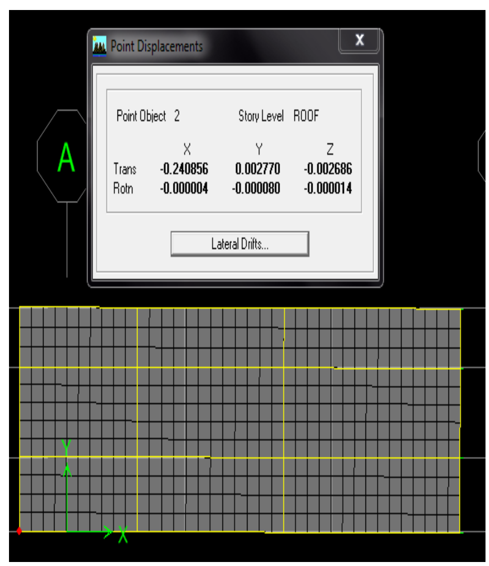


Fig 8.2.2: Maximum deflection point in WX of 6 storied

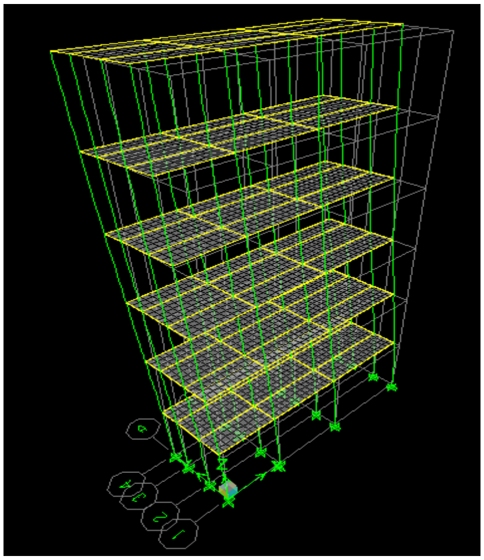


Fig 8.2.3: Deflection of 6 storied structure due to Wind-WY load

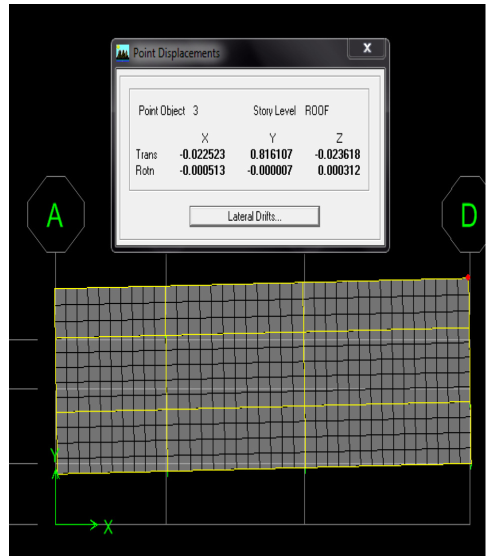


Fig 8.2.4: Maximum deflection point in WY of 6 storied structure.

Deflection of structure at ROOF due to Wind load was observed to be: 0.24in at X-direction and 0.82in at Y-direction

Below are the figures of the structure deflected due to Earthquake load and the maximum deflection point:

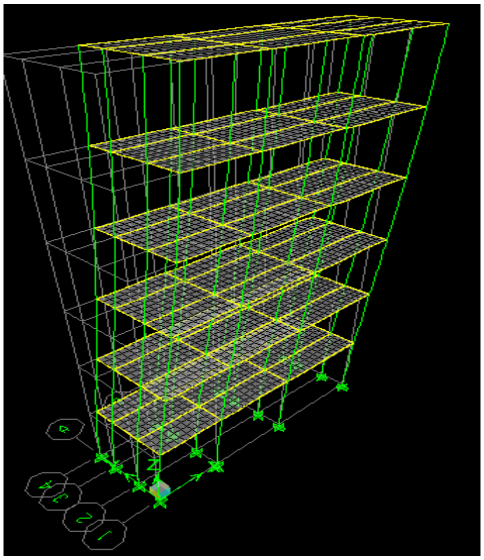


Fig 8.2.5: Deflection of 6 storied structure due to Earthquake-EQX load

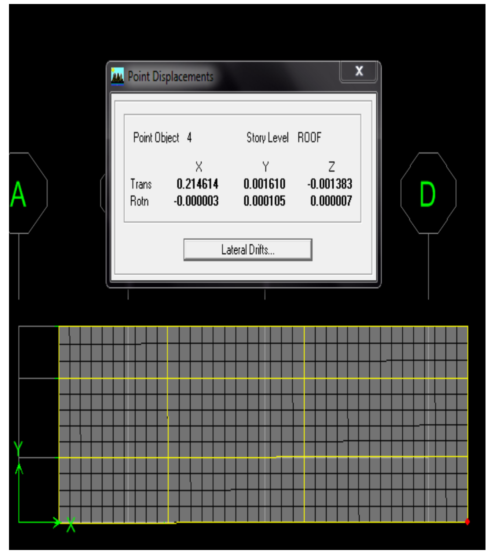


Fig 8.2.6: Maximum deflection point in EQX of 6 storied structure.

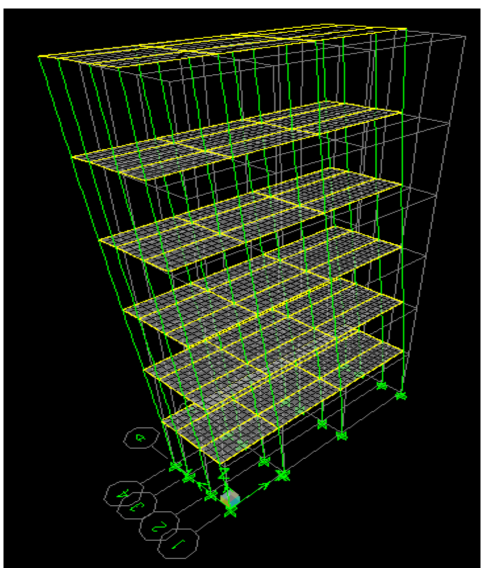


Fig 8.2.7: Deflection of 6 storied structure due to Earthquake-EQY load

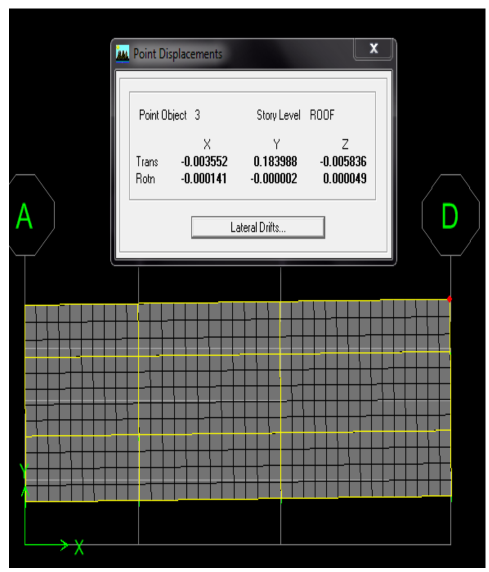


Fig 8.2.8: Maximum deflection point in EQY of 6 storied structure.

Deflection of structure at ROOF due to Earthquake load was observed to be: 0.21in at X-direction and 0.18in at Y-direction

It was observed that due to wind load at X direction the structure deflects 12.5% more and at Y direction the structure deflects 78% more than Earthquake load in X, Y directions respectively.

So in the case of this structure the wind forces were more dominant than earthquake forces.

Explanation:

- This is because our structure is on low hazard Earthquake zone being Zone 1 and our wind speed being 238km/h
- Secondly the length to width ratio is 3. At X axis our total length is 36.9 ft. and at Y axis our total length is 12.3 ft. So, X axis gets higher wind exposure compared to Y axis and that's why the structure deflects more in Y direction.

8.3 COMPARISON OF EQ AND WL IN 8 STORIED STRUCTURE:

Below are the figures of the structure deflected due to wind load and the maximum deflection point:

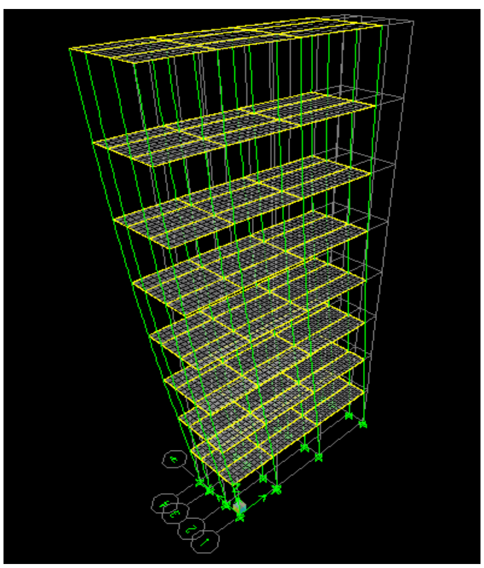


Fig 8.2.9: Deflection of 8 storied structure due to Wind-WX load

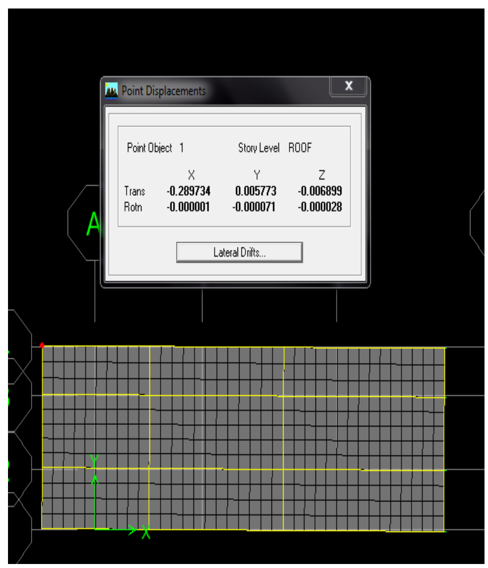


Fig 8.2.10: Maximum deflection point in WX of 8 storied structure.

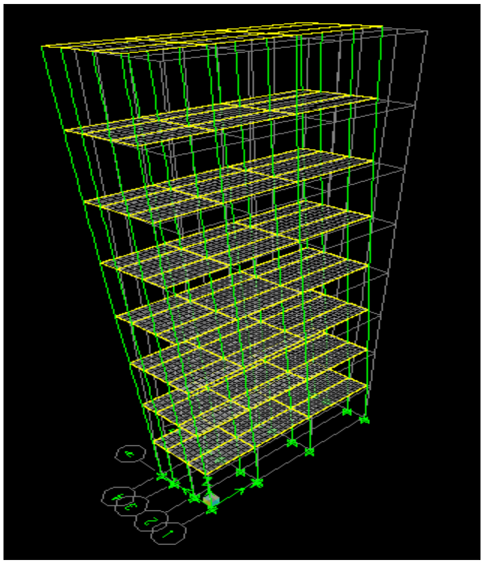


Fig 8.2.11: Deflection of 8 storied structure due to Wind-WY load

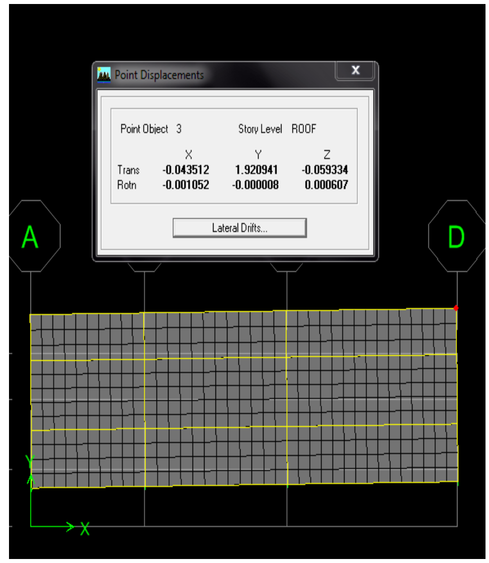


Fig 8.2.12: Maximum deflection point in WY of 8

Deflection of structure at ROOF due to Wind load was observed to be: 0.29in at X-direction and 1.92in at Y-direction

Below are the figures of the structure deflected due to Earthquake load and the maximum deflection point:

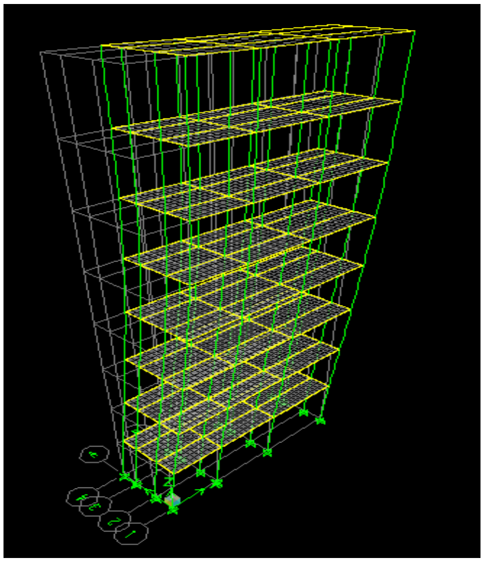


Fig 8.2.13: Deflection of 8 storied structure due to Earthquake-EQX load

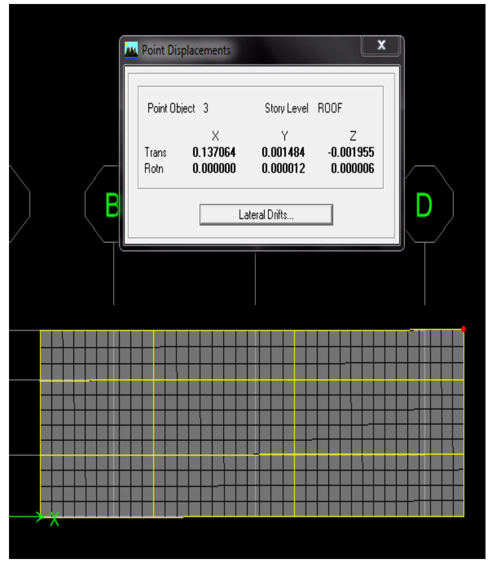


Fig 8.2.14: Maximum deflection point in EQX of 8 storied structure.

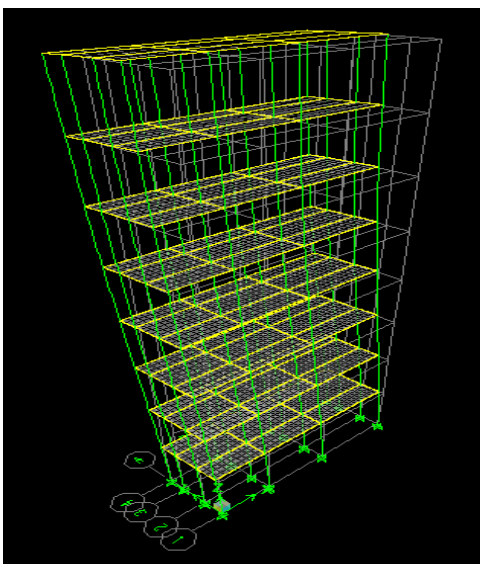


Fig 8.2.15: Deflection of 8 storied structure due to Earthquake-EQY load

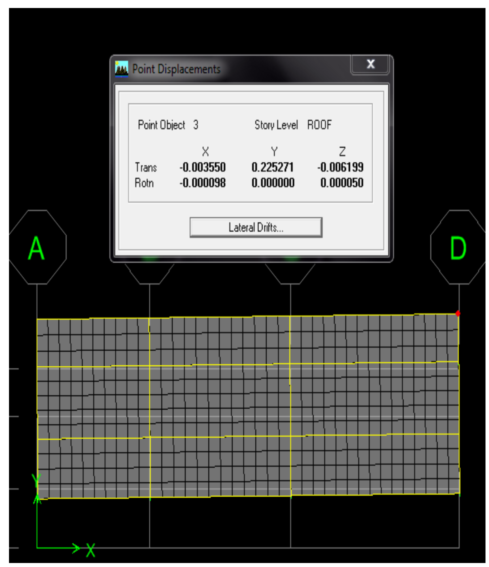


Fig 8.2.16: Maximum deflection point in EQY of 8 storied structure.

Deflection of structure at ROOF due to Earthquake load was observed to be: 0.14in at X-direction and 0.23in at Y-direction

Deflection of structure at ROOF due to Earthquake load was observed to be: 0.21in at X-direction and 0.18in at Y-direction

It was observed that due to wind load at X direction the structure deflects 51.72% more and at Y direction the structure deflects 88.02% more than Earthquake load in X, Y directions respectively.

So in the case of this structure the wind forces were more dominant than earthquake forces.

Explanation:

- This is because our structure is on low hazard Earthquake zone being Zone 1 and our wind speed being 238km/h
- Secondly the length to width ratio is 3. At X axis our total length is 36.9 ft. and at Y axis our total length is 12.3 ft. So, X axis gets higher wind exposure compared to Y axis and that's why the structure deflects more in Y direction.

8.4 COMPARISON OF WL BETWEEN 6 AND 8 STORIED STRUCTURE:

Maximum deflection due to wind load in the 6 storied structure was observed to be 0.24in at X-direction and 0.82in at Y-direction and maximum deflection due to wind load in the 8 storied structure was observed to be 0.29in at X-direction and 1.92in at Y-direction.

So, due to the addition of two extra floors the deflection increased by 21% in X direction and 134% in Y-direction.

CHAPTER NINE: CONCLUSION AND RECOMMENDATION

CONCLUSION:

Using ACI 318, a preliminary design of a six storied R.C.C structure was completed and then two more stories was added on top of it to create eight-storied version of it. Overall, the structures were done as efficiently as possible but the structure may need future edits and the structure may not fully comply with ACI 318 code. As the structure was not a likely structure to be applied in real life and the dimensions were fictional it contained many flaws which need many revisions and edits.

While designing a two-way slab it was noticed that moments on both negative and positive sides were very low as a result the corresponding area of steel was very low so minimum reinforcement (with extra tops) was provided to be safe which means the structure may not be as economical as it could have been.

In this report, only the basic planning and design of a six-storied residential building and an eight-storied residential building were completed and presented. Stair and footing design was skipped due to time shortage.

From the initial analysis, it was noticed that due to the extra load of two stories the designs changed for the eight storied structure and the cost per floor increased.

Also observed were the deflections of the structures due to earthquake load and wind load. In our case as our structure's length to width ratio was 3 and the wind speed was very high both of our structures deflected more due to wind load making it the dominant load acting upon our structure. The deflection of the eight-storied structure due to wind was much more than the deflection of the six-storied structure due to wind as expected.

Below given are the changes noticed in our study bullet point form:

- We compared the design changes and per floor rebar cost in the two structures. Where the per floor avg. rebar cost for beams increased by 39% for beams and for columns increased by 75%
- The dominating force in both structure was Wind due to high exposure.
- The deflection due to wind was more in 8 storied structure. To be exact 21% in X direction and 134% in Y-direction.

RECOMMENDATION:

This is only a study conducted by considering a fictional building with unreal dimensions as a result there may be many errors in design. Also only the preliminary and basic design was done in this study and the structure may need many revisions and corrections needed to be made.

Group 1.A would like to give some personal recommendations that could help improve this study:

- Updated software and materials (like codes) must be used for the study to be more up-to-date and realistic.
- It is to be noted that the design of beams and columns were done by considering the slab bending m11,m22, and m12 to be 1 instead of considering it close to zero for more accurate results. It is highly recommended that the design must be done by considering slab modifiers close to zero. (In the case of this study by group 1.A the values of longitudinal reinforcement for beam and column did not differ that much and the design was done in a way that there was some safety room while considering the reinforcement. But still the beams x-axis 1F #2, y-axis GF #C, y-axis 1F #C, y-axis 2F #C, y-axis 3F #C and y-axis 4F #C needs to be revised and redesigned).
- For time constraints only two beam sections from every floor were designed in this study. One on X-axis and One on Y-axis and the beams were selected based on the highest reinforcement requirement. It is recommended that analysis of all of the beams must be done.
- For time constraints we were able to design only 3 columns. It is recommended that all the columns should be designed. Also to note is that the rebar percentage for the structure of group 1 was very low (averaging 1.5%). As this structure was unrealistic there couldn't be

much done for this problem but it is good to keep the rebar percentage near 4% to make the design more efficient.

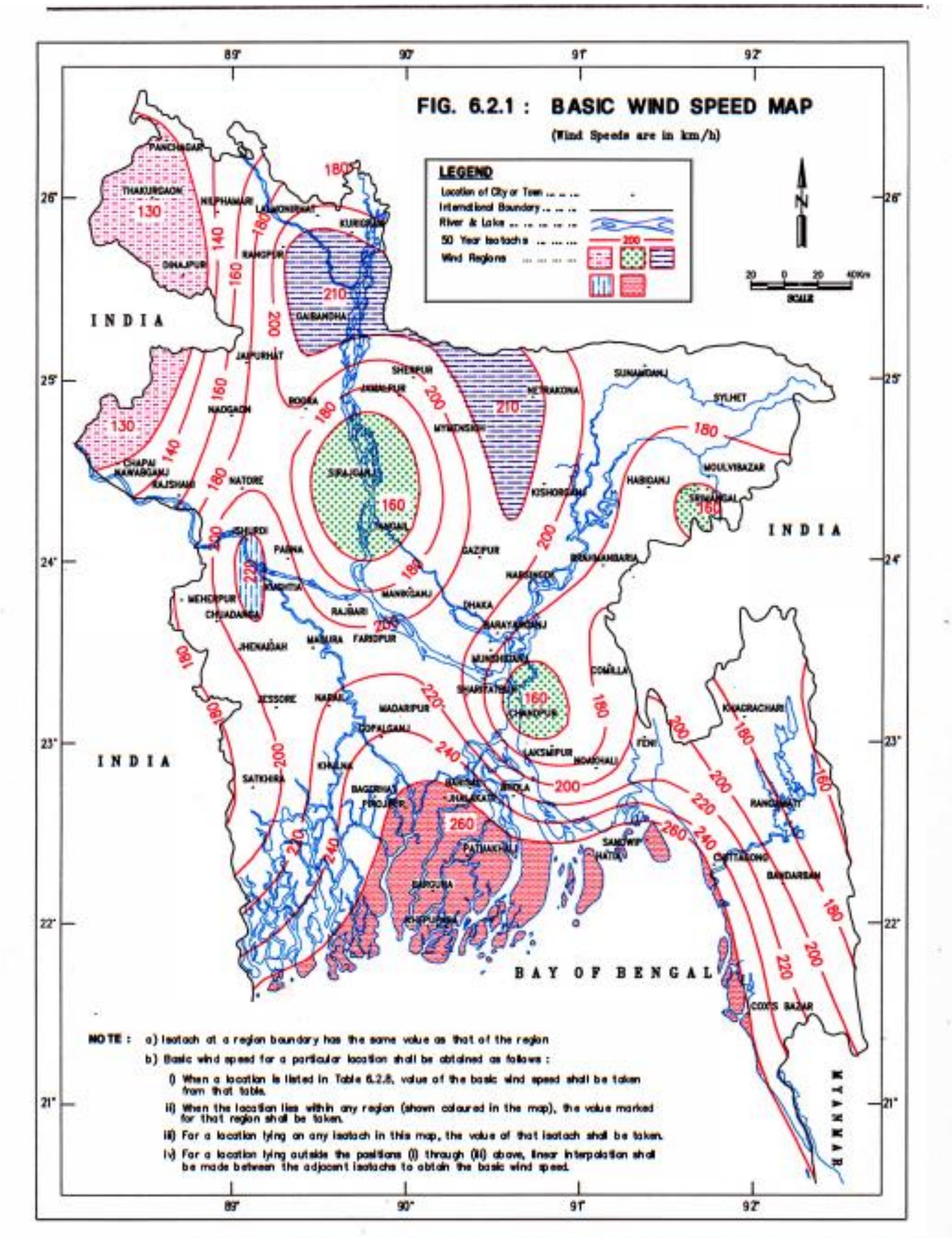
- For time constraints we only designed the GF slab in this study. It is recommended to design slabs of all the floors. (In the case of group 1.A structure GF was considered because it contained the highest moment which was still very low and minimum reinforcement was provided in slabs. As this is the case all the other floors will have similar results so the same design could apply to all the floors). Similar results for eight-storied structure.

APPENDICES

APPENDIX 1:

BNBC for wind loads:

Basic Wind speed map of Bangladesh



Basic Wind speed for selected locations in Bangladesh

Location	Basic Wind Speed (km/h)	Location	Basic Wind Speed (km/h)
Angarpota	150	Lalmonirhat	204
Bagerhat	252	Madaripur	220
Bandarban	200	Magura	208
Barguna	260	Manikganj	185
Barisal	256	Meherpur	185
Bhola	225	Moheshkhali	260
Bogra	198	Moulvibazar	168
Brahmanbaria	180	Munshiganj	184
Chandpur	160	Mymensingh	217
Chapai Nawabganj	130	Naogaon	175
Chittagong	260	Narail	222
Chuadanga	198	Narayanganj	195
Comilla	196	Narsinghdi	190
Cox's Bazar	260	Natore	198
Dahagram	150	Netrokona	210
Dhaka	210	Nilphamari	140
Dinajpur	130	Noakhali	184
Faridpur	202	Pabna	202
Feni	205	Panchagarh	130
Gaibandha	210	Patuakhali	260
Gazipur	215	Pirojpur	260
Gopalganj	242	Rajbari	188
Habiganj	172	Rajshahi	155
Hatiya	260	Rangamati	180
Ishurdi	225	Rangpur	209
Joypurhat	180	Satkhira	183
Jamalpur	180	Shariatpur	198
Jessore	205	Sherpur	200
Jhalakati	260	Sirajganj	160
Jhenaidah	208	Srimangal	160
Khagrachhari	180	St. Martin's Island	260
Khulna	238	Sunamganj	195
Kutubdia	260	Sylhet	195
Kishoreganj	207	Sandwip	260
Kurigram	210	Tangail	160
Kushtia	215	Teknaf	260
Lakshmipur	162	Thakurgaon	130

2.4.6.2 Sustained Wind Pressure : The sustained wind pressure, q_z on a building surface at any height z above ground shall be calculated from the following relation :

$$q_z = C_c C_I C_z V_b^2 \quad (2.4.1)$$

where, q_z = sustained wind pressure at height z, kN/m^2

C_I = structure importance coefficient as given in Table 6.2.9

C_c = velocity-to-pressure conversion coefficient = 47.2×10^{-6}

C_z = combined height and exposure coefficient as given in Table 6.2.10

V_b = basic wind speed in km/h obtained from Sec 2.4.5

Structure importance Coefficient, C_I

Structure Importance Category (see Table 6.1.1 for Occupancy)	Structure Importance Coefficient, C_I
I Essential facilities	1.25
II Hazardous facilities	1.25
III Special occupancy structures	1.00
IV Standard occupancy structures	1.00
V Low-risk structures	0.80

Combined height and Exposure co-efficient, C_z

Height above ground level, z (metres)	Coefficient, C_z (1)		
	Exposure A	Exposure B	Exposure C
0-4.5	0.368	0.801	1.196
6.0	0.415	0.866	1.263
9.0	0.497	0.972	1.370
12.0	0.565	1.055	1.451
15.0	0.624	1.125	1.517
18.0	0.677	1.185	1.573
21.0	0.725	1.238	1.623
24.0	0.769	1.286	1.667
27.0	0.810	1.330	1.706
30.0	0.849	1.371	1.743
35.0	0.909	1.433	1.797
40.0	0.965	1.488	1.846
45.0	1.017	1.539	1.890
50.0	1.065	1.586	1.930
60.0	1.155	1.671	2.002
70.0	1.237	1.746	2.065
80.0	1.313	1.814	2.120
90.0	1.383	1.876	2.171
100.0	1.450	1.934	2.217
110.0	1.513	1.987	2.260
120.0	1.572	2.037	2.299
130.0	1.629	2.084	2.337
140.0	1.684	2.129	2.371
150.0	1.736	2.171	2.404
160.0	1.787	2.212	2.436
170.0	1.835	2.250	2.465
180.0	1.883	2.287	2.494
190.0	1.928	2.323	2.521
200.0	1.973	2.357	2.547
220.0	2.058	2.422	2.596
240.0	2.139	2.483	2.641
260.0	2.217	2.541	2.684
280.0	2.910	2.595	2.724
300.0	2.362	2.647	2.762

Note : (1) Linear interpolation is acceptable for intermediate values of z.

2.4.6.3 Design Wind Pressure : The design wind pressure, p_z for a structure or an element of a structure at any height, z above mean ground level shall be determined from the relation :

$$p_z = C_G C_p q_z \qquad (2.4.2)$$

where, p_z = design wind pressure at height z , kN/m^2

C_G = gust coefficient which shall be G_z , G_h, or $\bar{G}$ as set forth in Sec 2.4.6.6

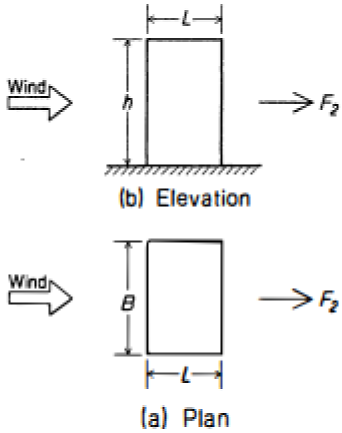
C_p = pressure coefficient for structures or components as set forth Sec 2.4.6.7

q_z = sustained wind pressure obtain from Eq (2.4.1).

Gust responses factor, G_h and G_z

Height above ground level (metres)	G_h (2) and G_z		
	Exposure A	Exposure B	Exposure C
0-4.5	1.654	1.321	1.154
6.0	1.592	1.294	1.140
9.0	1.511	1.258	1.121
12.0	1.457	1.233	1.107
15.0	1.418	1.215	1.097
18.0	1.388	1.201	1.089
21.0	1.363	1.189	1.082
24.0	1.342	1.178	1.077
27.0	1.324	1.170	1.072
30.0	1.309	1.162	1.067
35.0	1.287	1.151	1.061
40.0	1.268	1.141	1.055
45.0	1.252	1.133	1.051
50.0	1.238	1.126	1.046
60.0	1.215	1.114	1.039
70.0	1.196	1.103	1.033
80.0	1.180	1.095	1.028
90.0	1.166	1.087	1.024
100.0	1.154	1.081	1.020
110.0	1.114	1.075	1.016
120.0	1.134	1.070	1.013
130.0	1.126	1.065	1.010
140.0	1.118	1.061	1.008
150.0	1.111	1.057	1.005
160.0	1.104	1.053	1.003
170.0	1.098	1.049	1.001
180.0	1.092	1.046	1.000
190.0	1.087	1.043	1.000
200.0	1.082	1.040	1.000
220.0	1.073	1.035	1.000
240.0	1.065	1.030	1.000
260.0	1.058	1.026	1.000
280.0	1.051	1.022	1.000
300.0	1.045	1.018	1.000

Note : (1) For main wind-force resisting systems, use building or structure height h for z.
(2) Linear interpolation is acceptable for intermediate values of z.



Rectangular Building

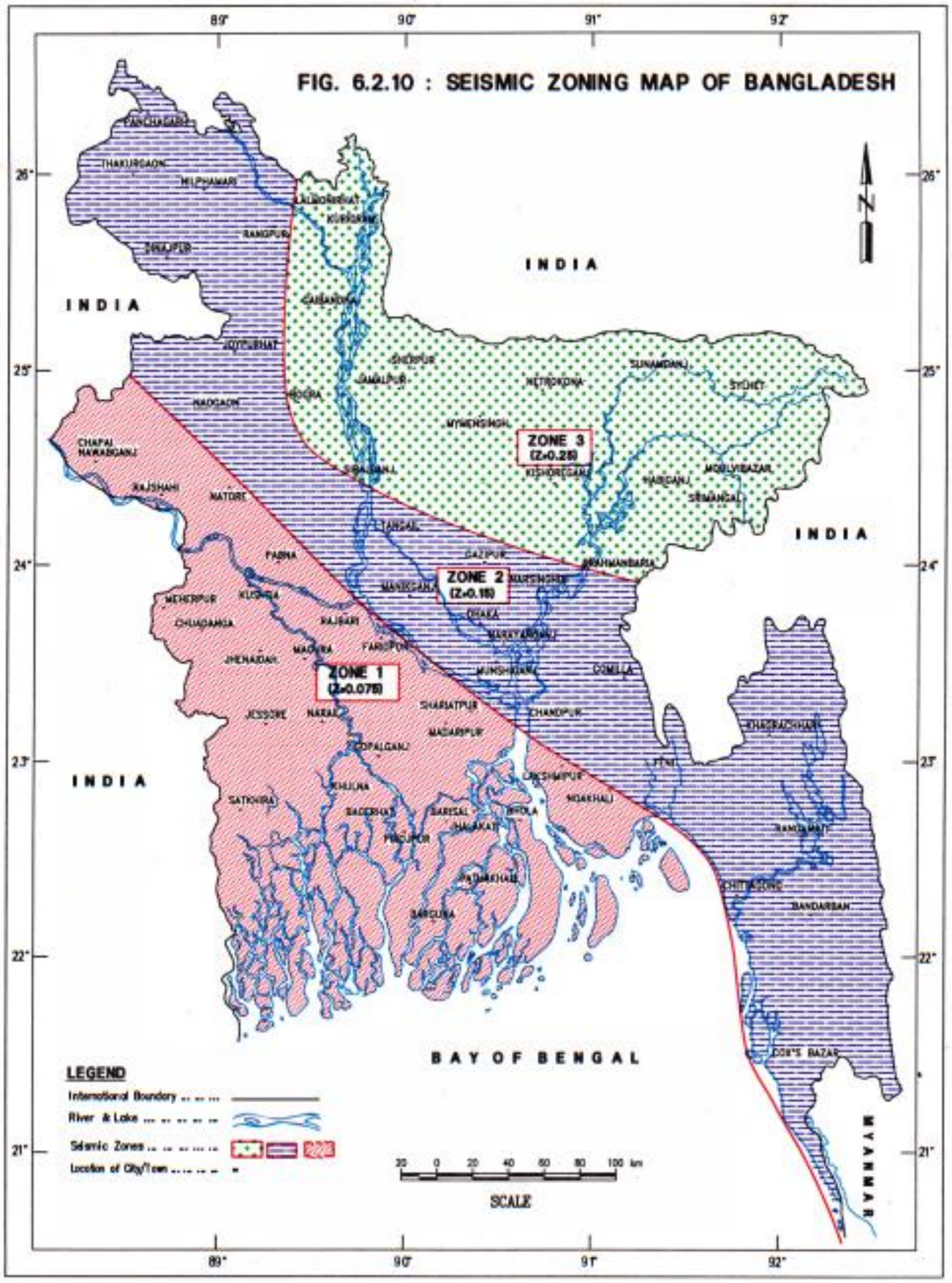
Overall Pressure Coefficients, $\bar{C}_p^{(2)}$ for Rectangular Buildings with Flat Roofs

h/B	L/B					
	0.1	0.5	0.65	1.0	2.0	≥ 3.0
≤5	1.40	1.45	1.55	1.40	1.15	1.10
10.0	1.55	1.85	2.00	1.70	1.30	1.15
20.0	1.80	2.25	2.55	2.00	1.40	1.20
≥40.0	1.95	2.50	2.80	2.20	1.60	1.25

Note:(1) These coefficients are to be used with Method-2 given in Sec 2.4.6.6a(ii). Use $\bar{C}_p = \pm 0.7$ for roof in all cases.
(2) Linear interpolation may be made for intermediate values of $\bar{h}/B$ and L/B.

APPENDIX 2:

Seismic zoning map of Bangladesh



2.5.5.2 Seismic Dead Load : Seismic dead load, W, is the total dead load of a building or a structure, including permanent partitions, and applicable portions of other loads listed below :

a) In storage and warehouse occupancies, a minimum of 25 per cent of the floor live load shall be applicable.

b) Where an allowance for partition load is included in the floor design in accordance with Sec 2.3.3.3, all such loads but not less than 0.6 kN/m^2 shall be applicable.

c) Total weight of permanent equipment shall be included.

2.5.6.1 Design Base Shear : The total design base shear in a given direction shall be determined from the following relation :

$$V = \frac{ZIC}{R}W \tag{2.5.1}$$

where, Z = Seismic zone coefficient given in Table 6.2.22
I = Structure importance coefficient given in Table 6.2.23
R = Response modification coefficient for structural systems given in Table 6.2.24
W = The total seismic dead load defined in Sec 2.5.5.2
C = Numerical coefficient given by the relation :

Seismic Zone Coefficients, Z

Seismic Zone (see Fig 6.2.10)	Zone Coefficient
1	0.075
2	0.15
3	0.25

Structure Importance Coefficients *I, I'*

Structure Importance Category (see Table 6.1.1 for occupancy)	Structure Importance Coefficient	
	I	I'
I Essential facilities	1.25	1.50
II Hazardous facilities	1.25	1.50
III Special occupancy structures	1.00	1.00
IV Standard occupancy structures	1.00	1.00
V Low-risk Structures	1.00	1.00

Structure Period : The value of the fundamental period, T of the structure shall be determined from one of the following methods :

a) **Method A** : For all buildings the value of T may be approximated by the following formula :

$$T = C_t (h_n)^{3/4} \tag{2.5.3}$$

where, C_t = 0.083 for steel moment resisting frames
= 0.073 for reinforced concrete moment resisting frames, and eccentric braced steel frames
= 0.049 for all other structural systems
h_n = Height in metres above the base to level n.

Response Modification Coefficient for Structural Systems, *R*

c. Moment Resisting Frame System	1. Special moment resisting frames (SMRF) i) Steel ii) Concrete 2. Intermediate moment resisting frames (IMRF), concrete(4) 3. Ordinary moment resisting frames (OMRF) i) Steel ii) Concrete (5)	 12 12 8 6 5

Table 6.2.25
Site Coefficient, *S* for Seismic Lateral Forces (1)

Site Soil Characteristics		Coefficient, *S*
Type	**Description**	
S_1	A soil profile with either : a) A rock-like material characterized by a shear-wave velocity greater than 762 m/s or by other suitable means of classification, or b) Stiff or dense soil condition where the soil depth is less than 61 metres	1.0
S_2	A soil profile with dense or stiff soil conditions, where the soil depth exceeds 61 metres	1.2
S_3	A soil profile 21 metres or more in depth and containing more than 6 metres of soft to medium stiff clay but not more than 12 metres of soft clay	1.5
S_4	A soil profile containing more than 12 metres of soft clay characterized by a shear wave velocity less than 152 m/s	2.0

Note : (1) The site coefficient shall be established from properly substantiated geotechnical data. In locations where the soil properties are not known in sufficient detail to determine the soil profile type, soil profile S_3 shall be used. Soil profile S_4 need not be assumed unless the building official determines that soil profile S_4 may be present at the site, or in the event that soil profile S_4 is established by geotechnical data.

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BOOKS, WEBSITES AND SOFTWARE USED:

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