SEISMIC PERFORMANCE OF AN RCC FRAME STRUCTURE WITH DIFFERENT SHEAR WALL ARRANGEMENTS

Final Year Project UG-2013

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This is to certify that the

Final Year Project Titled

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has been accepted towards the requirements

for the undergraduate degree

in

CIVIL ENGINEERING

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"In the name of Allah the most beneficent, the most merciful"

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Completion of this research work is the outcome of co-operation of many devoted and supportive people. It is tough to measure their assistances in helping us to carry out this research work. Besides our supervisor, we would also like to thank Mr.Arslan Mushtaq. Finally, we are thankful to our parents who were always supportive and encouraged us during the intervals of shear work stress.

ABSTRACT

Since the discovery of Reinforced Concrete in 1849 by Joseph Monier, due to its flexibility, speed of construction, sustainability, accessibility of raw materials and its easiness to cast, it quickly became the first choice of building materials by the civil engineers of 19th century. Many RC structures were constructed in the Era and many more in the following centuries up to the current date. The whole world, as well as our country Pakistan, has many ancient RC structures. RC structures experience lateral loads due to wind or earthquake thus to improve their life we provide Shear Walls. In Pakistan, especially Northern area experience a number of earthquakes around the year. For that reason structures should be able to resist these lateral loads loads and get least damage during and after the incident. These walls need not only be effective, but also be economically viable. These Shear walls are generally required for medium to high rise buildings, for shorter structures these are not necessary .For effective results these walls should be provided in an symmetrical arrangement.

In the first domain of the project analysis of structure under gravity loads was carried out. In the second domain of the project push over analysis, during an earthquake, was carried out and ATC 40 Capacity Spectrum were studied. The parameters of the study were Load deflection curve, Hinge formation, Spectral Acceleration, Spectral Displacement and Time Period. The research work concludes that there is an increase in the stability and stiffness due to the provision of the Shear Walls in efficient location.

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1 INTRODUCTION

1.1 General:

The phenomenon of earthquake has lived with the humans since the start of the times but it has been only over the last century when we have begun to understand what earthquakes are and what causes them. Earthquakes are a major problem for mankind, killing thousands each year along with a huge loss of economy. Mankind always struggled to get away with the devastating effects of Earthquakes. The nature of an earthquake event and resulting damages has explained by many researchers. In an earthquake event all economic and human losses are mainly due to failures of human made facilities like buildings, bridges, dames and transportation systems etc.

In the recent past there have been devastating events of Earthquakes. Some of the most disastrous events suffered by the different parts of the world in the last decade are, Bhuj in India (2001), Bam earthquake in Iran (2003), Sumatra which caused tsunami (2004), Kashmir (2005) in Pakistan with a magnitude of 7.6 (Sahahzada et al. 2011; Maqsood & Schwarz, 2011), Haiti (2010) having magnitude 7.0 (Narayanan et al. 2011), Chili Earthquake and tsunami (2010), China (2010), Indonesia (2010).

Losses during an earthquake event are mainly caused by the poor construction practices which prevail in most of the developing countries. These countries are mostly having non-engineered structures which either do not follow design guidelines or designed by following old codes. Most of these buildings are designed to resist loads due to gravity only, without proper seismic provisions and are highly vulnerable against earthquake generated loads. The use of different building codes, planning rules, construction techniques, quality assessment methods and materials means that the vulnerability of RC structures differs from place to place.

Enormous losses of life and economy in the last decade demonstrate the strong need for Earthquake Risk Assessment (ERA) for all countries of the world having a strong seismic risk. ERA has three main components; seismic hazard, vulnerability of exposed structures and the loss (Ahmad 2011). Development of ERA is helpful in earthquake preparedness and planning along with policy making in natural hazards such as earthquake in order to mitigate the harmful effects. Vulnerability is actually the main component of ERA and it develops the damage indicators for structures at different hazard levels. Different parts of the world differ in structural seismic vulnerability due to different building design codes, techniques of construction, and methods for quality assessment, construction materials and soil conditions.

Therefore, the earthquake vulnerability of building structures has remained a key area for the researchers in order to reduce the hazards of earthquake as much as possible. Country like Pakistan is under the risk of moderate to high level of earthquakes. October 8, 2005 Kashmir earthquake in Pakistan has brought huge damages and challenges for the researchers to improve measures to decrease the hazards.

Research has found that it is almost impossible to completely diminish the effects of earthquakes, but their risk can be reduced by taking suitable measures. There are several methods to improve the seismic response of a structure like Shock absorbers and tuned mass dampers. One of the most suitable and efficient way is to place shear walls at appropriate locations, which will provide adequate stiffness to the structure.

RC Multi-Storey Frame Buildings are sufficient for resisting both the vertical and horizontal load. When such structures are designed without shear wall, the beam and column sizes are relatively heavy, steel amount is also required in large quantity thus there is lot of mobbing at these joints. Thus, it is difficult to place, vibrate concrete at these joints and displacement is quite large which induces much more forces in the members. Shear wall may become critical from the point of view of economy and control of lateral deflections. In RC multi-story building R.C.C. lift well or shear wall are general requirement. Center of mass and center of rigidity of the building must coincide in this case. However, on many instances the design has to be based on the off center position of lift and stair case wall with respect to center of mass which results into an excessive forces in most of the structural members, unwanted torsion moment and deflection. Generally shear wall can be defined as structural vertical member that is able to resist combination of shear, moment and axial load induced by lateral load and gravity load transfer to the wall from other structural member. Reinforced concrete walls, which include lift wells or shear walls, are the usual requirements of Multi Storey Buildings. Design by coinciding center of mass and center of rigidity of the building is the ideal for any structure. Providing of shear wall represents a structurally effective solution to stiffen a building structural system because the main purpose of a shear wall is to increase the rigidity for lateral load resistance. The use of shear wall structure has gained fame in tall structures, especially in the construction of office buildings, commercial towers and plazas.

So keeping in view the whole discussion further research on seismic analysis of shear wall is carried out.

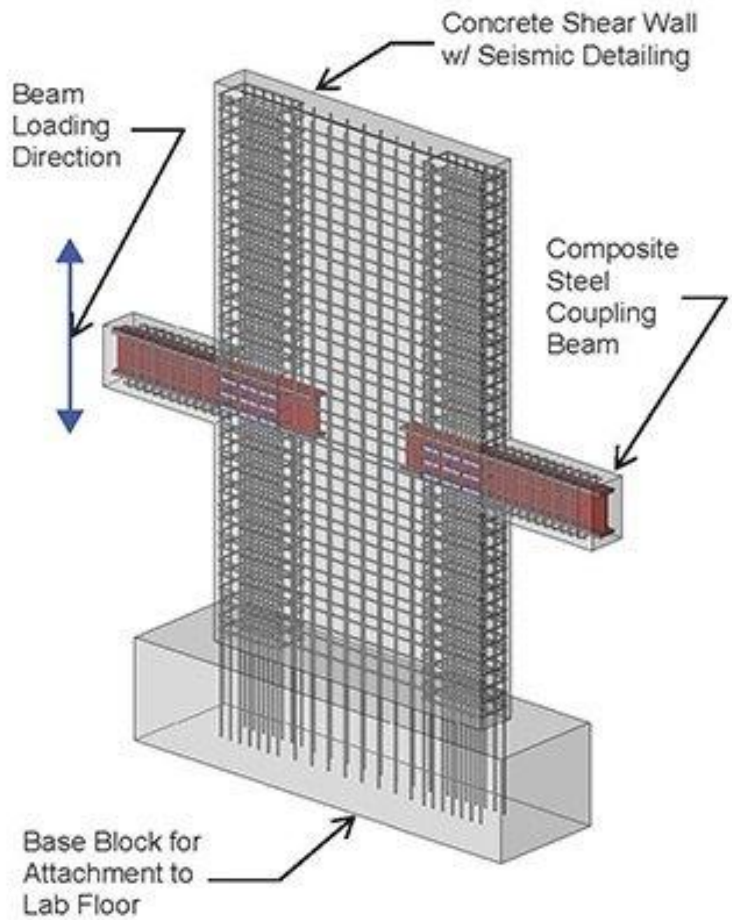


Figure 1.1 Typical Shear Wall

1.2 Problem Statement:

Shear wall construction is efficient way to provide safety against earthquakes. Performance of shear walls is highly dependent on its positioning and shear wall combinations. So, to achieve a combination of shear walls for better performance under earthquake, thorough study and analysis is required.

1.3 Work Procedure:

To check the seismic performance of an RCC frame Structure following steps have carried out.

- Selection of a Building
- Structural Modeling of selected Building
- Gravity analysis of Building
- Results of Gravity Analysis
- Pushover analysis of building with different models
- Compilation of Results.
- Conclusions and Recommendations

1.4 Aims & Objectives:

The focal **aims** of this research work are as follows;

- Study comparison of seismic response of structure with and without shear walls.
- This study will help the design experts to seismically analyse the existing buildings and new buildings.
- This study will also help the professionals about the idea what is the suitable arrangement of shear walls in a building.

The main **objectives** of this research work are as follow;

- Developing a basic model of slected building on SAP2000
- Gravity analysis of Model.
- Pushover Analysis of Model with different arrangements of shear wall

1.5 Utilization:

It can be utilized in the construction of commercial plaza's, buildings and residential buildings in earthquake vulnerable areas. After analyzing the conclusion of this project, it can be determined that which arrangement of shear walls is beneficial to the structure. It can also be used for seismic analysis of a constructed building as well as new buildings in the earthquake prone areas and seismic zone 3 such as KPK , FATA , PUNJAB and Federal Territory of Pakitan.

2 LITERATURE REVIEW

2.1 Introduction:

Structural engineers, building authorities and general public have realized that the evaluation of seismic vulnerability of the built environment is a matter of high priority due to catastrophic effects observed in recent earthquakes. The demand from the public is more rapid than the capability of the technical community to adequately manage it. This is because a lot of work has to be done towards the improvement of the seismic performance of buildings and structures. The lack of analysis of existing and new buildings is the major issue regarding disastrous effects of earthquakes. A lot of research has carried out to improve seismic performance of buildings in different countries. Conclusions of some research papers are listed below which will help us in carrying out our research project.

2.2 Nonlinear Static Pushover Analysis of an Eight Story RC Frame-Shear Wall Building in Saudi Arabia:

M. K. Rahman, M. Ajmal & M. H. Baluch King Fahd University of Petroleum & Minerals, Dhahran, Saudi Arabia, Z. Celep Istanbul Technical University, Istanbul, Turkey

Conclusions:

After studying this research paper we deduced that pushover analysis of the Madinah Municipality building showed the building is deficient to resist seismic loading. Formation of hinges clearly shows that the members of the building are designed purely for gravity loads as with a small increment of displacement, most of the members start yielding. Pushover curves show non-ductile behavior of the building, because almost all the seismic load is carried by the shear walls and at very small displacement, hinges start forming in shear walls. This indicates that strengthening of the shear walls in the building is required. The performance points of the building in positive and negative x-directions are 0.094m and 0.097m based on actual response spectra available for the Madinah area.

2.3 Seismic Evaluation of Reinforced Concrete Frames Using Pushover Analysis

Sofyan. Y. Ahmed, (Ph.D.) Civil Engineering Department, Mosul University, Mosul, Iraq

Conclusions:

The nonlinear static (Pushover) analysis has been utilized for the evaluation of an existing reinforced concrete building frame, in order to examine its applicability. The procedure showed that the frame is capable of withstanding the presumed seismic force with some significant yielding. The main conclusions can be drawn as follows:-

1. Sequence of formation of plastic hinges (yielding) in the frame members can be clearly seen in the beams only. The building clearly behaves like the strong column weak beam mechanism.
2. Through the comparison between different options of the plastic hinge behavior during the pushover analysis, the plastic hinge formed due to its brittle behavior put it in the greater severity level.

2.4 Pushover and nonlinear time history analysis evaluation of a RC building collapsed during the Van (Turkey) earthquake on October 23, 2011

Ozlem Cavdar • Alemdar Bayraktar

Conclusions:

Static pushover and nonlinear time history analyses were used to evaluate the seismic performance of the building collapsed during the Van earthquake on October 23, 2011.Pushover analysis, time history analysis were used to determine global displacements of the building corresponding to the performance levels considered above. It is investigated in situ after the earthquake that insufficient reinforcement and detailing, poor workmanship and low concrete quality can result in this performance level of the structure. In addition to these, the results from linear analysis and pushover analysis show lower damage ratios for the 1st story beams and columns than those of the nonlinear dynamic analysis.

2.5 PUSHOVER ANALYSIS OF A 19 STORY CONCRETE SHEAR WALL BUILDING

Rahul RANA1, Limin JIN2 and Atila ZEKIOGLU3

Conclusions:

After complete study of this research paper it was concluded that pushover analysis was performed on a nineteen story concrete building with shear wall lateral system and certain unique design features. Utilizing the results from this analysis, some modifications were made to the original code-based design so that the design objective of Life Safety performance is expected to be achieved under design earthquake.

2.6 Effect of shear wall location in buildings subjected to seismic loads

Prof. Jayasree Ramanujan1, Mrs. Bindu Sunil2, Dr. Laju Kottallil3, Prof. Mercy Joseph Poweth4
1Department of Civil Engineering, M.A. College of Engineering, Kothamangalam, India.

Conclusions:

From the present investigation and the results obtained it can be concluded as following:

1) In medium high rise buildings (i.e. greater than 10 storey's) provision of shear walls is found to be effective in enhancing the overall seismic capacity of the structure.

2) From the comparison of story drift values it can be observed that maximum reduction in drift values is obtained when shear walls are provided at corners of the building.

3) Lateral displacement values obtained from static method of analysis show that shear wall provision along longitudinal and transverse directions are effective in reducing the displacement values in the same directions. Response spectrum analysis results provide a more realistic behavior of structure response and therefore it can be seen that the displacement values in both X and Y directions are least in model with shear wall in core and corners when compared to all other models.

4) The reinforcement requirement in column is affected by the location and orientation of adjacent shear walls and columns, i.e. alignment along weaker or stronger axis for the structure under consideration. Though the demand is fluctuating, it could be seen that the columns situated near to core area show a reduction in steel requirement up to 44.6% when shear wall is provided at the core and 34.7% when shear wall is located at core and corner of the structure.

2.7 Seismic Analysis of RCC Building with Shear Wall at Different Locations

Ashwinkumar B. Karnale1 and D.N. Shinde2 1M

The results found plotted to get actual behavior of structure and to judge the objectives of study. The results and their importance discussed here briefly. From the graph of base shear for 6 story's it clears that, the base shear is maximum for model having shear wall at core of the structure. Base shear is least for structure without shear wall. When we increase the size of shear wall the seismic weight of structure increases and also the natural time period reduced so ultimately base shear increases. The graph of displacement reflects that for structure having core shear wall the displacement is least.

Conclusions:

- The shear wall located at core of building gives deflection within permissible limit but it also yields maximum base shear. Hence, it is more vulnerable to earthquakes.
- The shear wall located at corner of building gives deflection in permissible limit along with minimum base shear. So, it is less vulnerable to earthquake.
- The time period of frame with shear wall is less, therefore, it attracts more base shear compared to bare frame.
- The location of shear wall affects various structural parameters.
- For Shear wall at corners, the L shape is effective location.
- In low rise (6 storey) building, even by providing shear wall at different locations, the structural parameters are still barely affected.

2.8 Design of Multistoried R.C.C. Buildings with and without Shear Walls

M. S. Aainawala 1, Dr. P. S. Pajgade 2

Conclusions:

From above analysis, it is observed that in G+12, G+25, G+38 Storey building, constructing building with shear wall at corner (Model 3) location gives minimum drift and minimum displacement. From all the above analysis and design, it is observed that in G+38 Storey building, constructing building with shear wall at corner (Model 3) is economical as compared with bare frame structure (Model 1). Size of members like column can be reduced economically in case of structure with shear wall as compared to the same structure without shear wall. Variation in column size at different floors in Model 1 affects the storey drift while in case of Model 3 it does not affect the storey drift due to the presence of shear wall. More carpet area will be available in the building as the sizes of columns are reduced when shear wall is provided. Less obstruction will be there because of reduced size of column and provision of shear wall. As per analysis, it is concluded that displacement at different level in multistoried building with shear wall is comparatively lesser as compared to R.C.C. building Without Shear Wall.

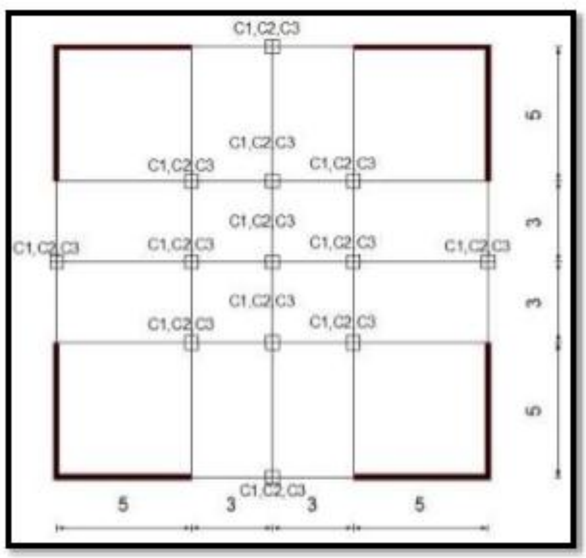


Figure 2.1 Model 4-Most efficient arrangement

2.9 Seismic Analysis of RCC Building with and Without Shear Wall

P. P. Chandurkar1, Dr. P. S. Pajgade2

Conclusions:

From all the above analysis, it is observed that in 10 story building, constructing building with shear wall in short span at corner is economical as compared with other models. From this it can be concluded that large dimension of shear wall is not effective in 10 stories or below 10 stories buildings. It is observed that the shear wall is economical and effective in high rise building. Also observed that

- Changing the position of shear wall will affect the attraction of forces, so that wall must be in proper position.
- If the dimensions of shear wall are large then major amount of horizontal forces are taken by shear wall.
- Providing shear walls at adequate locations substantially reduces the displacements due to earthquake

3 STRUCTURAL MODELING AND ANALYSIS

3.1 Selection of Building:

There are many types of buildings built and under construction in Pakistan. Islamabad being the capital of Pakistan is the hub of commercial, national and international activities. The city contains numerous high rise buildings.

We have selected a five story building for our analysis which have the characteristics given in Table 3.1 which is under construction. These types of buildings are very popular now a days and a lot of construction like this is being carried out in Islamabad DHA Phase-2 and other regions of Islamabad. This building was chosen due to its asymmetry bay. Moreover, it also has a lift located in critical position. Analysis of this type of building is necessary to check seismic performance.

Figure 3.1 Elevation of Selected Building

3.1.1 Building Characteristics:

The following table shows the main characteristics of our selected building. And the plane view of building is shown in fig 1.2. In this figure red portion shows elevator.

Table 3.1 Building Characteristics

Size of Model	100'x72'(26.5 marlas)
No. of Stories	5
Storey Height	12'
No. of Bays in X-Direction	5
No. of Bays in Y-Direction	4
Beam Size	12"x18"
Plinth Beam Size	9"x18"
Column Size	18"x18"
External Wall Thickness	9"
Lift Thickness	12"
Hall Internal	Partition
Reinforcement	#9 main bars, #3 stirrups
Cover	2.4"
Longitudinal Spacing	6"
Shear Wall Thickness	12"

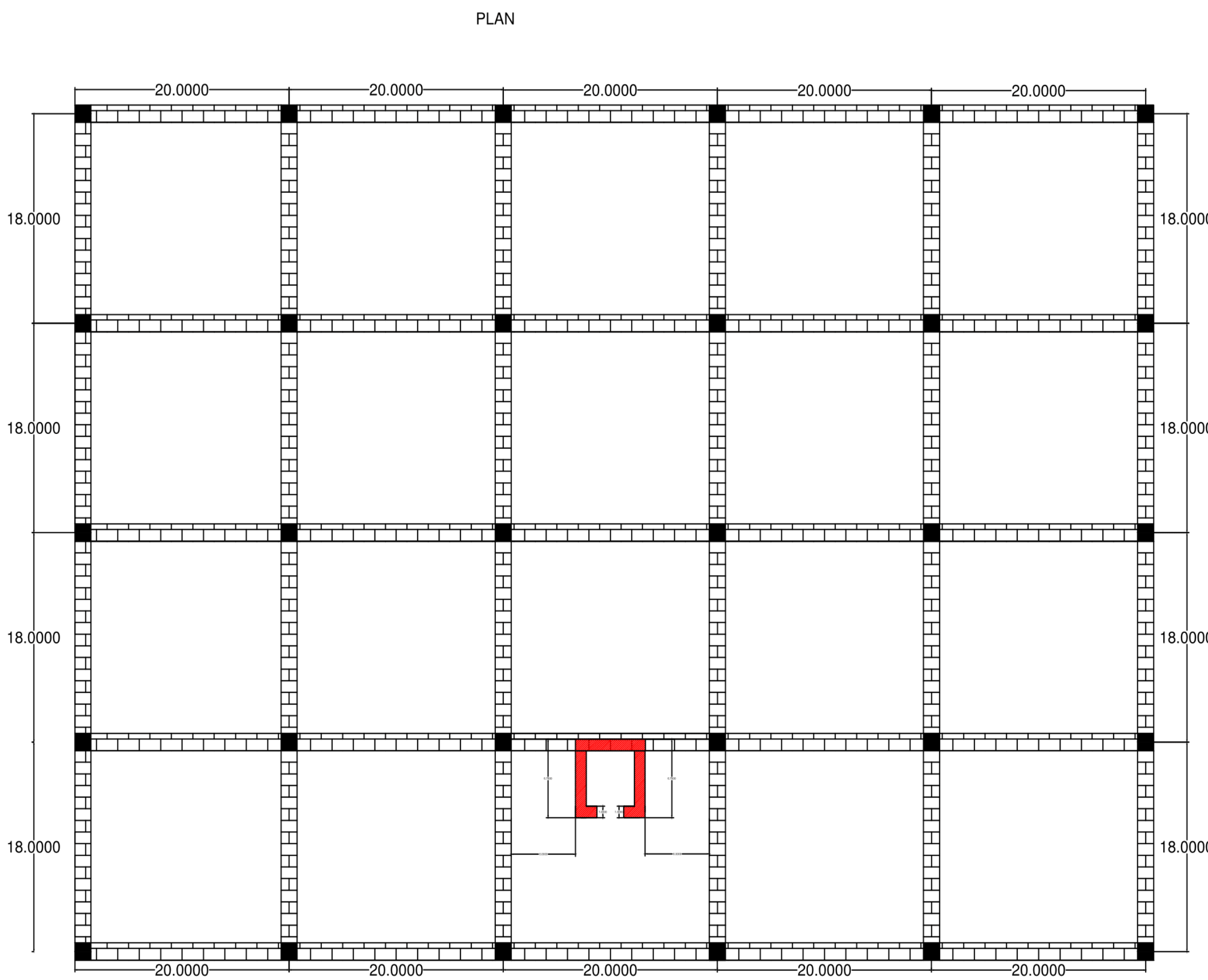


Figure 3.2 Plane View of Building

3.2 Gravity Analysis:

Gravity analysis is basically the analysis or examining the behavior of a structure under gravity loads. In order to perform push over analysis it is necessary that the structure passes under gravity loading. Passing of a structure means, that the structure remains under allowable limits and is safe.

Gravity loads are the vertical forces that act on a structure. The weight of the structure, human occupancy and snow are all types of loads that needs to have a complete load path to the ground. Engineered structures are made up of multiple types of members that connect in order to transfer the loads from the top to the bottom of the structure. Loads in any building have to travel from the roof and upper floors down to the ground. This is termed as "*Load Path*." which need to be continuous. Each consecutive member needs to support itself and the previous members and loads that connect to the floor slab is designed to support the imposed gravity load. The load path mechanism is explained below

1. This load travels from the floor slab to the beams that support it.
2. Upon reaching the beam, the load travels to the end of a beam, which is connected to a girder.
3. This girder is supporting the accumulated loads from the floor slab and beams and transmits the load to a connecting column.
4. The load then travels down the column to the foundation and is distributed to the ground.

3.2.1 Development of basic Model:

Whole of our work done was on SAP2000, the initial step was to build a basic 3D model on SAP which was carried out in the following way.

The first step was to define grids that is basically assigning coordinates or positions. The initial no. of grids in X, Y, Z direction were 5,6,7 respectively and the spacing between them was 20', 18', 12' . These grid lines are then modified according to our building requirement.

After that, the next step was to define materials which were given in building characteristics and then next step was to define frame sections in accordance with the actual building. Frame sections consisted of, Beams measuring 12" x 18", plinth beam of size 9" x 18" and the columns. The building had same columns throughout each measuring 18" x 18". These all were defined as rectangular concrete sections. The section properties and frame property/stiffness modification factors were set to default except for beams the torsional constant was set to 0.25 instead of 1 because we want to set beams to not take torsion so much for better analysis. The concrete reinforcement were defined. For that main bars were provided as of size #9 and confinement bars were provided of size #3. The cover for main bars was 2.4" and longitudinal spacing given was 6". At the end of defining frame sections reinforcement to be designed option is selected. Proceeding further we defined area section which were slab and elevator. These were defined as shell-thin. Slabs assigned were off 6'' throughout and Elevator thickness was kept constant at 12". The remaining properties were set to default.

The next step after defining Area sections was meshing, we did auto meshing, and that is basically to break a section into smaller parts for more uniform distribution of loads and ultimately more accurate results. Proceeding further we assigned mass source from loads i.e. from dead and we also assigned rigid diaphragm in Z direction which we basically did to prevent vertical movement of floor during an earthquake or any other lateral load. The main function of a diaphragm is to basically transfer horizontal forces to vertical members.

3.2.2 Input Parameters for Gravity Analysis:

The input parameters for gravity analysis is given in table 3.1. After modeling the final model for gravity analysis is shown in fig1.4.

MODELLING INPUTS			
MATERIAL		Concrete	4000 psi
		Steel	Grade 60
MODELLING	GRIDS	X	6
		Y	5
		Z	7
	SECTIONS DEFINED	Frame	Beam=12"x18"
		Area	Slab=6",
			Shear Wall=12"
		Meshing of Area Objects	
DIAPHRAGM		Rigid	Global Z Direction and different for each story
LOADS		Dead	
		live	

Table 3.2 Modeling Inputs for Gravity Analysis

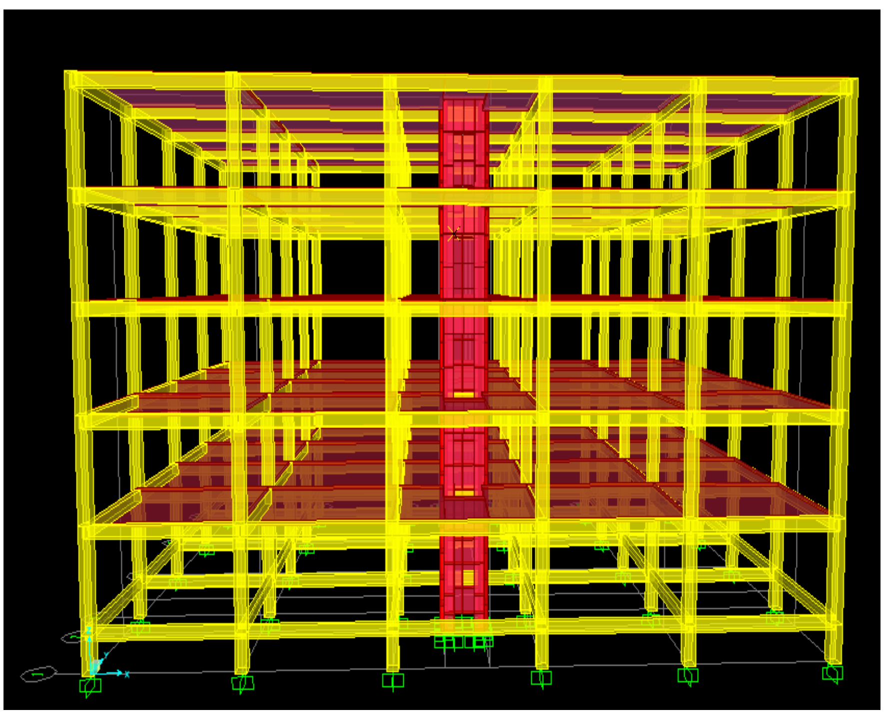


Figure 3.3 SAP 3D Model

3.2.3 Results of Gravity Analysis:

The first thing we did for gravity analysis was to assign load combinations which were

Combo-1= 1.4D

Combo-2= 1.2D + 1.6L

Combo-3= 1.0D +1.0L

The 1st two combinations are for gravity analysis and third combination is basically for foundation design.

The loads were assigned according to ACI code as following:

For external beams we have wall load of thickness 9".

UDL for external beams= 900 lb/ft.

For Roof Slab:

DL= self + 55psf

LL=30psf

For all other Floor Slabs:

DL=Self +76psf

LL=50psf

The self-weight multiplier for Dead Load was kept at 1 to set software to calculate the self-weight of slab itself.

After these, finally the analysis was run and the results were checked and noted. All columns and beams were checked and it was made sure none of these were **Over Stressed**. For this purpose we have selected the most critical planes to check our gravity analysis results which were for XZ plane Y=72' and for YZ plane X=60'.

Apart from this, reinforcement in columns were noted which were made sure to be within allowable limits that is 1%-8%. Deformations were noted at each junction. Curves and Values for Moments and shear force in 2 and 3 direction was obtained. The following figures shows deflected shape, reinforcement %age, shear & moment.

Max Values are given below:

V2= -42.534 kips, M2= -67.3157 kip-ft

V3= -10.887 kips, M3= -98.5446 kip-ft

P= -557.85 kips, T= -12.1778 kip-ft

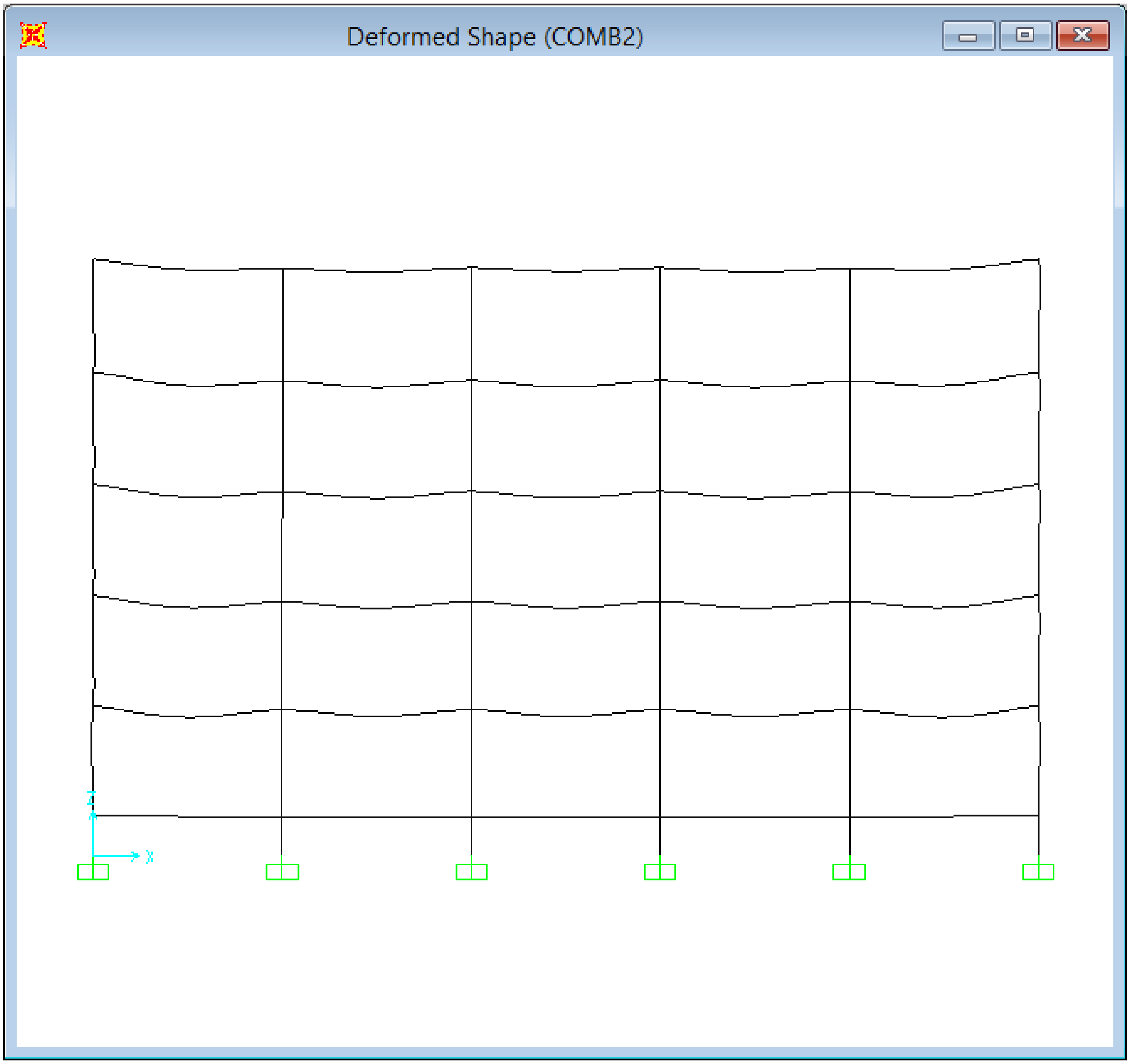


Figure 3.4 Deformed Shape XZ Plane

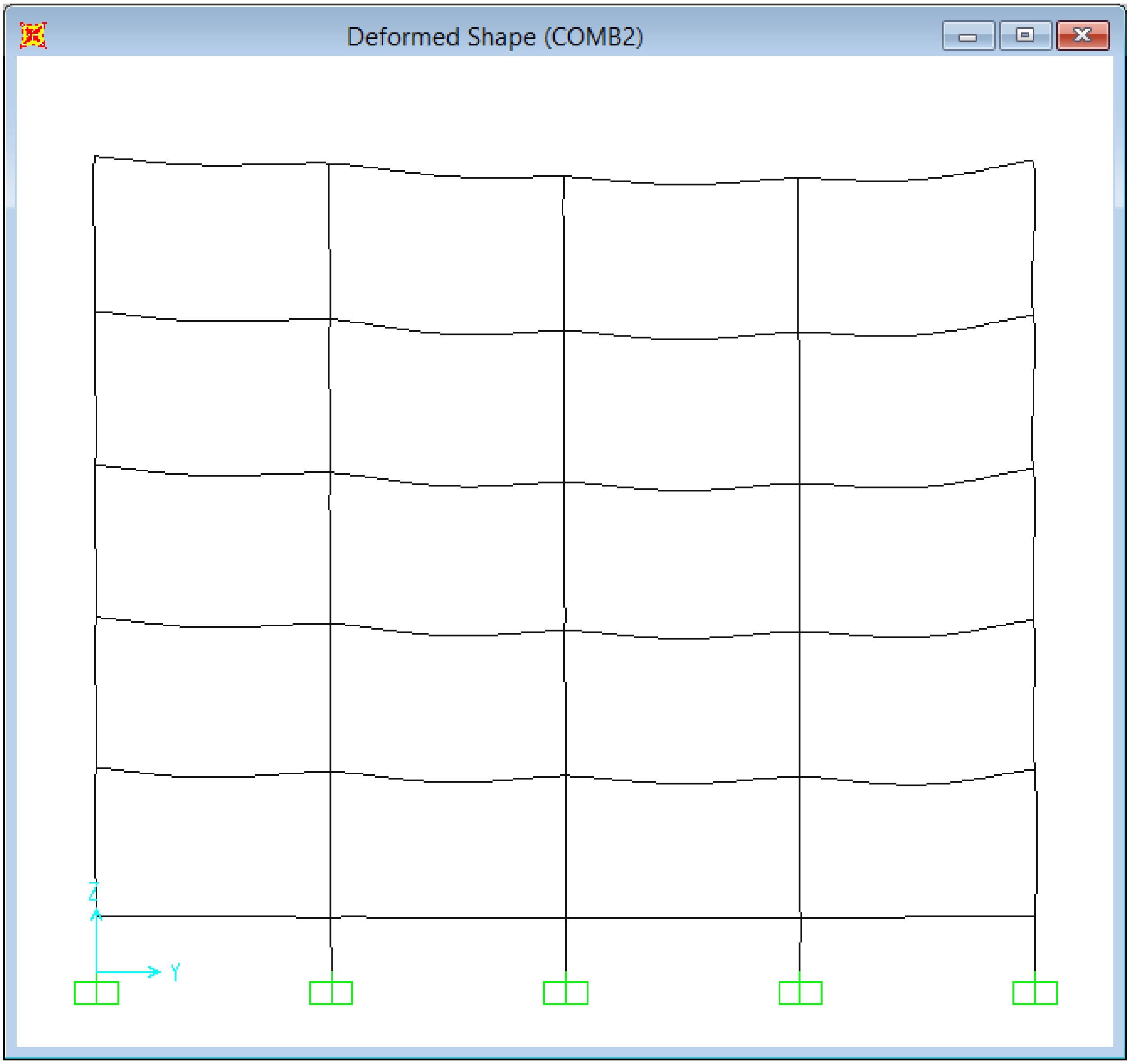


Figure 3.5 Deformed Shape YZ Plane

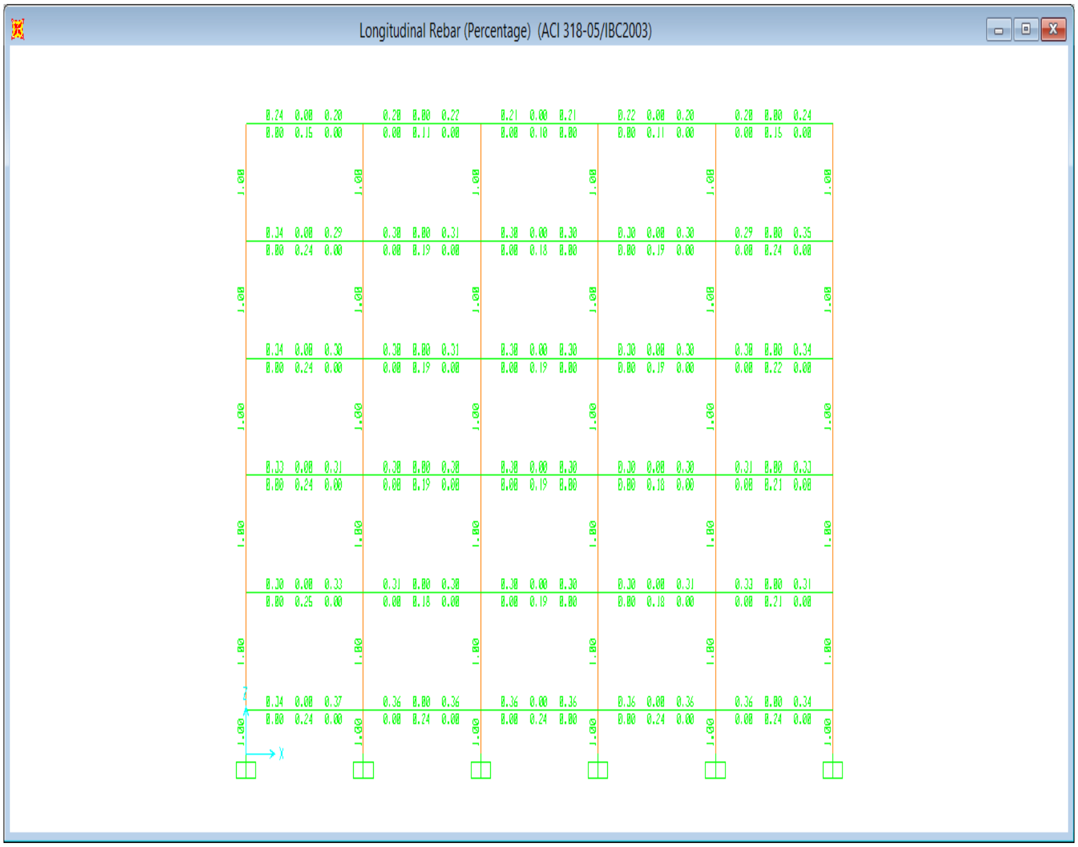


Figure 3.6 Rebar Percentage in XZ Plane

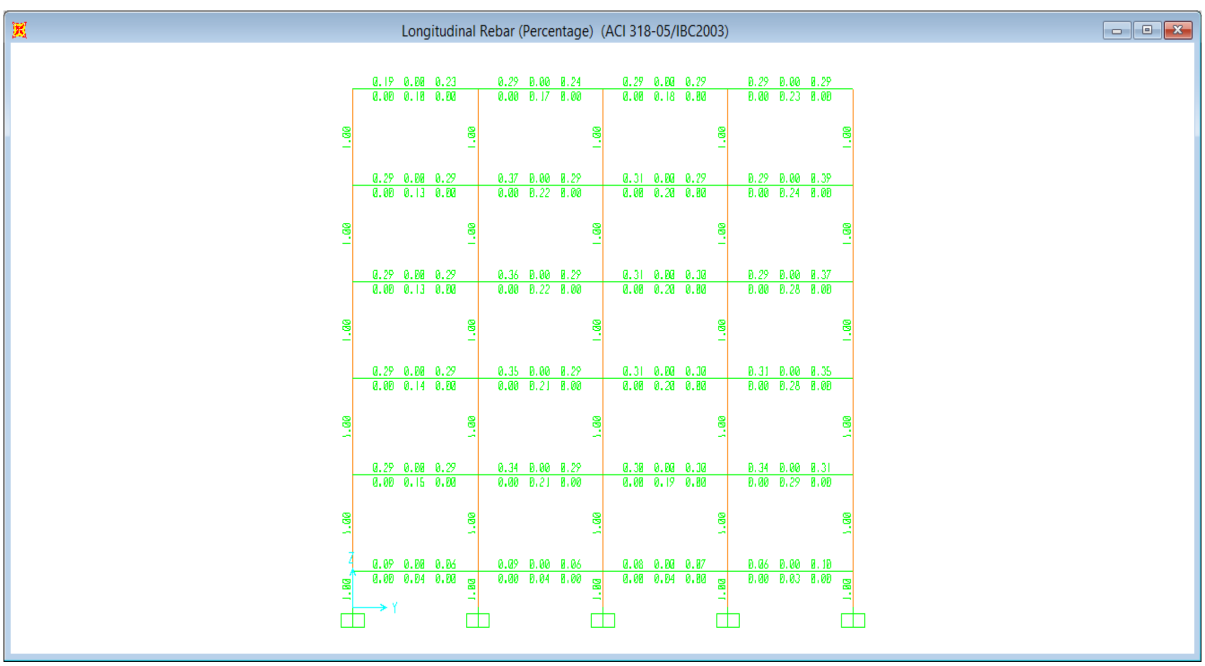


Figure 3.7 Rebar Percentage in XZ Plane

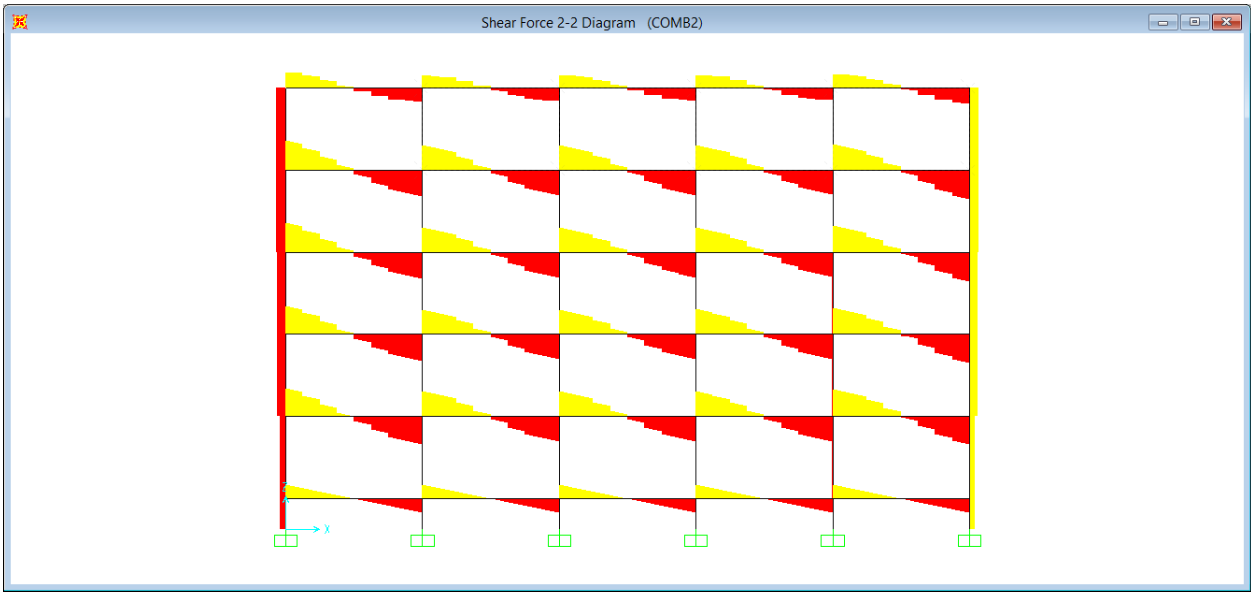


Figure 3.8 Shear Force Diagram for XZ Plane

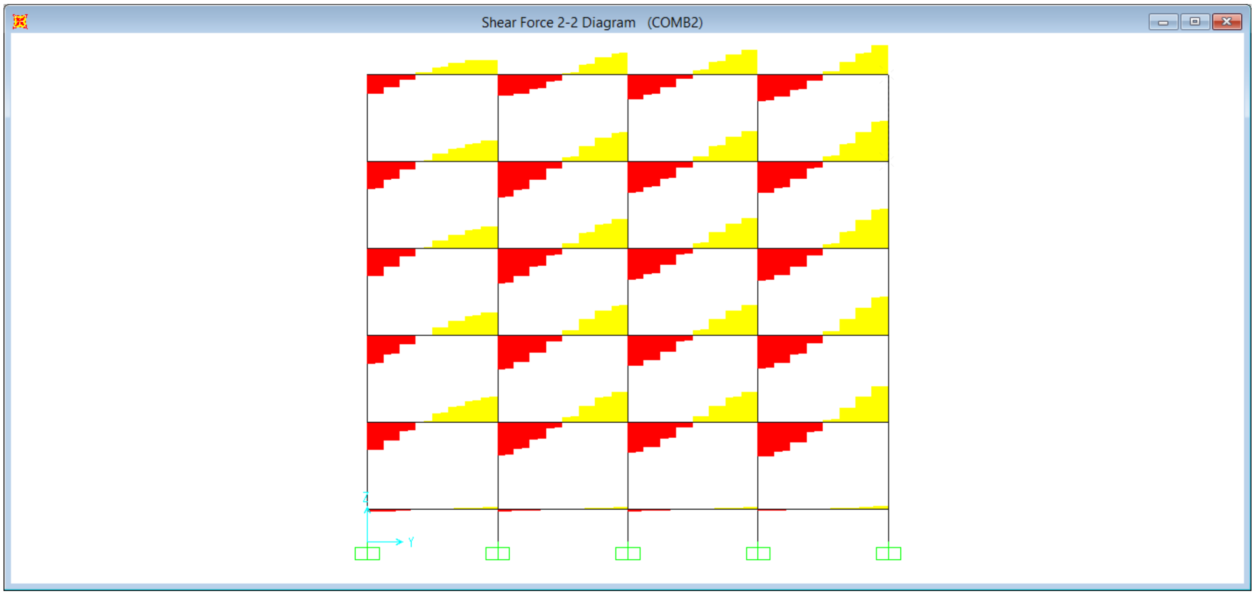


Figure 3.9 Shear Force Diagram for YZ Plane

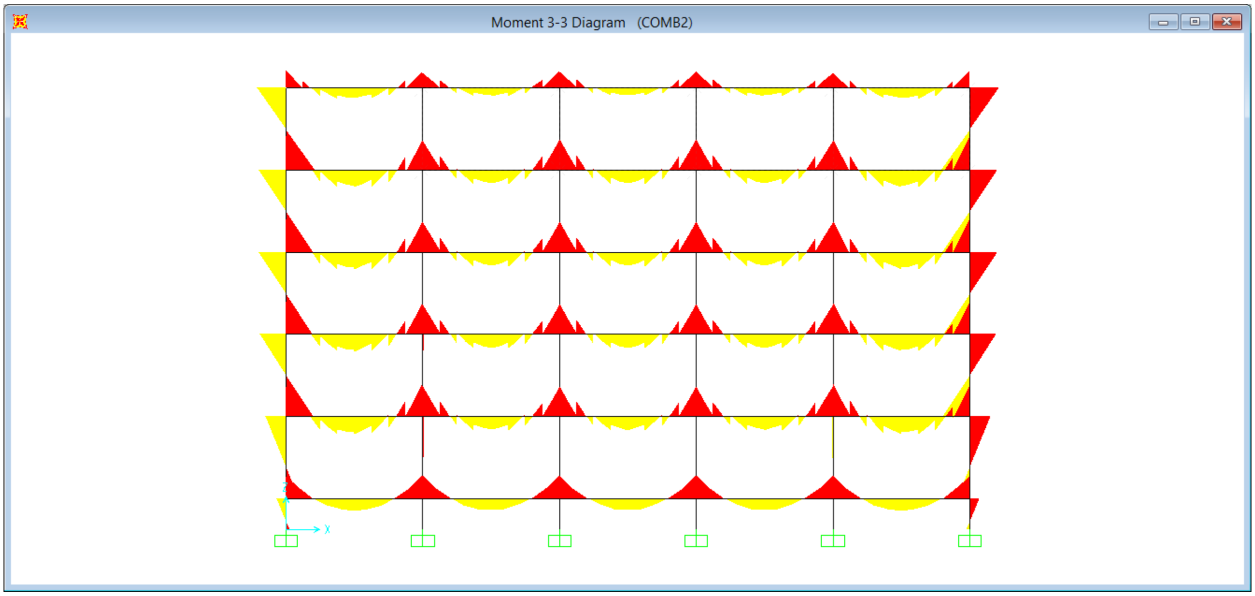


Figure 3.10 Bending Moment Diagram for XZ Plane

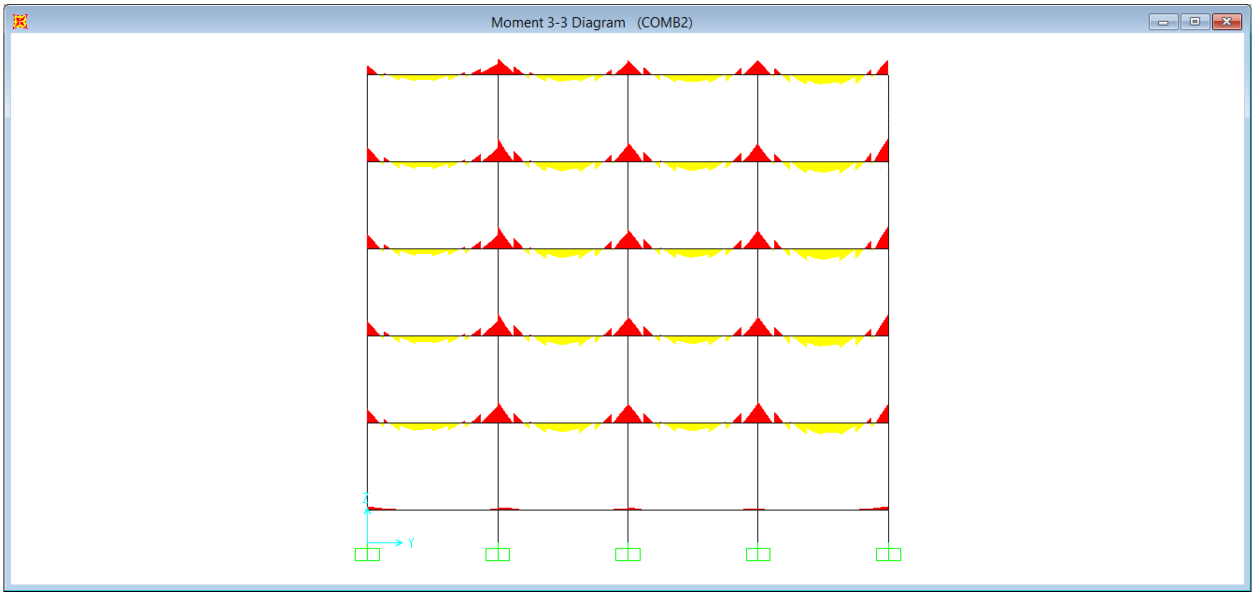


Figure 3.11 Bending Moment Diagram for XZ Plane

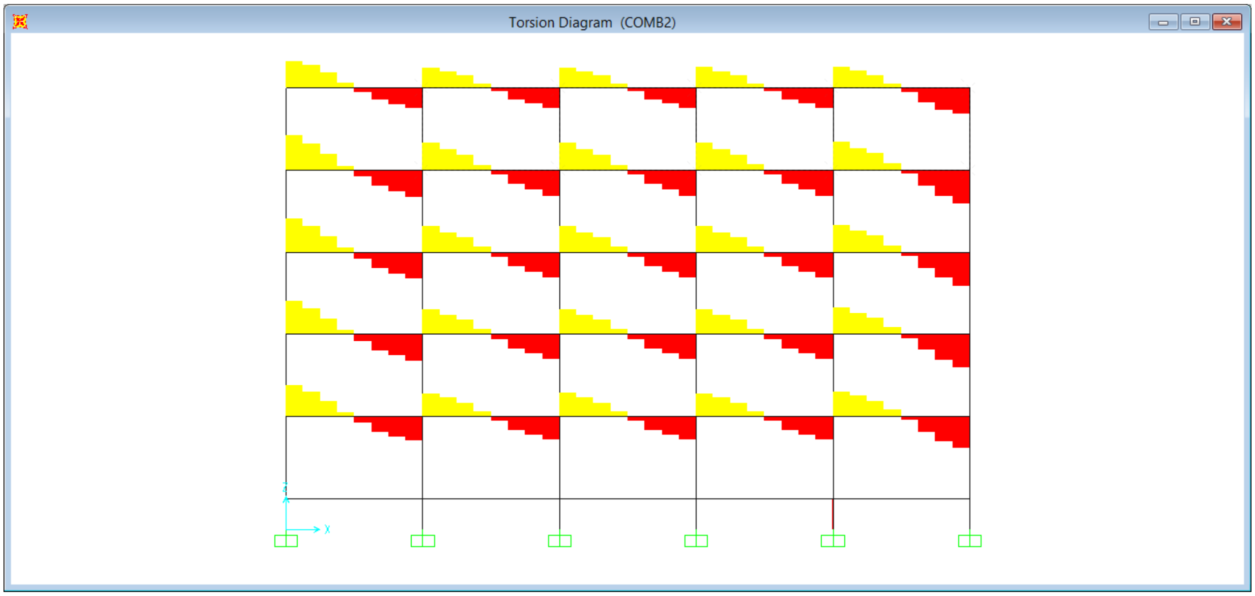


Figure 3.12 Torsion Diagram for XZ Plane

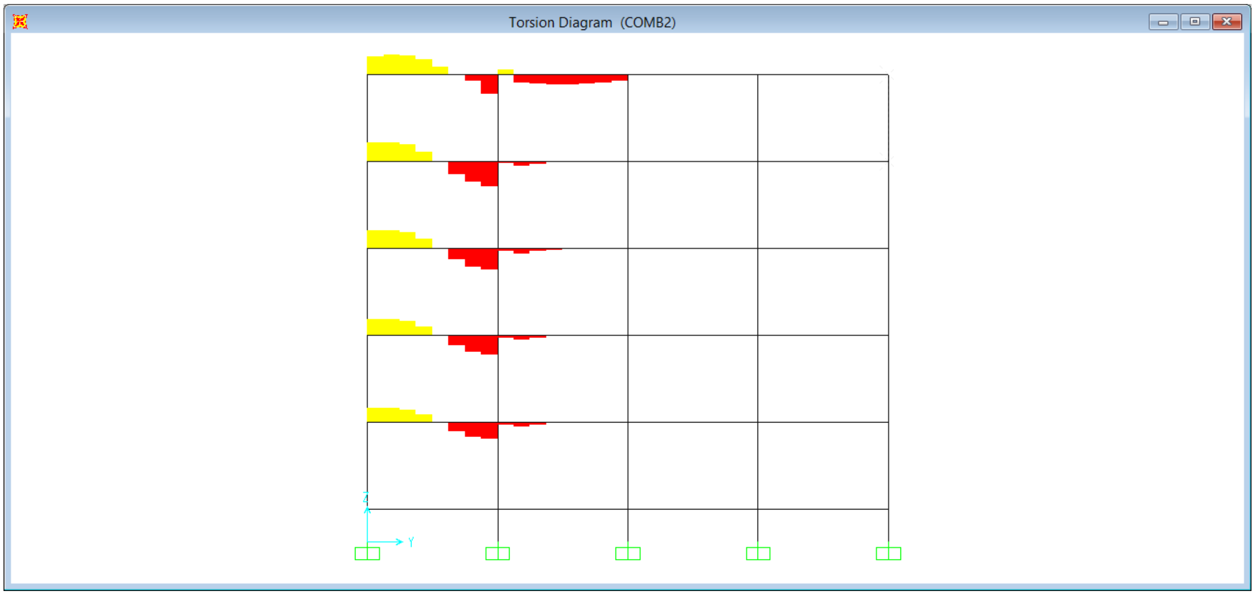


Figure 3.13 Torsion Diagram for XZ Plane

3.3 Pushover Analysis:

3.3.1 Inelastic Methods of Analysis:

Structures behave inelastically under a strong earthquake so inelastic analytical procedures are required for accurate analysis. Inelastic analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis (pushover analysis). The inelastic time history analysis is the most accurate method. However, the use of inelastic time history analysis is limited because dynamic response is very sensitive to modeling and ground motion characteristics. It requires proper modeling of cyclic load deformation characteristics and availability of a set of representative ground motion records. Also the computation time for input preparation and interpreting voluminous output make the use of inelastic time history analysis impractical for seismic performance evaluation.

Inelastic static analysis, or pushover analysis, has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. Inelastic static analysis procedures include Capacity Spectrum Method (ATC 40), Displacement Coefficient Method (FEMA 273) and the Secant Method. Displacement-based procedures provide a more rational approach to these issues compared to force-based procedures by considering inelastic deformations rather than elastic forces. The analytical tool for evaluation process should also be relatively simple which can capture critical response parameters that significantly affect the evaluation process.

3.3.2 Description of Pushover Analysis:

Pushover analysis is an approximate nonlinear analysis method in which the structure is subjected to monotonically increasing lateral loads until a target displacement is reached. A mathematical model of the building which includes load-deformation diagrams of all lateral force resisting elements is generated and gravity loads are applied initially. A predefined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield and the structural model is modified to account for the reduced stiffness of yielded members. The lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve. Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines because it is conceptually and computationally simple. Pushover analysis determines the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure. Pushover analysis determines the force demand for brittle members and deformation demand for ductile members. It identifies the weak links in the structure and is therefore very useful for efficient retrofitting. Pushover analysis exposes the design weaknesses that may remain hidden in an elastic analysis. These are story mechanisms, excessive deformation demands, strength irregularities.

3.3.3 Types and Procedures of Pushover Analysis:

Pushover analysis can be of **two types**. It can either be performed as **force-controlled** or **displacement-controlled**. Force-controlled is used when the load is known (such as gravity loading). In displacement-controlled procedure is used where the magnitude of applied load is not known in advance. The load is increased until the control displacement reaches a specified value. Generally, roof displacement at the center of mass of structure is chosen as the control displacement. The internal forces and deformations at the target displacement give inelastic strength and deformation demands which is compared with available capacities to find a performance point. In this study displacement based procedure is used for Seismic Loads and forced based procedure for gravity loads.

Available simplified conventional **nonlinear static procedures** are as follow.

• **Capacity Spectrum Method**, that uses intersection of capacity (pushover) curve and a reduced response spectrum in spectral coordinates (Acceleration Displacement Response Spectrum Format) to find a performance point. The specifications of this method are covered in ATC40.

• **Displacement Coefficient Method** described in FEMA-356 [20] is a noniterative approximate procedure based on displacement modification factors. The expected maximum inelastic displacement of nonlinear MDOF system is obtained by modifying the elastic spectral displacement of an equivalent SDOF system with a series of coefficients. The procedure proposed by Newmark and Hall is based on the estimation of inelastic response spectra from elastic response spectra while displacement modification factor varies depending on the spectral region.

3.3.4 Limitations of Pushover Analysis:

Although pushover analysis has advantages over elastic analysis procedures, however the assumptions for pushover analysis and limitations of current pushover procedures must be identified. The estimate of target displacement, selection of lateral load patterns and identification of failure mechanisms due to higher modes of vibration are important issues that affect the accuracy of pushover results.

In pushover analysis, the target displacement of a MDOF system is estimated as the displacement demand for the corresponding equivalent SDOF system. A shape vector representing the deflected shape of the MDOF system is used to obtain the properties of an equivalent SDOF system. A fixed shape vector, elastic first mode, is used for simplicity without considering the higher mode effects by conventional approaches.

The distribution of inertia forces vary with the severity of earthquake and with time during earthquake since however, in pushover analysis, generally an invariant lateral load pattern is used. The lateral load patterns used in pushover analysis are proportional to product of story mass and displacement associated with a shape vector at the story under consideration. Commonly used lateral force patterns are uniform, elastic first mode, “code" distributions and a single concentrated horizontal force at the top of structure. The invariant lateral load patterns could not predict potential failure modes due to middle or upper story mechanisms caused by higher mode effects. Invariant load patterns can provide adequate predictions if higher mode effects are not significant.

These limitations have led many researchers to propose adaptive load patterns which consider the changes in inertia forces with the level of inelasticity. Although some improved predictions have been obtained from adaptive load patterns, they make pushover analysis computationally demanding and conceptually complicated**.**

3.3.5 Pushover Analysis Using SAP2000:

In our study we have done Pushover Analysis regardless of its limitations because it was easy to perform and interpretation of results was simple.

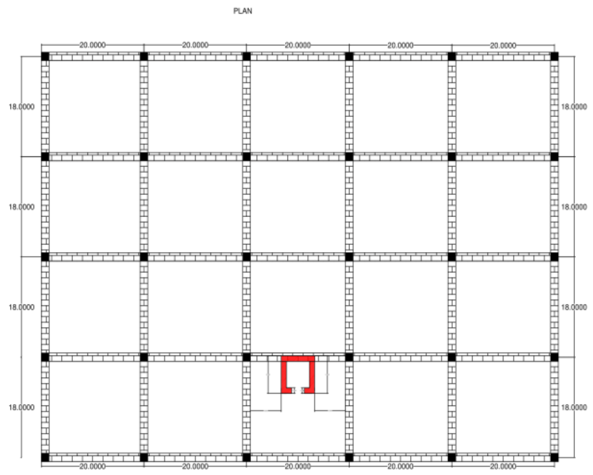
We have performed the Pushover Analysis of our building model in the following way:

- Linear Static Analysis of our Reference Model/Building using Seismic Loading
- Linear Static Analysis on different models which have different arrangements of shear walls
- Redesign of beams or columns if any failed.
- Pushover Analysis using different arrangements of shear wall.

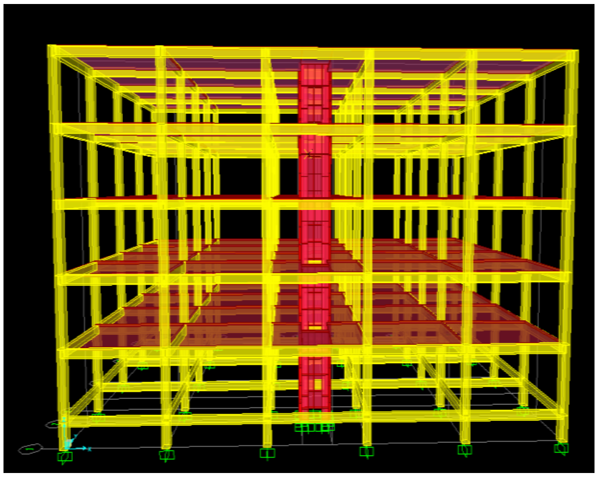
3.3.6 Linear Static Analysis:

In the Linear Static Procedure, the building is modeled with linearly-elastic stiffness and equivalent viscous damping that approximate values expected for loading to near the yield point. In Equivalent static analysis it is assumed that the structure responds in its fundamental mode. The response is read from a design response spectrum, given the natural frequency of the structure. This method work well for low to medium-rise buildings without significant coupled lateral–torsional modes, in which only the first mode in each direction is of significance.

Fig3.14-Fig3.17 shows different models/buildings i.e. Reference, A, B, C. These models have different arrangements of shear walls provided to check the seismic performance.

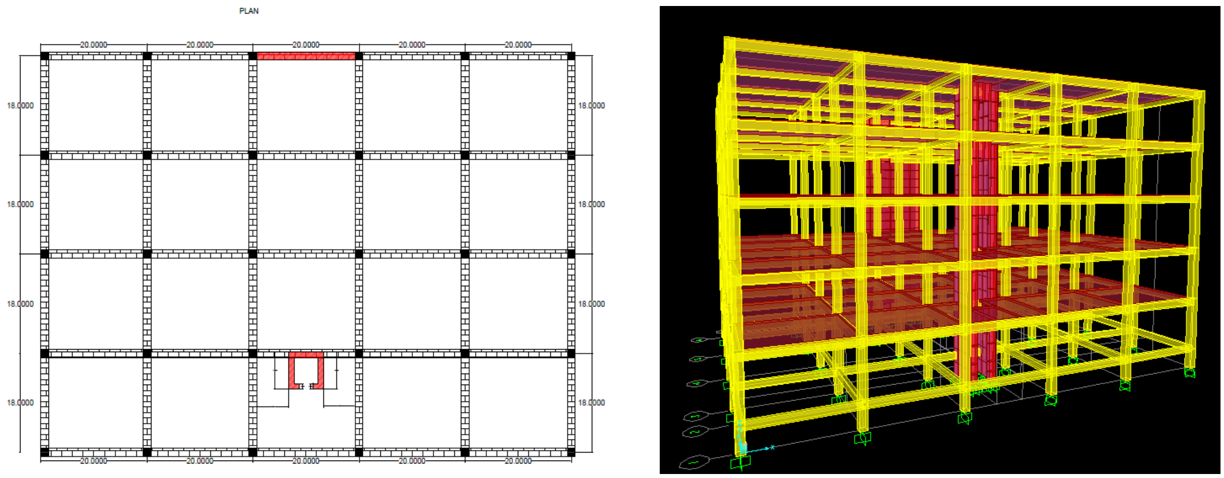


a) Plane View



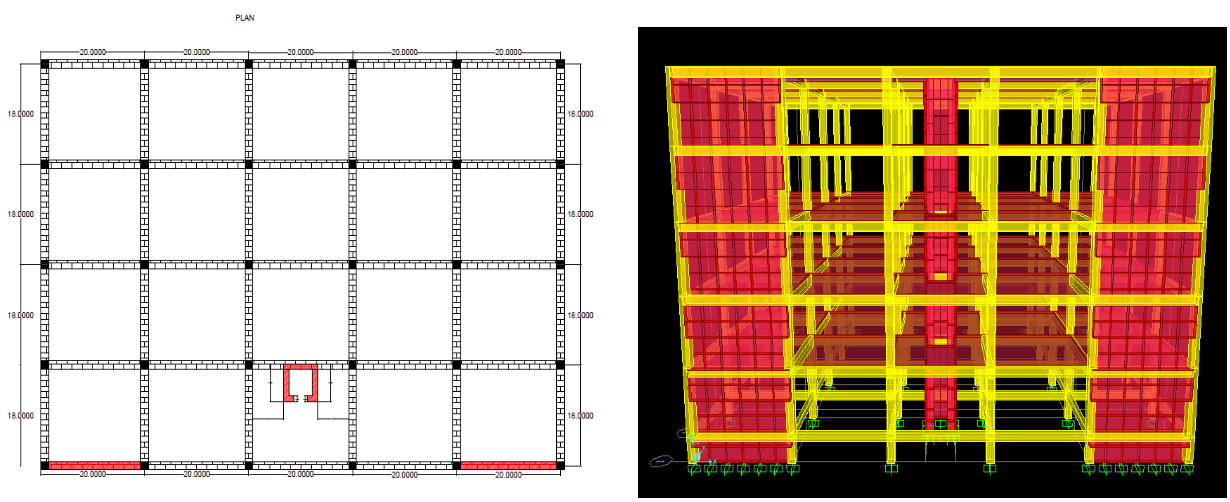
b) 3d Model

Figure 3.14 Reference Building



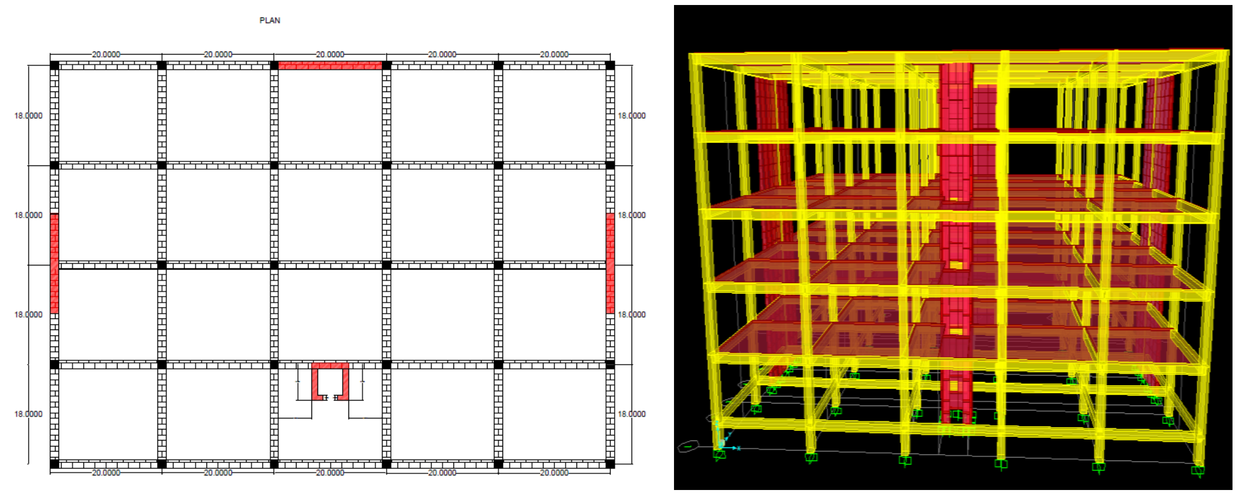
a) Plane View b) 3D model

Figure 3.15 Building A



a) Plane View b) 3D Model

Figure 3.16 Building B



a) Plane view b) 3D Model

Figure 3.17 Building C

After the gravity analysis the next step was to perform linear static analysis which was a preliminary step for Pushover analysis. In this analysis Seismic loads were defined in both directions (i.e. Earthquake in X and Y direction) and additional load combinations of seismic loads were defined which are given in fig 1.8. The reference building was then checked to see which of the frame sections failed in seismic loading and those failed beams and columns, were redesigned by increasing their sizes. Reinforcement, deflection and torsion values are also checked and these were within the specified limits. The same step was performed by providing different arrangements of shear walls i.e. for building A, B, C. At the end those sections were obtained which qualified all the models for seismic loading. For the comparison, we provided those qualified sections in all our four buildings including reference building.

The seismic load patterns are defined and modified according to UBC 97 code. X and Y Directional earthquakes were defined according to global directions. Seismic zone selected was zone 3, soil profile was type-SE and the seismic source type was B. We have selected special moment resisting frame for our seismic loading according to UBC 97 guidelines for which overstrength factor(R) was 8.5 and building importance factor(I) was 1.

The Appendix A shows the guidelines of UBC 97.

3.3.7 Input Parameters for Pushover Analysis:

For pushover analysis the initial input was the hinge assignment. Hinges can be assigned at any location of potential yielding. Uncoupled moment (M2 and M3), torsion (T), axial force (P) and shear (V2 and V3) force-displacement relations can be defined. As the column axial load changes under lateral loading, there is also a coupled P-M2-M3 (PMM) hinge which yields based on the interaction of axial force and bending moments at the hinge location. More than one type of hinge can be assigned at the same location of a frame element.

SAP2000 considers three types of hinge properties. These are default hinge properties, user-defined hinge properties and generated hinge properties. Default hinges and user-defined hinges can be assigned to frame elements. When these hinge properties (default or user-defined) are assigned to a frame element, the program automatically creates generated hinge property. Default hinge properties could not be modified and are section dependent. The built-in default hinge properties for steel and concrete members are based on ASCE 41_13 and idealized flexural hinge criteria. User-defined hinge properties can be based on default properties or they can be fully user defined.

Based on the above discussion we have defined default hinges according to ASCE 41_13 at the both ends (i.e. at relative distance 0 and 1) in beams & columns. These hinges are then overwritten by using hinge over write command. Hinge Assignment step is necessary to check formation of hinges at different levels i.e. IO(Initial Occupancy), LS(Life Safety), CP(Collapse Prevention).

Hinge Assignments Auto (From Tables in ASCE)	
Hinge Type	
M3 for beams	PM3 for columns
Hinge Overwrites	Auto (Subdivide Line Objects)0.02

After the hinge assignment step the 2nd major step was to define Push Load Case. As gravity loads are always present in a building so we set our push load case to start from dead load. Therefore, dead load case is set to non-linear state.

Push load case is defined according to following table 1.8 and it was also set to non-linear. Push load case is applied to only universal X direction so all results are obtained according to this directional push. Scale factor was set to minus one to record positive increments only. Cv and Ca are seismic coefficients i.e. coefficient of acceleration and velocity. There values are taken from Table 16-Q and Table 16-R.

When everything is done pushover analysis was performed which took a lot of time to complete and results were analyzed and discussed.

Table 3.3 Push Load Case Parameters

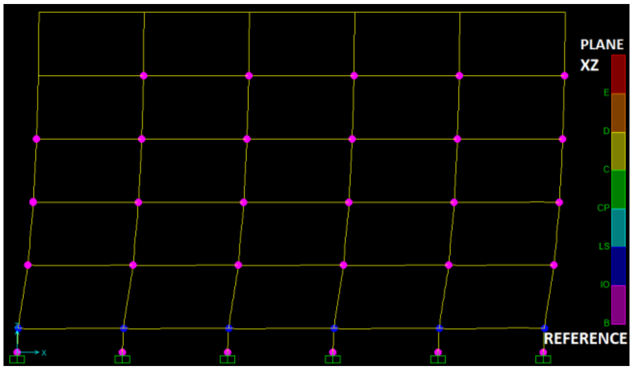
Load Type	Acceleration
Load Name	Ux
Scale Factor	-1
Displacement Controlled	**2.5%** of total storey height=**1.6125'**
constants	Cv= 0.4
	Ca= 0.4

4 RESULTS AND DISCUSSION:

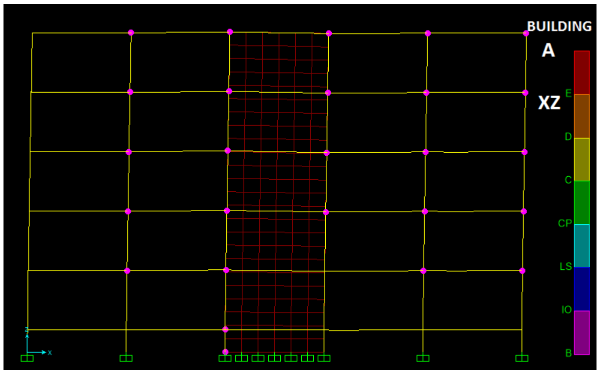
4.1 Hinge Formation:

4.1.1 In XZ Plane:

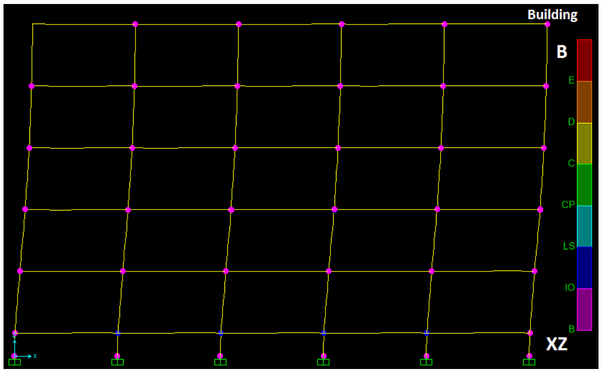
The following figures shows hinge formation in XZ plane for Y=72’ which was a critical plane.



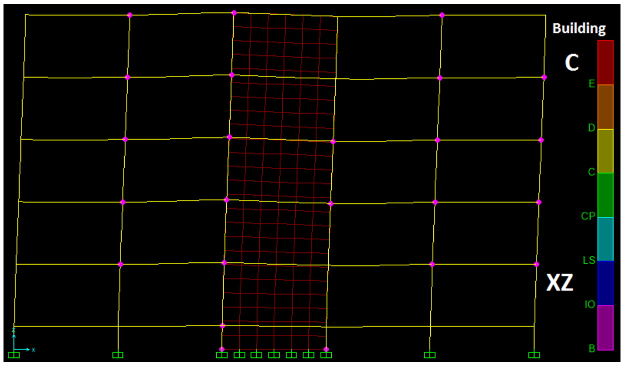
a) Reference Building



b) Building A



c) Building B

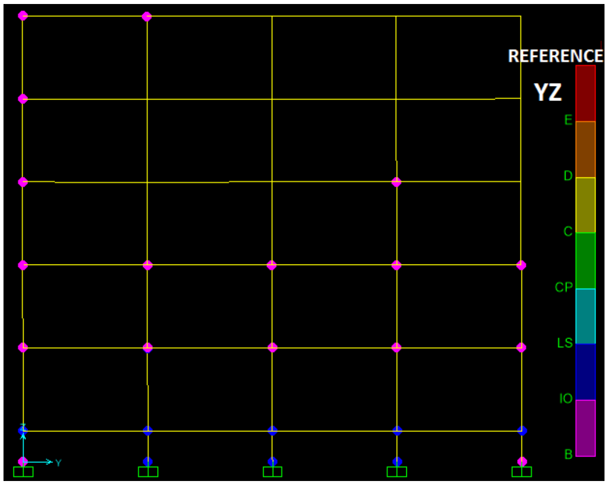


d) Building C

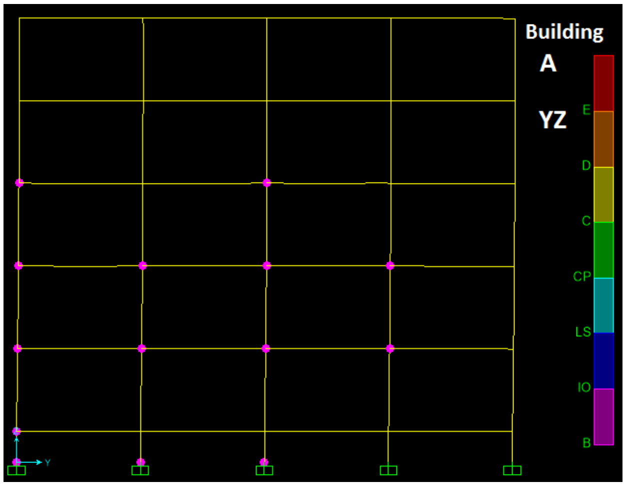
Figure 4.1 Hinge Formation in XZ Plane

4.1.2 In YZ Plane:

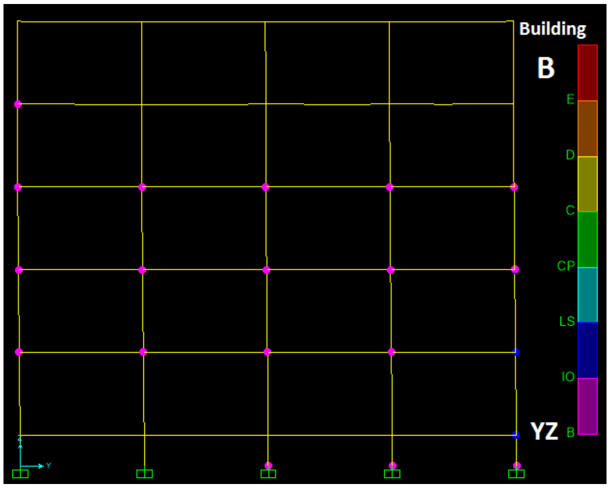
The following figures shows hinge formation in YZ plane for X=60’ which was a critical plane.



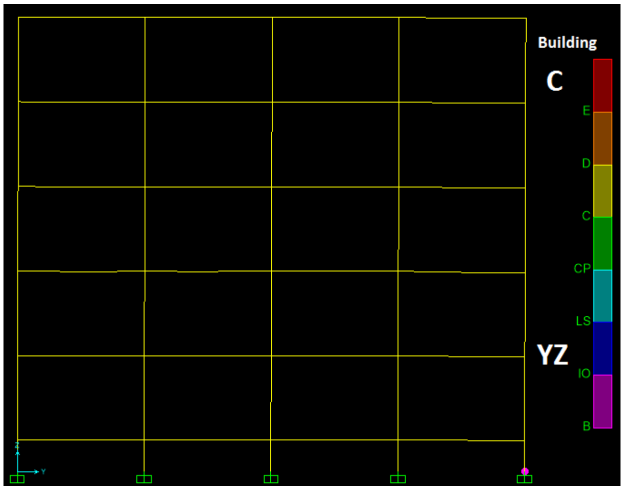
a) Reference Building



b) Building A



c) Building B



d) Building C

Figure 4.2 Hinge Formation in YZ Plane

Table 4.1 Hinge Color Description

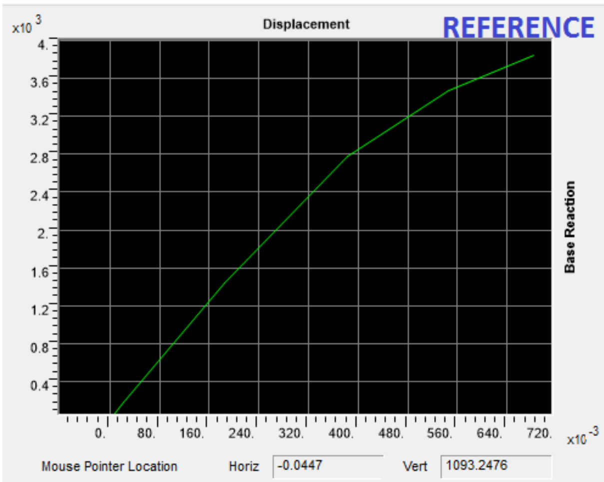
Hinge Color	Description
Pink	Initial Occupancy
Blue	Life Safety
Light Blue	Collapse Prevention

Formation of hinges is shown in the results. The results show that collapse hinges are developing in columns at Basement storey level, which is undesirable. All other hinges are developing in the beams, so Basement story column should be strengthened. Then the building under consideration is strengthened by providing shear walls at different locations. Pushover analysis is run using same load cases and results are viewed.

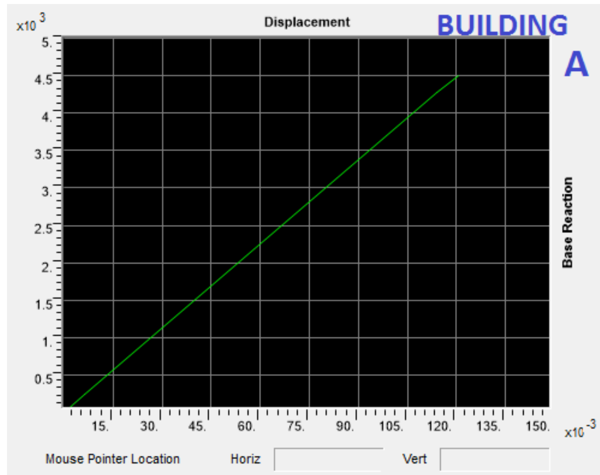
The reference building had developed a lot of hinges. By providing shear walls at appropriate locations (Building A & C), we can clearly see that total no. of hinges have reduced significantly. Besides this, we can also see that the color of hinges changed from blue to pink. In contrast, the building model B, which had 3 shear wall placed in same direction, showed that the no. of hinges increased as well as more hinges were in LS state (blue color) comparative to other building models.

4.2 Base Shear VS Displacement:

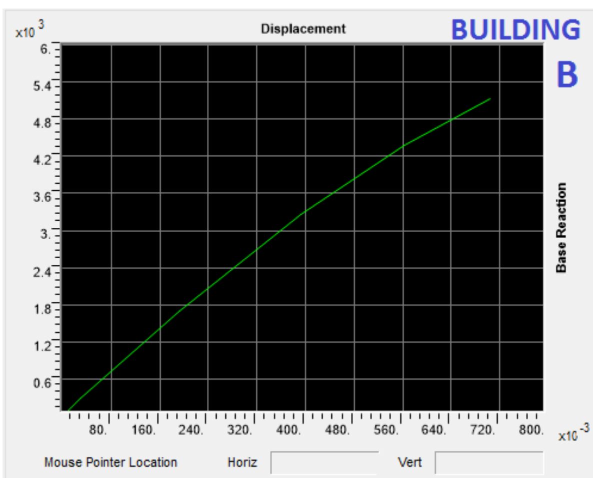
The following figures are the graph of base shear VS displacement for our 4 models.



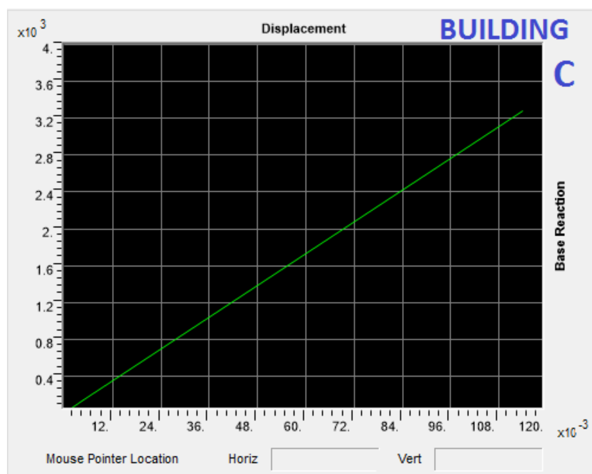
a) Reference Building



b) Building A



c) Building B



d) Building C

Figure 4.3 Base Shear VS Displacement

The graph of base shear VS roof displacement clearly shows that for a specific base shear value the displacement is maximum for building B followed by reference building. On contrary, building C had the least displacement for a specific base shear value as shown by following table.

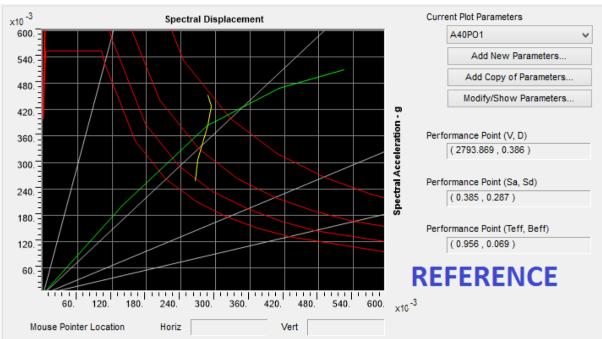
Table 4.2 Base Shear and Displacement Comparison

Model	Base shear(KN)	Displacement(ft)
Reference Building	3000	440(10^-3)
Building A	3000	82
Building B	3000	360
Building C	300	102

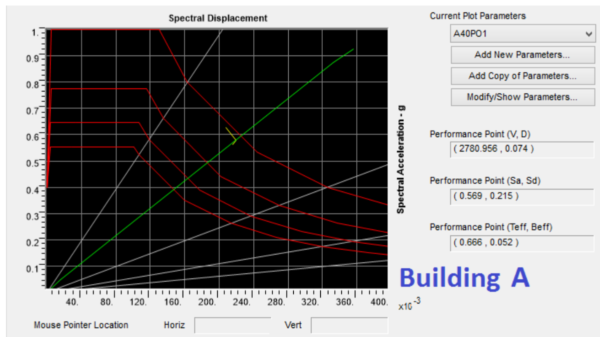
4.3 ATC 40 Capacity Spectrum:

4.3.1 Sa VS Sd:

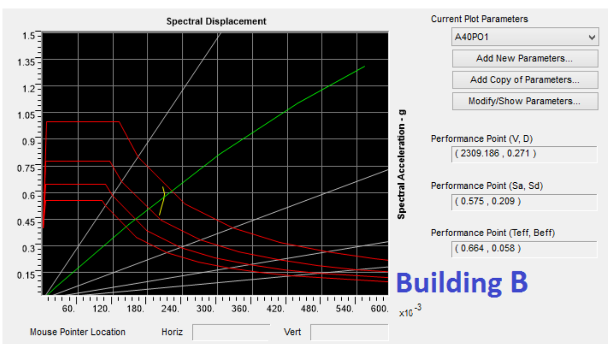
The following figures shows the graph of Spectral Acceleration VS Spectral Displacement for our 4 different models.



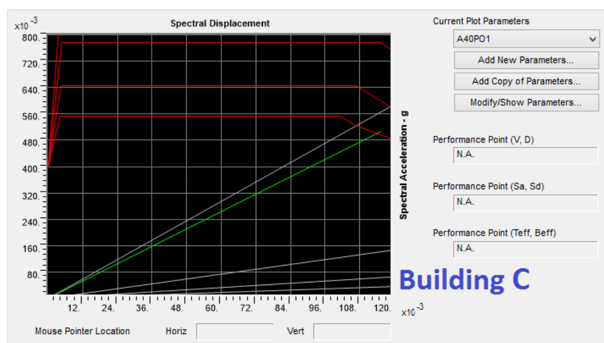
a) Reference Building



b) Building A



c) Building B

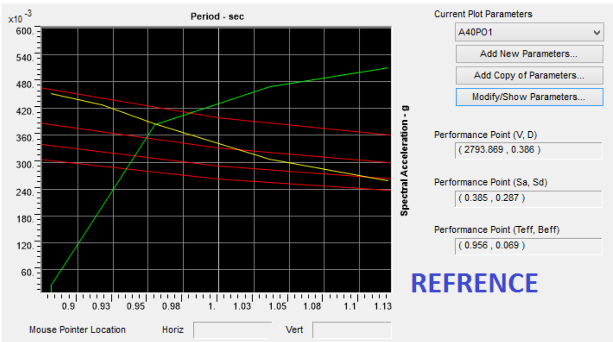


d) Building C

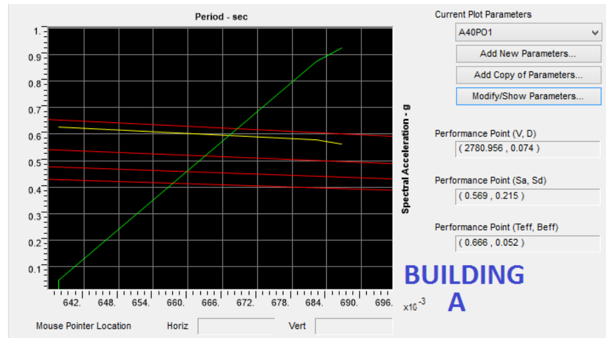
Figure 4.4 Spectral Acceleration VS Spectral Displacement

4.3.2 Sa VS T:

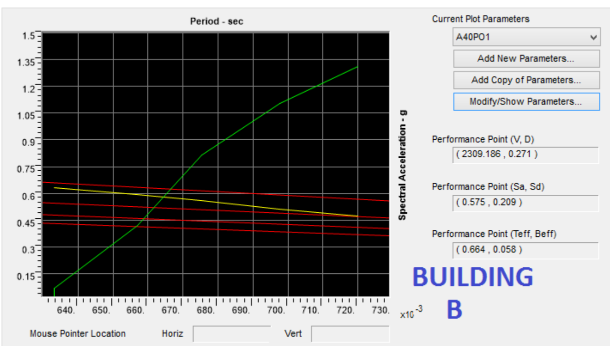
The following figures shows the graph of Spectral Acceleration VS Time Period for our 4 different models.



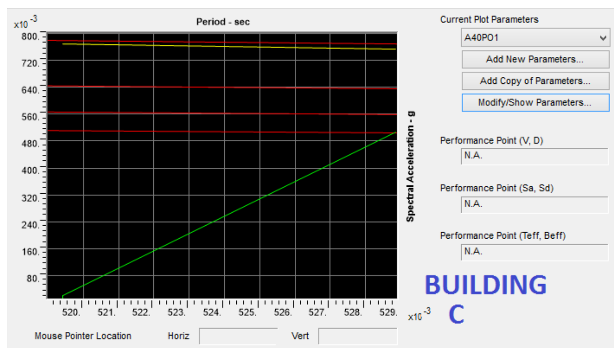
a) Reference Building



b) Building A



c) Building B

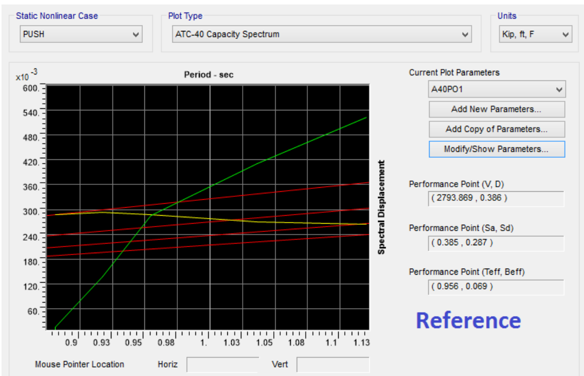


d) Building C

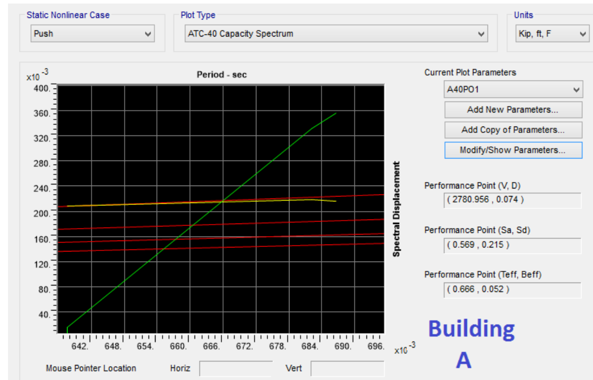
Figure 4.5 Spectral Acceleration VS Time Period

4.3.3 Sd VS T:

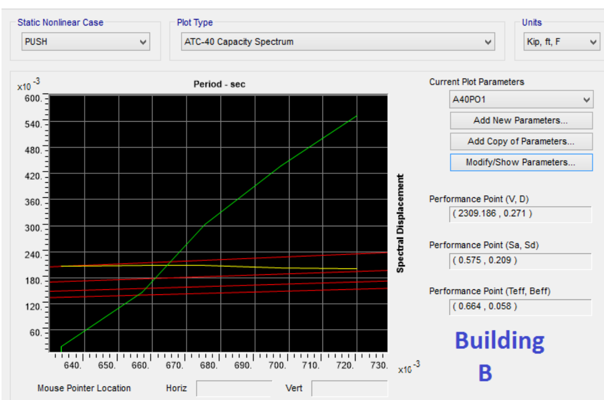
The following figures shows the graph of Spectral Displacement VS Time Period for our 4 different models.



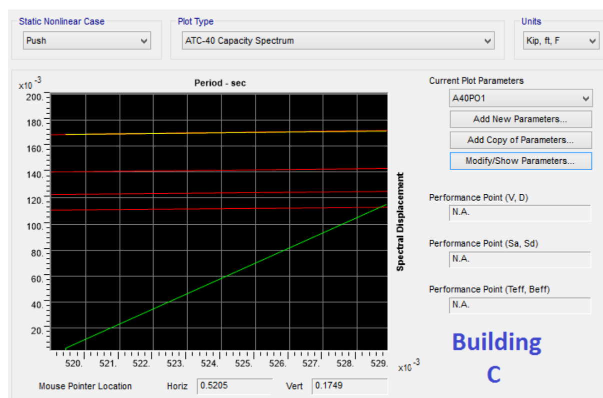
a) Reference Building



b) Building A



c) Building B



d) Building C

Figure 4.6 Spectral Displacement VS Time Periods

The green line shows the capacity whereas the red lines depicts the demand. The intersection between yellow and green line is basically the **Performance Point.** Performance point tells us how building behaves during inelastic range. It includes spectral acceleration, spectral displacement and time period. These three parameters basically governs the performance of a structure.

Spectral Acceleration is basically the peak or max acceleration of an object reached during an earthquake whereas **Spectral Displacement** is the max displacement of the building during an earthquake. A better and more stable arrangement of shear walls in a building would result in decreased displacement and an increased acceleration. This is evident from the graph, where the values in reference building were .385 for Sa and .287 for Sd which changed to .569 for Sa and .215 for Sd for Model B. These values basically demonstrate the **Performance Point.**

Performance point tells us about the performance of a building under Earth quake. These three parameters acceleration, displacement and Time period tells us about Performance point. As, you can see in Model C which is the most stable arrangement depicts no Performance point because the building is too stable and remains within its elastic limit.

Likewise figure 4.5 shows relationship between Spectral acceleration and Time Period. The time period of a building is dependent upon mass and stiffness. It is linked by the following equation $T= \sqrt{M/K}$ where M is mass and K is stiffness. A better arrangement of Shear Walls, result in increased stiffness while the mass remains constant hence resulting in reduced Time period. This can quite clearly be seen from the above graphs where T reduces from 0.956 in reference model to 0.666 in Model A.

5 CONCLUSIONS AND RECOMMENDATIONS:

5.1 Conclusions:

Following conclusions were drawn out from our analysis results:

- From results we found that efficient arrangement was for building C but it isn't economical. Hence, after that arrangement we have building A as most efficient arrangement.
- Base shear and displacement decrease as we go towards efficient arrangement, Curve of base shear VS Displacement also smoothen.
- Hinges transferred to upward floors by providing shear walls, these dropped from LS to IO and number of Hinges reduced overall.
- Performance Point improved as Spectral Acceleration increased, Time Period decreased and Spectral Displacement also decreased.
- Deflections are less in YZ plane when we provide push in X direction.
- Deflections are less in XZ plane when we provide push in Y direction.
- Push over analysis results provides an insight into the performance of structures in post elastic range which thereby helps in assessing the weakness and possible failure mechanisms of structure which is not possible when using equivalent static and response spectrum method of analysis .This could be useful in rectifying the detrimental effects in the design stage itself or for adopting suitable retrofitting methods in case of post-earthquake seismic hazard estimation.

5.2 Recommendations for Better Seismic Performance:

For the improvement of seismic performance of a building following are the recommendations from our analysis results:

- ❑ If only the elevator have to be provided in the building, it should be placed at the center to minimize the difference of center of rigidity and center of mass.
- ❑ If it is not possible to provide elevator in the center than at least one shear wall should be provided on the opposite side of the elevator.
- ❑ Size of members like columns can be reduced economically in case of structure with shear wall as compared to the same structure without shear wall.
- ❑ Base columns should be strong as lower story attracts more seismic forces.
- ❑ Columns near the shear walls attract earthquake forces and moments, so these columns should be strong relative to other columns.
- ❑ Shear Wall arrangements should be kept symmetrical to balance the center of rigidity on both the sides. If arrangement will not be symmetrical there will be more center of rigidity on one side than the other and it will cause torsional effect in the building during earthquake.
- ❑ Even number of Shear Walls should be provided if possible.

5.3 Recommendations for Future Research:

For future research on this topic following are the recommendations:

- In our building elevator isn't located in the center, so for to check better seismic performance same building with elevator in the center can be analyzed and corresponding results can be easily compared.
- This study can be carried forward by performing pushover analysis under push load case in UY direction
- There are 2 types of pushover analysis

  1) Force Controlled

  2) Displacement Controlled.

  We performed Displacement controlled pushover analysis. Force controlled pushover analysis can also be carried out in future research.

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Appendix A

UBC 19 Guidelines

The following tables shows Guidelines of UBC 1997 which is being used for our analysis

EQ Load Combinations (UBC 97)

Combo1= 1.2D + 1.0 EQx
Combo2= 1.2D + 1.0 EQy
Combo3= 1.2D – 1.0 EQx
Combo4= 1.2D – 1.0 EQy
Combo5= 0.9D + 1.0 EQx
Combo6= 0.9D + 1.0 EQy
Combo7= 0.9D – 1.0 EQx
Combo8= 0.9D – 1.0 EQy

TABLE 16-I—SEISMIC ZONE FACTOR *Z*

ZONE	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

NOTE: The zone shall be determined from the seismic zone map in Figure 16-2.

TABLE 16-J—SOIL PROFILE TYPES

SOIL PROFILE TYPE	SOIL PROFILE NAME/GENERIC DESCRIPTION	AVERAGE SOIL PROPERTIES FOR TOP 100 FEET (30 480 mm) OF SOIL PROFILE		
		Shear Wave Velocity, $\bar{v}_s$ feet/second (m/s)	Standard Penetration Test, $\bar{N}$ [or $\bar{N}_{CH}$ for cohesionless soil layers] (blows/foot)	Undrained Shear Strength, $\bar{s}_u$ psf (kPa)
S_A	Hard Rock	> 5,000 (1,500)	—	—
S_B	Rock	2,500 to 5,000 (760 to 1,500)		
S_C	Very Dense Soil and Soft Rock	1,200 to 2,500 (360 to 760)	> 50	> 2,000 (100)
S_D	Stiff Soil Profile	600 to 1,200 (180 to 360)	15 to 50	1,000 to 2,000 (50 to 100)
S_E[1]	Soft Soil Profile	< 600 (180)	< 15	< 1,000 (50)
S_F	Soil Requiring Site-specific Evaluation. See Section 1629.3.1.			

[1]Soil Profile Type S_E also includes any soil profile with more than 10 feet (3048 mm) of soft clay defined as a soil with a plasticity index, $PI > 20$, $w_{mc} \geq 40$ percent and $s_u < 500$ psf (24 kPa). The Plasticity Index, PI, and the moisture content, w_{mc}, shall be determined in accordance with approved national standards.

TABLE 16-K—OCCUPANCY CATEGORY

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTIONS OF STRUCTURE	SEISMIC IMPORTANCE FACTOR, I	SEISMIC IMPORTANCE[1] FACTOR, I_p	WIND IMPORTANCE FACTOR, I_w
1. Essential facilities[2]	Group I, Division 1 Occupancies having surgery and emergency treatment areas Fire and police stations Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category 1 facilities Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures	1.25	1.50	1.15
2. Hazardous facilities	Group H, Divisions 1, 2, 6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2 or 7 Occupancy	1.25	1.50	1.15
3. Special occupancy structures[3]	Group A, Divisions 1, 2 and 2.1 Occupancies Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1 Group I, Division 3 Occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generating stations, and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation	1.00	1.00	1.00
4. Standard occupancy structures[3]	All structures housing occupancies or having functions not listed in Category 1, 2 or 3 and Group U Occupancy towers	1.00	1.00	1.00
5. Miscellaneous structures	Group U Occupancies except for towers	1.00	1.00	1.00

[1]The limitation of I_p for panel connections in Section 1633.2.4 shall be 1.0 for the entire connector.
[2]Structural observation requirements are given in Section 1702.
[3]For anchorage of machinery and equipment required for life-safety systems, the value of I_p shall be taken as 1.5.

TABLE 16-N—STRUCTURAL SYSTEMS[1]

BASIC STRUCTURAL SYSTEM[2]	LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION	R	Ω_0	HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (feet) × 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	5.5	2.8	65
	b. All other light-framed walls	4.5	2.8	65
	2. Shear walls			
	a. Concrete	4.5	2.8	160
	b. Masonry	4.5	2.8	160
	3. Light steel-framed bearing walls with tension-only bracing	2.8	2.2	65
	4. Braced frames where bracing carries gravity load			
	a. Steel	4.4	2.2	160
	b. Concrete[3]	2.8	2.2	—
	c. Heavy timber	2.8	2.2	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	7.0	2.8	240
	2. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	6.5	2.8	65
	b. All other light-framed walls	5.0	2.8	65
	3. Shear walls			
	a. Concrete	5.5	2.8	240
	b. Masonry	5.5	2.8	160
	4. Ordinary braced frames			
	a. Steel	5.6	2.2	160
	b. Concrete[3]	5.6	2.2	—
	c. Heavy timber	5.6	2.2	65
	5. Special concentrically braced frames			
	a. Steel	6.4	2.2	240
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)			
	a. Steel	8.5	2.8	N.L.
	b. Concrete[4]	8.5	2.8	N.L.
	2. Masonry moment-resisting wall frame (MMRWF)	6.5	2.8	160
	3. Concrete intermediate moment-resisting frame (IMRF)[5]	5.5	2.8	—
	4. Ordinary moment-resisting frame (OMRF)			
	a. Steel[6]	4.5	2.8	160
	b. Concrete[7]	3.5	2.8	—
	5. Special truss moment frames of steel (STMF)	6.5	2.8	240
4. Dual systems	1. Shear walls			
	a. Concrete with SMRF	8.5	2.8	N.L.
	b. Concrete with steel OMRF	4.2	2.8	160
	c. Concrete with concrete IMRF[5]	6.5	2.8	160
	d. Masonry with SMRF	5.5	2.8	160
	e. Masonry with steel OMRF	4.2	2.8	160
	f. Masonry with concrete IMRF[3]	4.2	2.8	—
	g. Masonry with masonry MMRWF	6.0	2.8	160
	2. Steel EBF			
	a. With steel SMRF	8.5	2.8	N.L.
	b. With steel OMRF	4.2	2.8	160
	3. Ordinary braced frames			
	a. Steel with steel SMRF	6.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
	c. Concrete with concrete SMRF[3]	6.5	2.8	—
	d. Concrete with concrete IMRF[3]	4.2	2.8	—
	4. Special concentrically braced frames			
	a. Steel with steel SMRF	7.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
5. Cantilevered column building systems	1. Cantilevered column elements	2.2	2.0	35[7]
6. Shear wall-frame interaction systems	1. Concrete[8]	5.5	2.8	160
7. Undefined systems	See Sections 1629.6.7 and 1629.9.2	—	—	—

N.L.—no limit

[1]See Section 1630.4 for combination of structural systems.

[2]Basic structural systems are defined in Section 1629.6.

[3]Prohibited in Seismic Zones 3 and 4.

[4]Includes precast concrete conforming to Section 1921.2.7.

[5]Prohibited in Seismic Zones 3 and 4, except as permitted in Section 1634.2.

[6]Ordinary moment-resisting frames in Seismic Zone 1 meeting the requirements of Section 2211.6 may use a R value of 8.

[7] Total height of the building including cantilevered columns.

[8]Prohibited in Seismic Zones 2A, 2B, 3 and 4. See Section 1633.2.7.

TABLE 16-Q—SEISMIC COEFFICIENT C_a

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	$Z = 0.075$	$Z = 0.15$	$Z = 0.2$	$Z = 0.3$	$Z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

TABLE 16-R—SEISMIC COEFFICIENT C_v

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	$Z = 0.075$	$Z = 0.15$	$Z = 0.2$	$Z = 0.3$	$Z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

TABLE 16-S—NEAR-SOURCE FACTOR N_a[1]

SEISMIC SOURCE TYPE	CLOSEST DISTANCE TO KNOWN SEISMIC SOURCE[2,3]		
	≤ 2 km	5 km	≥ 10 km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

[1]The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.

[2]The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

[3]The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

TABLE 16-T—NEAR-SOURCE FACTOR N_v[1]

SEISMIC SOURCE TYPE	CLOSEST DISTANCE TO KNOWN SEISMIC SOURCE[2,3]			
	≤ 2 km	5 km	10 km	≥ 15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

[1]The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.

[2]The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

[3]The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

TABLE 16-U—SEISMIC SOURCE TYPE[1]

SEISMIC SOURCE TYPE	SEISMIC SOURCE DESCRIPTION	SEISMIC SOURCE DEFINITION[2]	
		Maximum Moment Magnitude, M	Slip Rate, SR (mm/year)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity	$M \geq 7.0$	$SR \geq 5$
B	All faults other than Types A and C	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	$M < 6.5$	$SR \leq 2$

[1]Subduction sources shall be evaluated on a site-specific basis.

[2]Both maximum moment magnitude and slip rate conditions must be satisfied concurrently when determining the seismic source type.

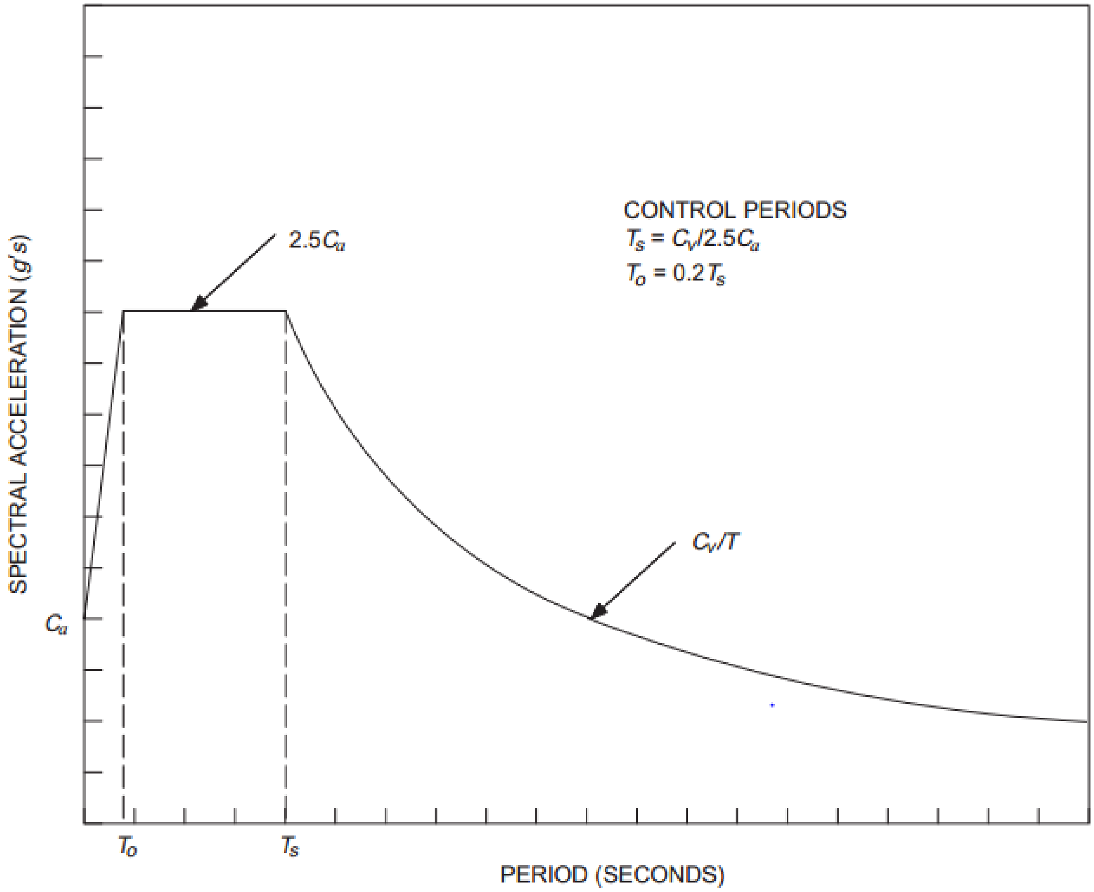

FIGURE 16-3—DESIGN RESPONSE SPECTRA