A COMPREHENSIVE STUDY ON PARAMETERS AFFECTING STIFFNESS OF SHEAR WALL-FRAME BUILDINGS UNDER LATERAL LOADS

By

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ABSTRACT OF THE THESIS

A Comprehensive Study on Parameters Affecting Stiffness of

Shear wall-Frame Buildings under Lateral Loads

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Lateral Stiffness is one of the most important properties of a building which not only defines resistance to displacements under lateral loads but it can also have a great impact on natural period of a structure. Different stiffness values can ultimately affect the behavior of a structure under seismic loads and lateral forces that will be applied to it. In this study several parameters that can affect lateral stiffness of shear wall-frame buildings have been studied.

At first, different configurations of shear walls in plan were analyzed in both medium rise and high rise buildings. In both cases, models with a central concrete core showed higher stiffness and smaller lateral displacements but at the same time, higher story forces were applied to the structure in seismic analysis.

Cracking in shear walls is the other parameter considered in this study. Results indicate that cracked modification factors introduced in ACI can greatly impact the stiffness and other related properties of a building. A more precise approach is used to identify cracked elements based on finite element methods.

Openings in shear walls have also been studied. Analysis results showed that in the case considered, openings with an area up to 10% of the wall did not have a significant influence on stiffness of the structure but higher opening ratios eventually resulted into severe loss of stiffness and large displacements. Stiffness of walls not parallel to direction of load is also investigated. Results indicate that even small angles between direction of load and shear wall can considerably reduce the stiffness of a structure in a specific direction. In the end, two parameters affecting flexural capacity of shear walls have been studied, vertical reinforcement and wall thickness. Even though an increase in each of the two parameters can be helpful, results show that each of them can increase flexural capacity of a section more efficiently under different conditions.

Analysis in all of the models is performed by ETABS 2013. The intention is not to get involved in too many complex calculations, but rather compare the behavior of buildings in different conditions as a practical guide.

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Chapter 1

Thesis Overview

1.1 Introduction and Problem Statement

Shear walls are generally used to resist lateral loads caused by earthquake or wind acting parallel to the plane of the wall in addition to gravity loads from adjacent floors. These walls can often provide lateral bracing for the structure by reducing lateral displacements and resisting applied forces. In shear wall-frame buildings, lateral loads are resisted in part by the wall and in part by frames which the combination of the two provides lateral stiffness for buildings. But what is stiffness and why is it important?

Stiffness in definition is the rigidity of an object. It defines the resistance to deformation caused by applied loads. The higher the resistance to deformation is, the stiffer the object is assumed to be. This concept of stiffness is defined in many principal laws of physics such as Hook's law where it states that forces imposed on a solid, are directly proportional to displacements produced within elastic limits. The relationship between forces and displacements is established when a constant factor characteristic of the object is introduced. This force constant is also called "Stiffness". Stiffness is a function of material properties and geometry. In structural engineering, stiffness is proportional to material's Young modulus of elasticity and the section's moment of inertia which is the second moment of area. It is measured in force per unit length and is the equivalent of force constant in Hook's law.

The importance of lateral stiffness is that it plays a decisive role in analysis and design of the structure by affecting some of the most major design factors and considerations. Stiffness values and matrices are typically the key information which by knowing them, many problems and equations can be solved in analysis of structures, especially if finite element methods are being used. Besides that, some of the principal factors are also a function of lateral stiffness such as lateral displacements, natural period and seismic forces. These parameters must be described in order to define the problem.

Lateral Displacements

Lateral displacements can be one of the most decisive factors in design of a building. In cases where maximum displacements must be limited because of adjacent buildings or serviceability issues, the biggest challenge can be how to reduce displacements to allowable amounts. As mentioned previously, Hook's law relates applied forces to displacements using the concept of stiffness. In buildings, this law also applies using the equation below

$$\{F\} = [K]\{d\}$$

Where F is the applied forces, d is the produced displacement and K is the stiffness which established the relation between the two. Equation above clearly shows the inverse relationship between stiffness of a structure and produced lateral displacements. As a result, displacements are highly influenced by stiffness of a structure and therefore, it is very important to acknowledge how changes in stiffness can impact the behavior of structure in terms of maximum lateral displacements, which is part of the study in this research.

Natural Period

The fundamental building period is the inverse of the building frequency at the lowest harmonic and represents the time it takes for the structure to oscillate back and forth once. Period is independent of loads that are applied to the structure and is only a function of mass and stiffness as observed in equation below

$$T = 2\pi\sqrt{\frac{m}{k}}$$

It is seen that stiffness can influence natural period of a structure considerably. The stiffer a system is, the shorter its period will be. Modal periods in buildings are very important and can greatly impact analysis of a structure. Part of the study in this research is investigating how stiffness affects natural period and consequently, analysis results.

Seismic forces

The fact that variations in stiffness can change fundamental period of the structure can also affect seismic forces that the building will experience. If a typical design spectral acceleration diagram is considered (such as figure below from ASCE), changes in natural period of the structure can impact design accelerations which may lead to different seismic forces. Therefore, it is seen that a change in stiffness of a structure can also impact applied forces in seismic analysis.

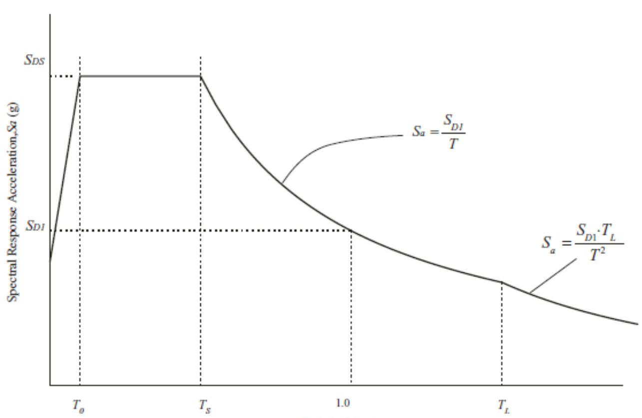


Figure 1.1: Design response spectrum
Source: ASCE 7-10

1.2 Research Objectives and Scope

The key role that stiffness of the structure has in analysis and design of shear wall-frame structures is the focus of this study. The concentration is mainly on shear walls and how various parameters can affect the stiffness of these walls and other design factors explained. The parameters considered in this study are explained below.

Shear Wall Configuration in Plan

Different shear wall distributions in plan can result into different stiffness values for the same structure. Effects of shear wall configurations are studied for both medium rise buildings and high rise towers in chapter 3.

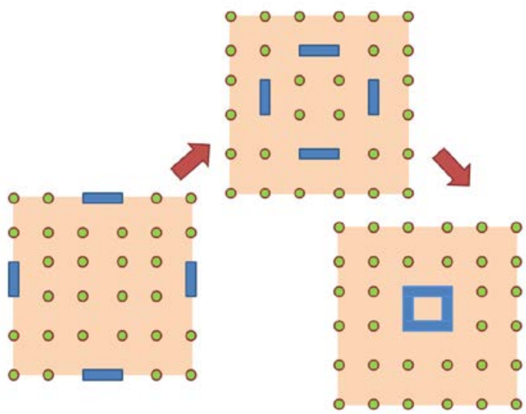


Figure 1.2: Various shear wall configurations

Effect of Cracking on Stiffness of Shear Walls

The intention is to investigate how cracking can reduce the stiffness of shear walls and how critical the influence can be in terms of lateral displacements of the structure. Chapter 4 covers this topic.

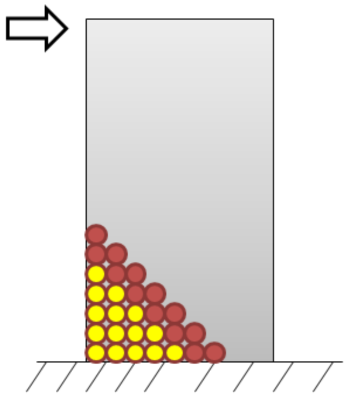


Figure 1.3: Cracking in shear walls

Effects of Openings on Stiffness of Shear Walls

When there's a need to provide openings in shear walls, how will it affect the stiffness of the wall and structure as a result? How important is the size of openings compared to the wall and should all of them be considered in the analysis? These are the questions that are investigated in chapter 5 of this study.

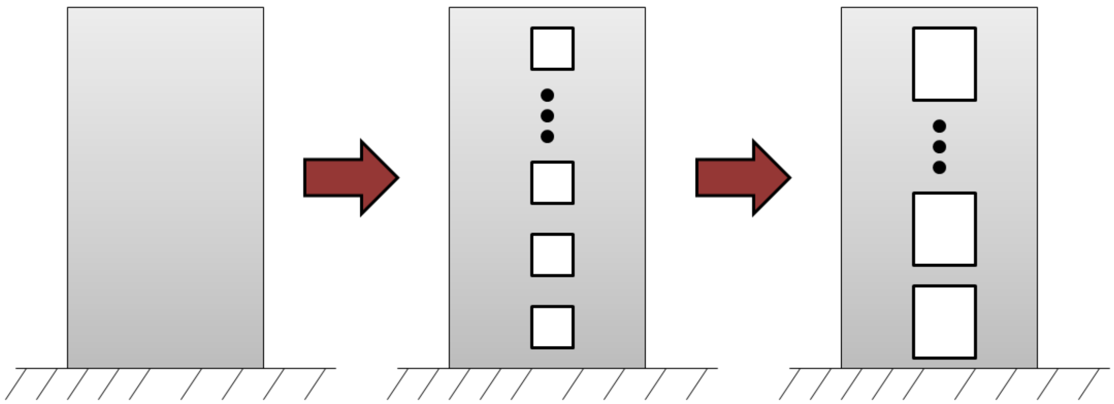


Figure 1.4: Openings in shear walls

Skewed Walls and Stiffness of Structures

The attempt is to see how behavior of the wall changes when it is not parallel to direction of loading, and how different angles can result into different stiffness values for the structure.

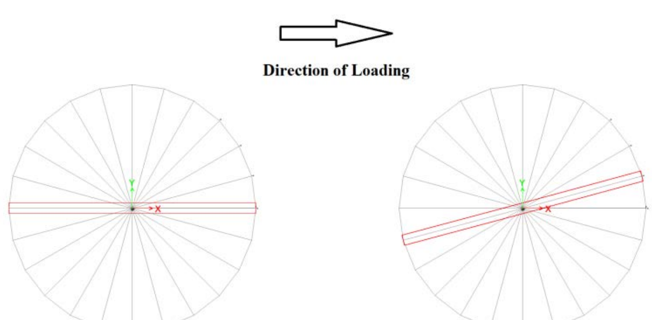


Figure 1.5: Skewed walls

Flexural Capacity of Wall Sections

After considering some of the factors that can affect stiffness of a shear wall-frame structure, several parameters that influence flexural capacity of shear walls have also been investigated such as vertical reinforcement and thickness of the wall.

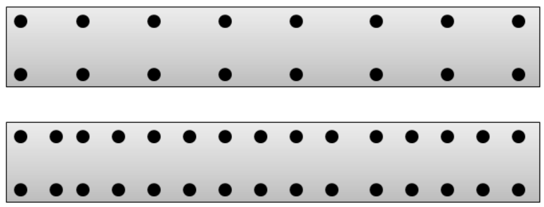


Figure 1.6: Various vertical reinforcement ratios

Since multiple parameters are being investigated, it is necessary to use powerful software products for analysis and design of numerous models, each with different properties. Some chapters require a full 3D analysis of high rise towers meaning that besides modern software and tools, strong computers are also required. In this study, the most recent version of CSI Berkley integrated building design software, ETABS 2013 ultimate version 13.1.1 is used which is one of the most popular tools among structural engineers. Values obtained from analysis performed by ETABS are considered to be final results and are presented in tables and graphs using Microsoft EXCEL 2007.

The intention of this research is not to get deeply involved in complex calculations and theories regarding a certain parameter. Instead, the effort is to cover multiple important factors that can affect stiffness of shear walls in a comprehensive study. Parameters that structural engineers deal with in regular design procedures and might be interested in knowing how they can each influence the overall behavior of a structure. In other words, the intention is to produce a practical guide on how several factors can change the stiffness and related properties in a shear wall-frame building subjected to lateral loads.

Chapter 2

General Model Information

2.1 Introduction

Throughout this study, several parameters are investigated that can have a significant influence on stiffness of shear walls and the structure. Since each of these parameters is different in many ways than others, a single model may not be adequate to study all of the desired factors. Therefore, each chapter will have a separate section which describes the model used for that particular study and the methodology behind it. However, some of the properties and conditions must remain the same to limit the influence of other parameters and prevent them from having an effect on the results. If the goal is to investigate the impact of a certain factor, other factors must remain the same in order to conclude that alterations in results are only due to changes of parameter under study and no other factor is involved. To simplify model descriptions in different chapters and avoid repetition, certain information which remains unchanged is summarized in this chapter. This general model information includes material properties, gravity loads and seismic parameters used in several chapters. Properties and loads described in this chapter remain unchanged throughout the research unless noted and different sections will refer to data described here. Codes that are used in obtaining some of the information are ACI 318-08 and ASCE 7-10 and seismic information is obtained from USGS design maps based on ASCE. As mentioned earlier, all of the analyses in this research are performed by ETABS 2013 and the information provided in this chapter will be defined as input values for the software.

2.2 Material Properties

Since this research is mainly about shear walls and parameters affecting a shear wall-frame building, concrete structures will be studied. As a result, the only materials used in all of the models are concrete and reinforcement steel. Table below summarizes all the material properties that are used.

Table 2.1: Material Properties

Material	**F'_c - Compressive strength / F_y - Yield Strength (psi)**	**E Modulus of Elasticity (Ksi)**	**Weight Per Unit Volume (lb/ft^3)**
Concrete (slabs and columns)	4,000 psi	3,600 ksi	150 lb/ft^3
Concrete (shear walls)	5,000 psi	4,000 ksi	150 lb/ft^3
Reinforcement Rebar	60,000 psi	29,000 ksi	490 lb/ft^3

These material properties will not change in any of the chapters and cover all the materials that are used in different models. For instance, if one chapter describes a model for a shear wall, all of the properties are those described in "Concrete (shear wall)" row in table above. It is a common practice to use a concrete with higher strength and elasticity in shear walls since it can help the structure gain higher stiffness and better resist the applied loads.

2.3 Gravity Loads

Gravity loads are assumed to be constant and equal in all levels. Three cases are considered for gravity loads

1. Dead Load (Self Weight)
2. Super Dead Load
3. Live Load

Dead load is the total self weight of all members which is automatically calculated using ETABS and is not an input. Other two loads are defined for the program as presented in table below.

Table 2.2: Gravity Loads

Load Case	Value
Super Dead Load	60 psf
Live Load	100 psf

2.4 Seismic Loads

For chapters where seismic load is present, information is obtained from U.S seismic design maps and the location is considered to be "Piscataway, NJ" where Rutgers engineering campus is located. Data is provided for this location at "earthquake.usgs.gov" website based on ASCE figures 22-1 through 22-6. The information required in order to obtain data are the zip code, site class and risk category. Based on ASCE provision 11.4.2, the default site class is assumed to be "Class D" and risk category for all of the models is considered as "Category II".

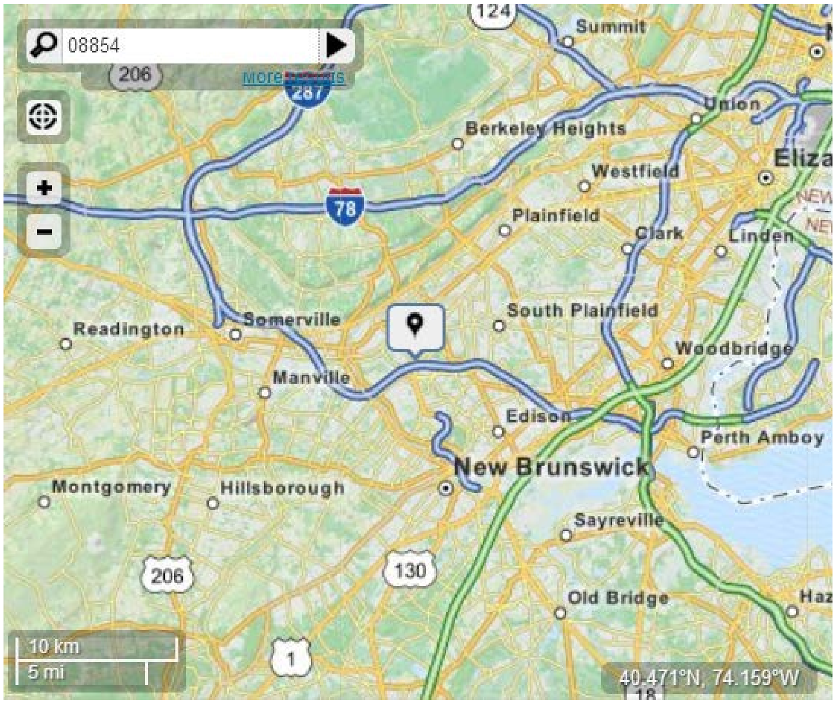


Figure 2.1: Location of seismic information - Piscataway, NJ

Based on information from USGS and ASCE figures 22-1 through 22-6, information below is provided.

$$S_s = 0.255\ g$$

$$S_1 = 0.069\ g$$

Where S_s is the mapped MCE_R spectral response acceleration parameter at short periods and S_1 is the mapped MCE_R spectral response acceleration parameter at a period of 1 second. "g" is acceleration due to gravity and is used as unit of acceleration for S_s and S_1.

ASCE provision 11.4.3 provides the following equations in order to obtain The MCE_R spectral response acceleration parameter for short periods (S_{MS}) and at 1 second (S_{M1}), adjusted for Site Class effects (equations 11.4-1 and 11.4-2)

$$S_{MS} = F_a S_s$$

$$S_{M1} = F_v S_1$$

Site coefficients F_a and F_v are defined in ASCE Tables11.4-1 and 11.4-2, respectively. These coefficients adjust the accelerations for site class based on spectral response acceleration parameters and interpolation between values provided in tables.

Design earthquake spectral response acceleration parameter at short period, S_{DS}, and at 1 second period, S_{D1}, shall be determined from equations 11.4-3 and 11.4-4 respectively. These equations are provided below.

$$S_{DS} = \frac{2}{3} S_{MS}$$

$$S_{D1} = \frac{2}{3} S_{M1}$$

USGS provides a profound and detailed report which completely illustrates how the values for F_a and F_v are obtained based on ASCE. Figure below is part of the report which highlights the corresponding values and provides the final result based on linear interpolation.

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.255 g, F_a = 1.596

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.069 g, F_v = 2.400

Figure 2.2: Calculation of site coefficients
Source: www.usgs.gov

Since the values of F_a and F_v are obtained, results for S_{MS}, S_{M1}, S_{DS} and S_{D1} can also be calculated using the equations provided before.

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} F_a S_s = \frac{2}{3} \times 1.596 \times 0.255\ g = 0.272\ g$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} F_v S_1 = \frac{2}{3} \times 2.400 \times 0.069\ g = 0.110\ g$$

Besides these two values, several other parameters are required in order to be able to construct “Design Response Spectrum”. These factors include T_0 and T_s which are both addressed in ASCE section 11.4.5 as bellow.

$$T_0 = 0.2 \frac{S_{D1}}{S_{Ds}} = 0.2 \times \frac{0.110\ g}{0.272\ g} = 0.081$$

$$T_s = \frac{S_{D1}}{S_{Ds}} = \frac{0.110\ g}{0.272\ g} = 0.404$$

Other parameter required is T_L which is long-period transition period obtained from ASCE figures 22-12 through 22-16. In this case, T_L is equal to 6 seconds.

All of the information required to construct the design response spectrum are now available. Figure 11.4-1 in ASCE code provides instructions on how to obtain the spectrum. This figure is very important in seismic analysis and design since by using it, based on what the fundamental period of the structure is, spectral response acceleration can be calculated. This acceleration is directly related to forces applied to the structure. Since the acceleration is influenced by fundamental period of a structure, and period is a function of stiffness, it can be concluded that stiffness can play an important role in seismic loads and analysis of buildings. This discussion will be further investigated in following chapters.

Design response spectrum is displayed in figure below.

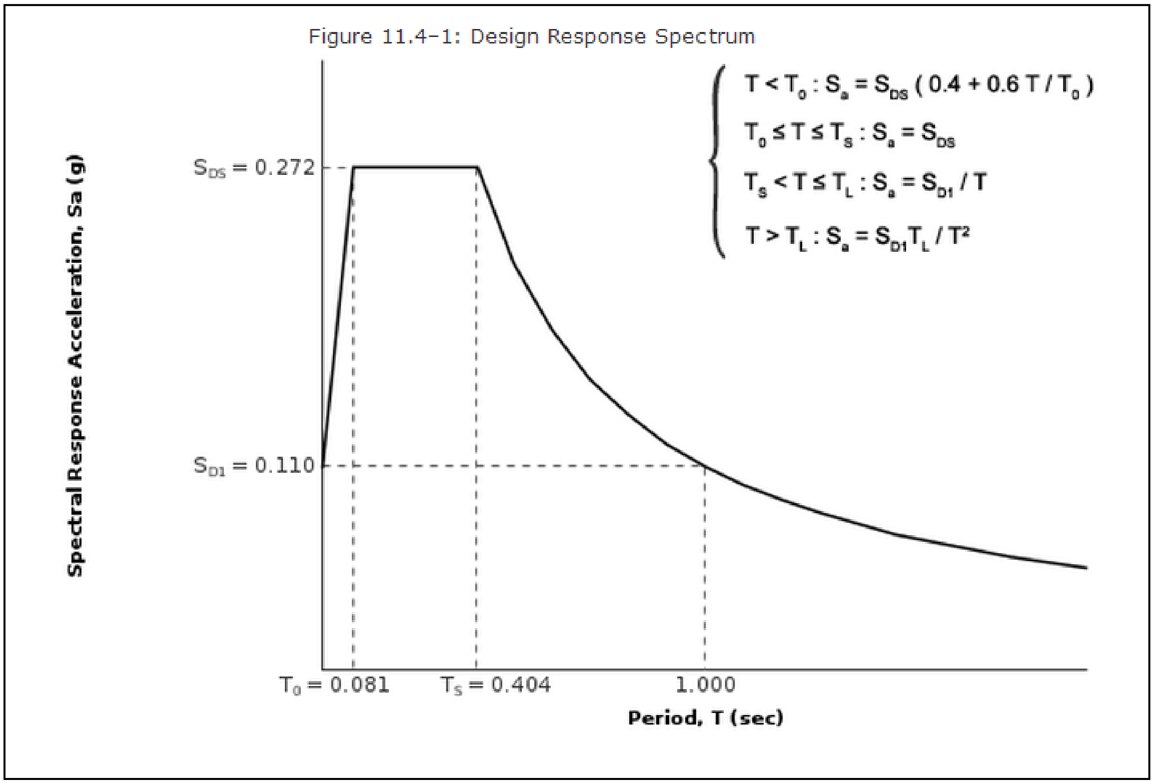


Figure 2.3: Design response spectrum
Source: www.usgs.gov

The other important piece of information that is necessary in seismic analysis of buildings is “Seismic Design Category”. Structures shall be assigned a seismic design category based on section 11.6 of ASCE. In this section ASCE explains that “structures shall be assigned to a Seismic Design Category based on their Risk Category and the design spectral response acceleration parameters, S_{DS} and S_{D1}, determined in accordance with Section 11.4.4. Each building and structure shall be assigned to the more severe Seismic Design Category in accordance with Table 11.6-1 or 11.6-2, irrespective of the fundamental period of vibration of the structure, *T*.”

Simplified calculations are displayed in figures obtained from USGS. Highlighted cells show the result in each table.

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.272 g, Seismic Design Category = B

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.110 g, Seismic Design Category = B

Figure 2.4: Seismic design category - Source: www.usgs.gov

In this case, both tables 11.6-1 and 11.6-2 indicate that the seismic design category is category B. As it is seen, seismic design category is a function of seismic ground motions, site classification and building importance factor. Therefore a category B can represent a structure where either large ground motions might occur, or soil is soft in that location or building has a high importance. ASCE provides specific design requirements for concrete buildings based on their seismic design category such as limitations on seismic force resisting systems and structural height. By knowing that the structure is in seismic design category B, “Equivalent Lateral Force Analysis” can be performed on all of the models and structures in this study based on ASCE table 12.6-1. Therefore, ELF is the method of analysis used for seismic forces in ETABS.

Information that is obtained so far is enough to define earthquake forces in ETABS and prepare the analysis. Seismic loads must be applied to the structure in both X and Y directions, but when a building is subjected to large lateral displacements, stiffness of lateral resisting systems may change in a non uniform fashion. This will result into relocation of center of rigidity and center of mass which can increase torsional forces accidentally. To account for this issue, ASCE section 12.8.4.2 states that "Where diaphragms are not flexible, the design shall include the inherent torsional moment (M_t) resulting from the location of the structure masses plus the accidental torsional moments (M_{ta}) caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces." This section implies that besides the two main directions, there must be other seismic cases which include the effect of accidental torsions. As a result, there are 6 different load cases considered for earthquake forces, which is described in the table below. Several chapters will refer to these load cases by labels presented in this table.

Table 2.3: Earthquake load cases and labels

Label	**Description**
Earthquake 1	X Direction
Earthquake 2	Y Direction
Earthquake 3	X Direction + Eccentricity
Earthquake 4	Y Direction + Eccentricity
Earthquake 5	X Direction - Eccentricity
Earthquake 6	Y Direction - Eccentricity

In the end there are several other considerations in analysis and design for seismic loads which can impact the results and must be noted.

- In any of the models that diaphragms are used, they are considered to be rigid diaphragm and will distribute lateral loads based on stiffness of lateral resisting elements.
- Seismic loads are applied from base of the structure to the top floor in all of the models which earthquake loads are present
- All of the analyses are performed in full 3D with six active degrees of freedom for the building
- All of the load combinations, load and resistance factors and design procedures are based on ACI 318-11
- All models described throughout this study have been analyzed and designed for a preliminary stage to assure that they are stable and do not fail under applied loads.

Chapter 3

Shear Wall Configuration in Plan

3.1 Introduction

Distribution of walls in a building plan can have a very important role in determining lateral stiffness and other related factors. Most of the time, structural engineers may not have many options to choose from because of limitations caused by architecture of a building or other utilities. For instance, large windows or openings are some of the main reasons that prevent engineers from placing shear walls at a certain location. A common practice among engineers is to minimize the distance from center of mass to center of rigidity which is provided by shear walls and frames. The reason is that lateral loads are applied to the center of mass and if there is a substantial distance between where the load is applied and where center of rigidity is, significant torsional moments will be generated. But assuming there are several positions that shear walls can be placed and in all of the possibilities, the distance between CM to CR is minimized and equal to each other, then which factors should be considered in choosing the configuration of shear walls in plan? How can distribution of the same number of shear walls, impact the lateral stiffness of a structure? Is there a single best solution for different cases and considerations?

To answer these questions, two major types of buildings are considered in this chapter, medium rise buildings and high-rise towers. The influence of shear wall distribution in plan is studied in both cases and analysis results based on ETABS are compared in parameters such as lateral stiffness, lateral displacements, story drifts, pier forces and natural periods.

3.2 Medium Rise Building

3.2.1 Model Description and Methodology

For this case study, a 10 story building is considered that is symmetric in plan in both X & Y directions. This square shaped plan is 100ft on each side which is divided by columns to five equal 20ft spans. Columns are evenly spaced at every 20ft in both directions. Each story height is the same and equal to 12ft. Figure below shows a typical plan view without shear walls and 3D rendered view of the entire structure.

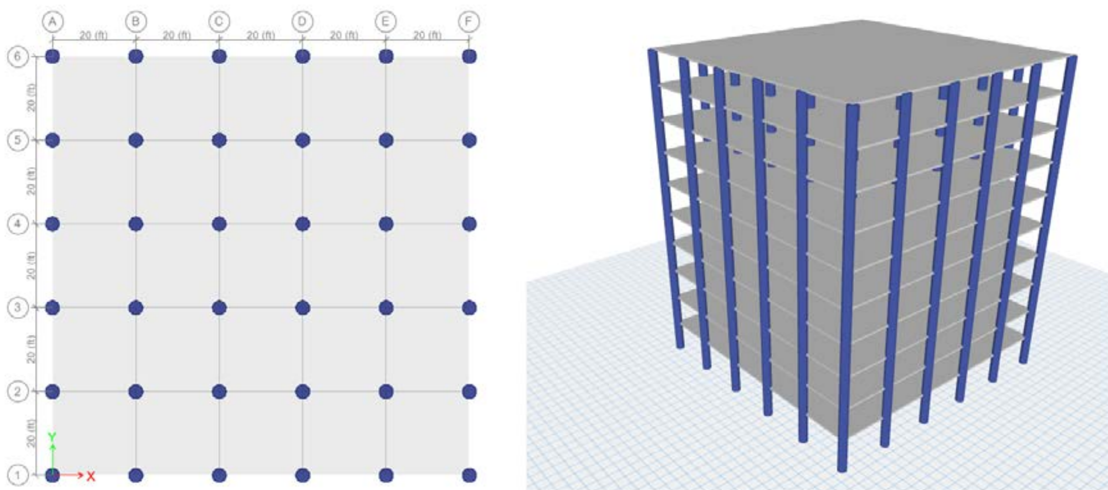


Figure 3.1: Plan and 3D view of medium rise building without shear walls

The floor at each level is a flat slab (no drop panels) that acts as a rigid, two-way slab and transfers the lateral and gravity loads to columns and shear walls. Lateral loads are resisted through ordinary shear walls that are fixed at the base. In this model, all of the columns and shear walls are identical and have the same dimension, regardless of where they are located at. There is no reduction in size of columns and shear walls in upper levels. Slab is also having the same thickness in all floors. In other words, all of the properties and dimensions are the same at each floor and remain constant throughout this chapter. For information regarding material properties and loading, refer to general model information in chapter 2.

After main elements are created, shear walls are distributed in three different configurations and therefore, three separate models are created. All of the design parameters are the same for these models and the only difference between the three is configuration of shear walls. Three main models that are studied in this chapter are shown below. In the first model, walls are located at the perimeter of the plan, along with external frames. For simplicity, this model will be called “Perimeter Walls”. In the second model, walls are pushed inside between the center and perimeter of the plan. This model will be referred to as “Intermediate Walls”. Finally in the third model, walls are located at the center of the plan forming a concrete core. As a result, this model will be referred to as the “Central Core”.

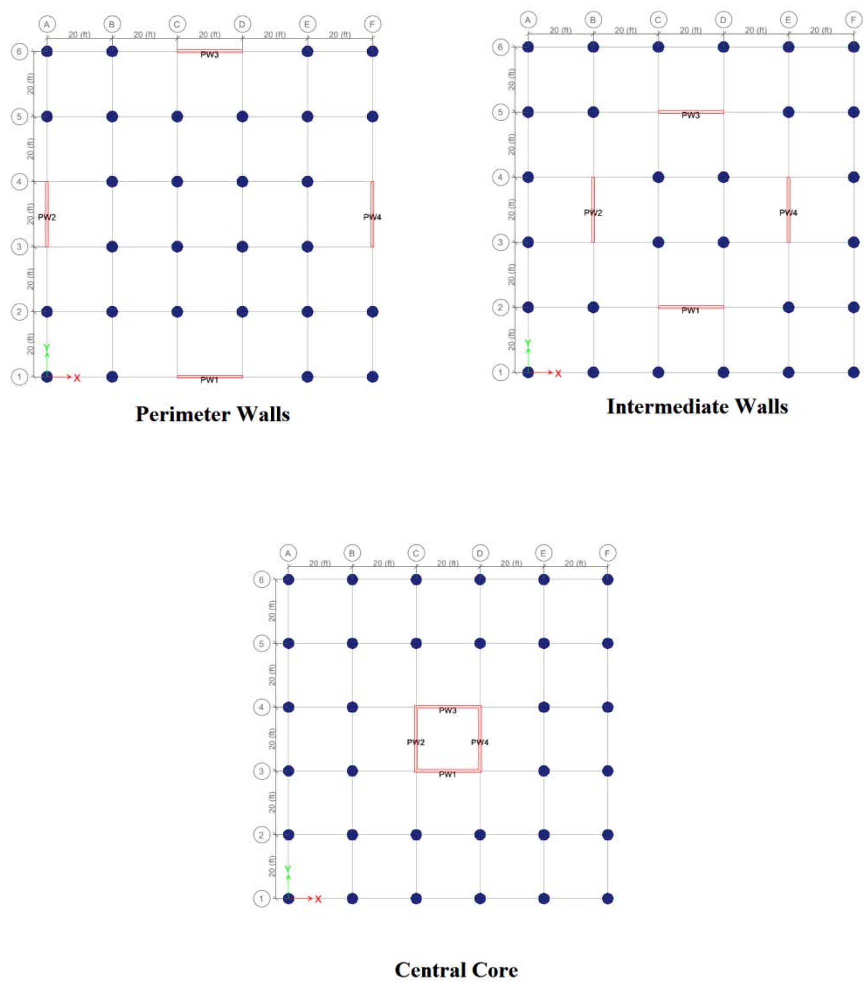


Figure 3.2: Shear wall configurations considered for medium rise building

Size and dimension of sections used medium rise models are described in table below. These sections remain the same throughout the entire structure and in all of the models created in this chapter.

Table 3.1: Section properties - medium rise

Element	**Dimensions**
Flat Slab (4 ksi)	Thickness: 9 in.
Circular Columns (4 Ksi)	Diameter: 40 in.
Shear Walls (5 Ksi)	Thickness: 10 in. Width: 20 ft

Using ASCE Table 12.2-1, design coefficients and factors for seismic force resisting systems are obtained. The system used for resisting lateral loads in this model is "ordinary reinforced shear walls" under building frame systems. The information obtained from chapter 2 indicate that the seismic design category for this project is category B. Table 12.2-1 in ASCE shows that there are no limitations for structures with ordinary reinforced shear walls in design category B. ASCE 11.4.2 states that "Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used". Risk category for residential buildings in ASCE table 1.5-1 is category II, which in table 1.5-2 the importance factor for risk category II under seismic loads is I_e = 1.00. Design factors that are required in order to model the structure are summarized in the table below.

Table 3.2: Seismic design coefficients and factors - medium rise

Design Coefficients & Factors Obtained from ASCE	**Value**
Response Modification Coefficient (R)	5
Overstrength Factor (Ω_0)	2.5
Deflection Amplification Factor (C_d)	4.5
Site Class	D
Seismic Importance Factor (I_e)	1.00

There are major considerations in analysis and design of models described in this section that must be acknowledged.

- All of the models in this chapter are symmetric in plan along X & Y axes. As a result, center of mass and center of rigidity in all three models will be at the center of the plan and therefore, the only torsional forces that will be applied to these structures is the torsion caused by accidental eccentricity.
- Each shear wall shown in the plan is 20ft wide (width of one span). When shear wall is placed in a span, columns are removed from the two ends of that span. Thickness of shear wall is uniform in plan and boundary elements are not considered.
- Number of shear walls (thickness and width of shear walls in plan) remains the same in each direction and in all of the models. In other words, none of the buildings have more shear walls resisting lateral forces in one direction than other models and all of them have the same number of shear walls resisting seismic forces. But these walls are placed in different locations and that is the only difference between models that are considered.
- Slabs are assumed to act as rigid diaphragms. Both slabs and shear walls are created using shell elements in ETABS.
- Shear walls are fixed at the base but columns are modeled as pinned restraints. For shear walls, restraints are at the two ends of the wall.
- Column, wall and slab dimensions are constant throughout the entire structure and in all of the models.
- All of the models were analyzed and preliminary design was performed in order to verify that failure does not occur in any of the elements under described loads. Therefore, all of the models described in this chapter are stable and do not fail under different load combinations described in ASCE and ACI codes.

These important notes indicate that the attempt is to keep all of the factors consistent and unchanged in all of the models. This way, it can be assumed that any change in the result of analysis is directly related to the only difference that these models have which is the layout of shear walls in plan and no other parameter is responsible for alterations in results. Tables and graphs are used extensively in order to better illustrate how each parameter affects the results.

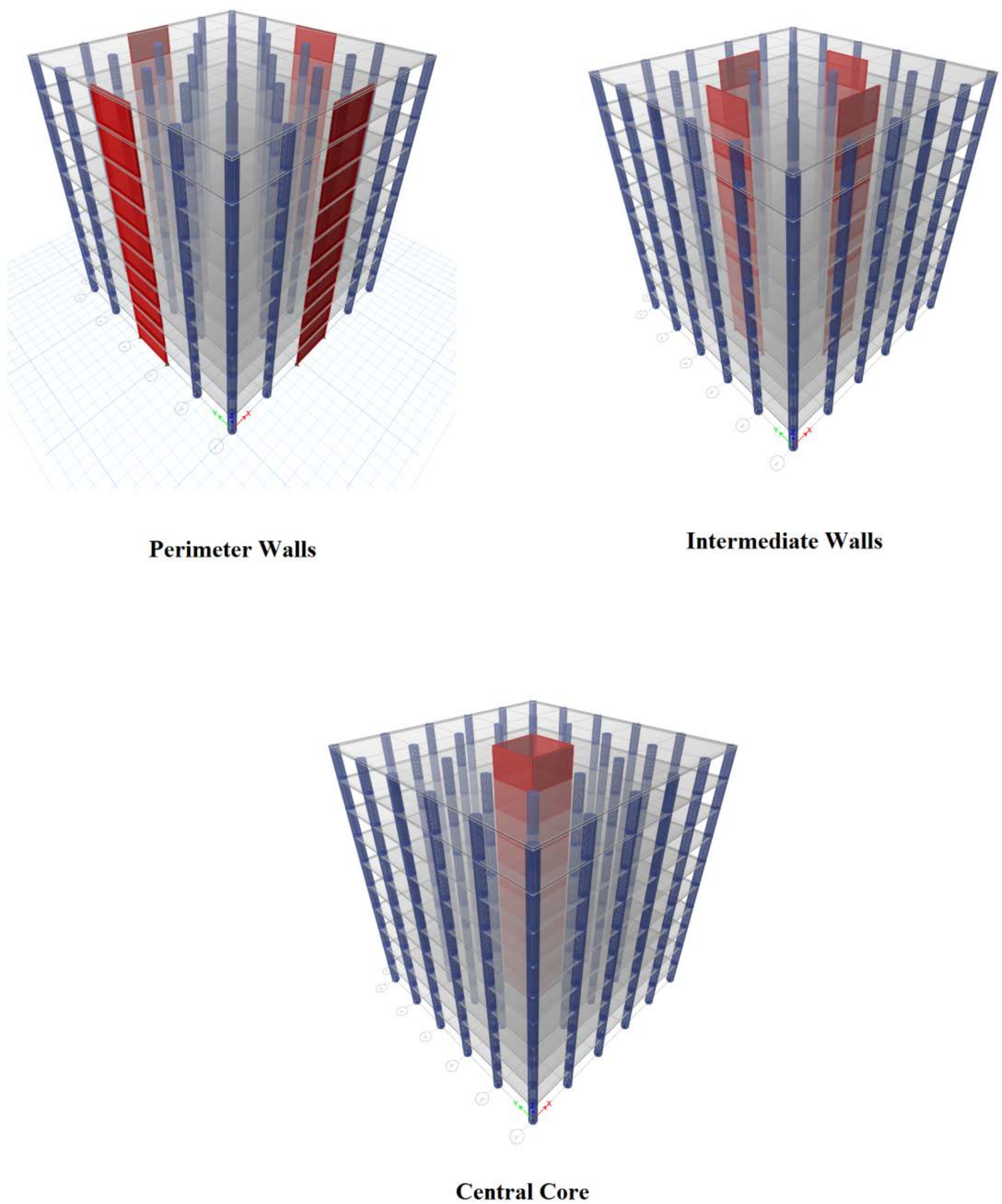


Figure 3.3: 3D view of all medium rise models

3.2.2 Results

a) Lateral Stiffness

Lateral stiffness of a structure, as mentioned earlier, has a great impact on several other key parameters and most of the categories that are being discussed in this chapter are a function of lateral stiffness. Therefore, if stiffness is changed, lateral displacement, forces and natural period of the building will change as a result. The goal in this section is to compare stiffness results of different shear wall configurations and analyze the data. Table below provides stiffness values at each story for all three models.

Table 3.3: Stiffness values - medium rise

Stories	Stiffness (Kips/in.)				
	Perimeter Walls	Intermediate Walls	%	Central Core	%
Story1	11,807.87	12,792.42	+8.3 %	23,996.52	+103.2 %
Story2	5,658.11	6,224.53	+10 %	13,102.38	+131.6 %
Story3	3,873.89	4,310.72	+11.3 %	9,109.69	+135.2 %
Story4	3,067.64	3,450.19	+12.5 %	7,141.82	+132.8 %
Story5	2,575.85	2,927.99	+13.7 %	5,885.34	+128.5 %
Story6	2,207.00	2,536.41	+14.9 %	4,926.65	+123.2 %
Story7	1,872.17	2,176.60	+16.3 %	4,073.90	+117.6 %
Story8	1,514.32	1,781.83	+17.7 %	3,210.72	+112 %
Story9	1,085.94	1,293.20	+19.1 %	2,247.91	+107 %
Story10	551.77	663.02	+20.2 %	1,118.28	+102.7 %

"%" column displays the increase in each model compared to "perimeter walls"

$$\% = \left(\frac{stiffness\ of\ intermediate\ or\ central\ core}{stiffness\ of\ perimeter\ walls} - 1\right) \times 100$$

It is observed that the model with intermediate walls is having a relatively higher lateral stiffness than the model with perimeter shear walls but the model with a central core is showing a significant increase in lateral stiffness compared to other two models. It can be observed that the central core at most levels is having a stiffness that is almost double the stiffness of perimeter walls and intermediate walls. Graph below can visually describe the difference between results obtained from analysis.

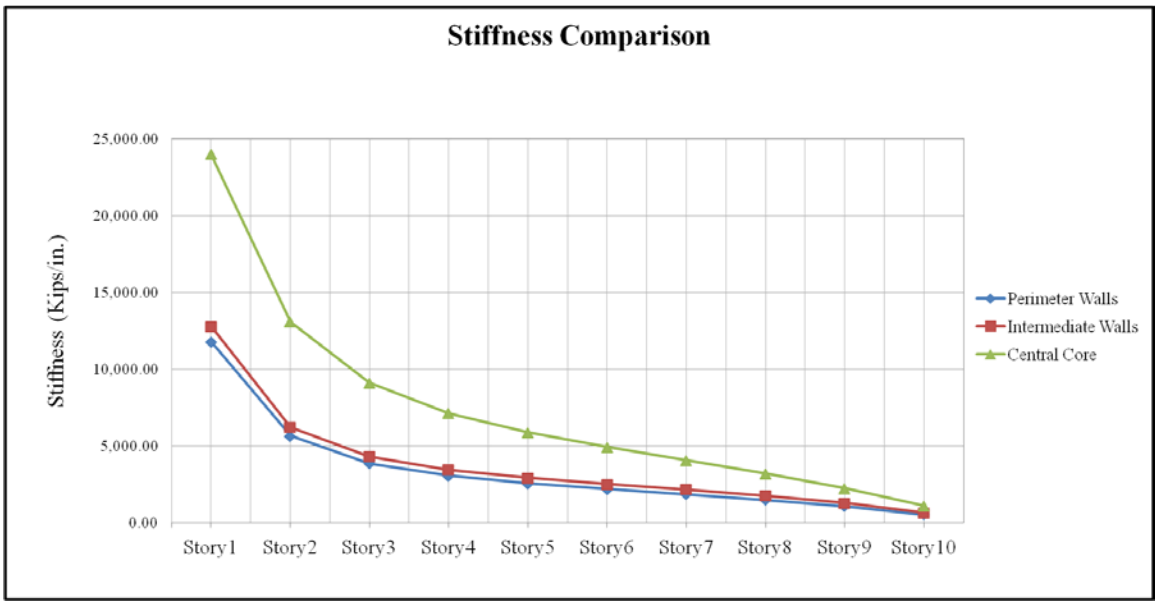


Figure 3.4: Stiffness comparison - medium rise

In the graph, perimeter walls and intermediate walls are moving very close to each other, but intermediate walls tend to show slightly higher stiffness values at all stories. Central core however, is showing much more stiffness compared to other two models and the line representing its stiffness is placed well above the other two. Note that the stiffness of central core and other two models are much higher at lower levels, but gradually decrease in top floors. This is due to the influence of height on stiffness of shear walls and frames at different levels which will be further investigated.

b) Lateral Displacements and Story Drifts

Based on Hook's law which was described earlier, it is concluded that lateral displacement of a structure is in inverse relationship with stiffness. In other words, the higher the stiffness, the smaller the lateral displacement will be and vice versa. This means that according to the results described in stiffness comparison, it is expected to see the smallest story displacements in model with a central core since highest stiffness values were obtained for this model. Table below provides data related to maximum story displacements in all levels and shows that analysis results are in agreement with expectations.

Table 3.4: Maximum story displacements - medium rise

Stories	Maximum Story Displacements (in.)				
	Perimeter Walls	Intermediate Walls	%	Central Core	%
Story1	0.03	0.03	-0.6 %	0.02	-16.5 %
Story2	0.08	0.08	-2.5 %	0.06	-31.1 %
Story3	0.16	0.16	-3.6 %	0.10	-37.1 %
Story4	0.26	0.25	-4.5 %	0.16	-40 %
Story5	0.37	0.35	-5.3 %	0.21	-41.5 %
Story6	0.48	0.45	-6 %	0.28	-42.4 %
Story7	0.59	0.55	-6.7 %	0.34	-42.9 %
Story8	0.71	0.65	-7.3 %	0.40	-43.2 %
Story9	0.82	0.75	-8 %	0.46	-43.4 %
Story10	0.92	0.84	-8.5 %	0.52	-43.5 %

The "%" column shows the reduction in lateral displacement compared to the model with perimeter walls. As expected, central core (which had shown significant stiffness) is having a maximum lateral displacement of almost half the displacement that perimeter walls are showing.

Intermediate walls also result into less displacements compared to perimeter walls by about 8.5% which is not very significant compared to the difference that central core is having. Graphs below compare both the lateral displacements and the story drifts. In both figures, the model with central core is showing a better performance in terms of lateral displacements.

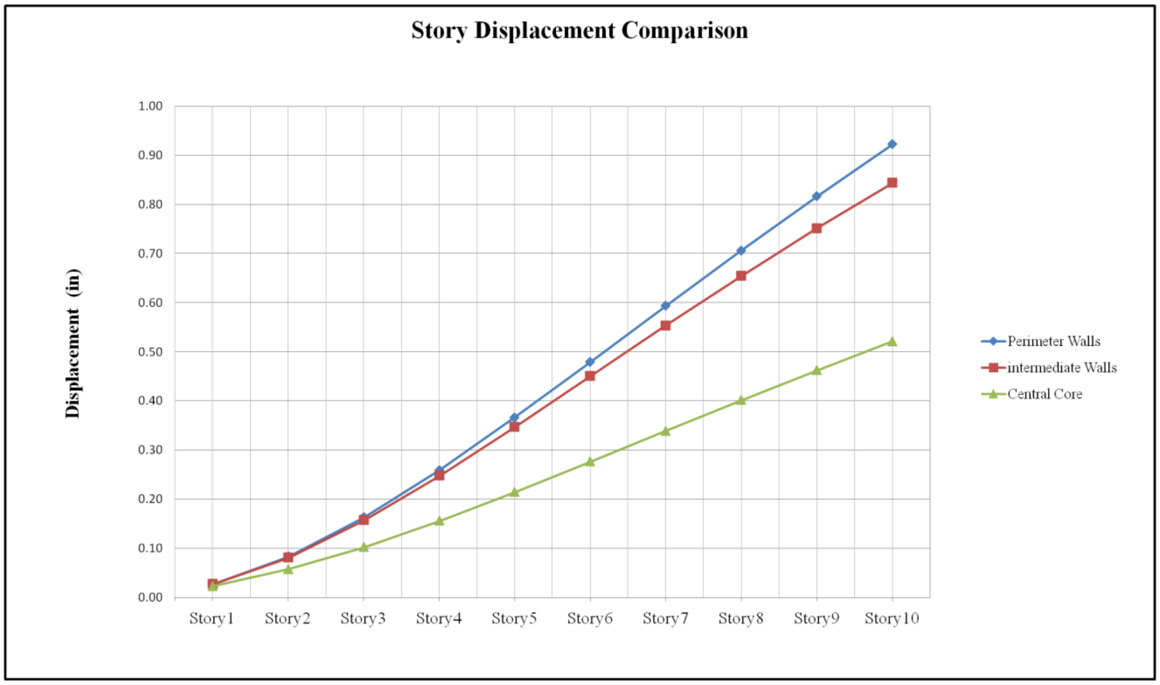


Figure 3.5: Displacement comparison - medium rise

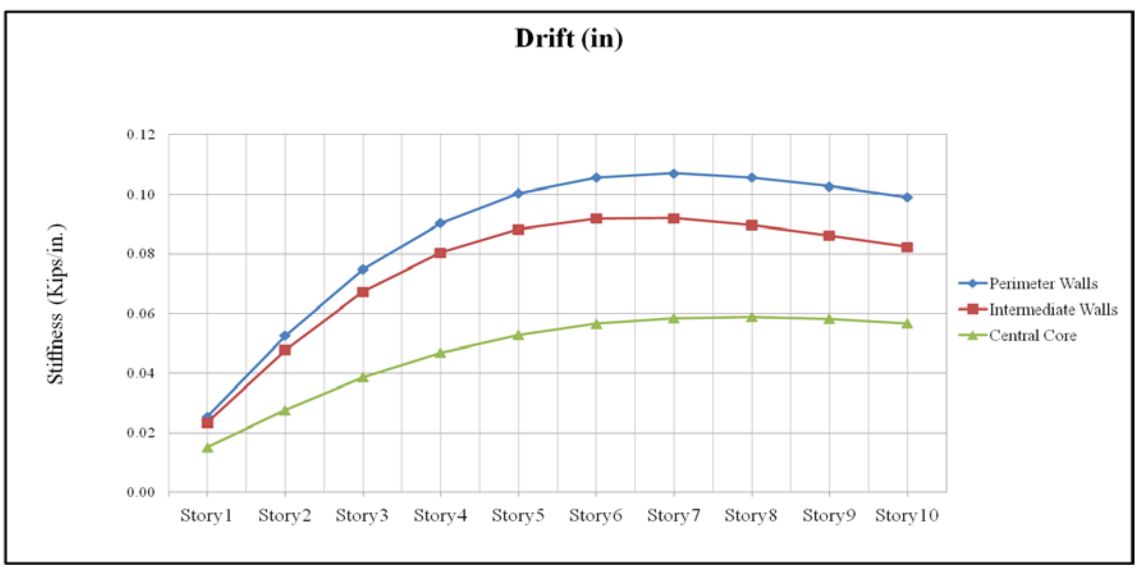


Figure 3.6: Drift comparison - medium rise

All of the story displacements presented are based on maximum displacement that occurs at each floor in all of the earthquake load cases. As explained in general model information, there are six different earthquake load cases that have been defined for ETABS. In this case, maximum story displacement and drift, happens at earthquake loadings plus or minus accidental eccentricities. Since the model is symmetric about X & Y axes, this maximum displacement occurs at earthquake load cases 3, 4, 5 and 6 (3 and 5 in X direction, 4 and 6 in Y direction) and shows the same results in all these cases which is the expected behavior.

Maximum story drift is limited in ASCE7-10. Table 12.12-1 in ASCE limits story drift to $0.020h_{sx}$ (Risk category I or II) where h_{sx} is the story height below level x. Knowing each story height is 12ft we have

$$0.020 \times \mathrm{h_{sx}} = 0.020 \times 12 = 0.24' = 2.88'' \ Allowable\ Story\ Drift$$

As it is seen in the graph, drift values for all cases are well below the allowable amount provided in ASCE but central core is showing much less story drift than the other two models.

Macgregor in his articles suggests limiting the horizontal deflection of a story (under service loads) to about 1/500 of story height. This suggestion will result to a story drift of

$$\frac{1}{500} \times \mathrm{h_{sx}} = 0.002 \times 12 = 0.024' = 0.288''$$

All of the models still satisfy the suggested lateral drift.

Story drifts can be very dangerous for the structure and excessive drifts may cause hazardous damage to the building. It is seen that in story drifts, central core is once again showing better performance by displaying the least amount of drift. Followed by that, intermediate walls are showing less story drift than perimeter walls but this difference is not as significant as the difference that central core displays.

c) Pier Forces

Pier forces are the main loads that are used in order to design shear walls. These forces include moments, axial loads, shear forces and torsional forces that are applied to each shear wall individually after analysis. Based on distribution of shear walls in plan, these forces may change and as a result, design of the shear wall may also change. More importantly, these forces may impact the behavior of shear walls, especially in higher levels. For this study, we have to choose controlling load case and critical pier number. Since all of the models are symmetric, all of the piers will behave the same but under different load conditions. In other words, any pier can be chosen as critical pier if appropriate loading is considered. Therefore, Pier 1 (PW1 in model description, shown in plan view) is chosen. Since PW1 resists lateral loads in X direction, we have to choose between earthquakes 1, 3 and 5. Earthquake 5 will result into higher loads since it includes accidental eccentricity in a direction which torsional forces add to shear forces and produce slightly higher loads. Shear forces and moments are presented both in tables and in figures.

Table 3.5: Shear on pier 1 - medium rise

Stories	**Shear on Pier 1 – Earthquake 5 (Kips)**		
	Perimeter Walls	**Intermediate Walls**	**Central Core**
Story 1	226.40	228.99	258.01
Story 2	104.71	111.09	196.22
Story 3	109.32	116.39	192.91
Story 4	95.73	103.93	179.07
Story 5	84.11	92.98	163.15
Story 6	70.54	79.79	143.13
Story 7	55.12	64.43	119.02
Story 8	36.81	46.12	90.52
Story 9	17.80	25.68	58.29
Story 10	-24.21	-10.36	15.75

It is observed that the pier in central core is attracting more shear force than the other two models. Based on concepts of structural dynamics, if a structure is stiffer, it means the structure will have a smaller natural period. By looking at design response spectrum (Figure 11.4-1 in ASCE), smaller natural period usually results into higher spectral response accelerations (S_a) that will lead to higher forces being applied to the structure. The fact that central core model which has a higher stiffness than other two models is also undergoing larger shear forces is an example of the effect of structural stiffness on seismic forces. This discussion will be further investigated in next section.

A graph based on shear forces applied to pier one, compares all of the models and clearly shows that shear forces are higher in central core than intermediate model. Perimeter walls are also showing the least amount of shear.

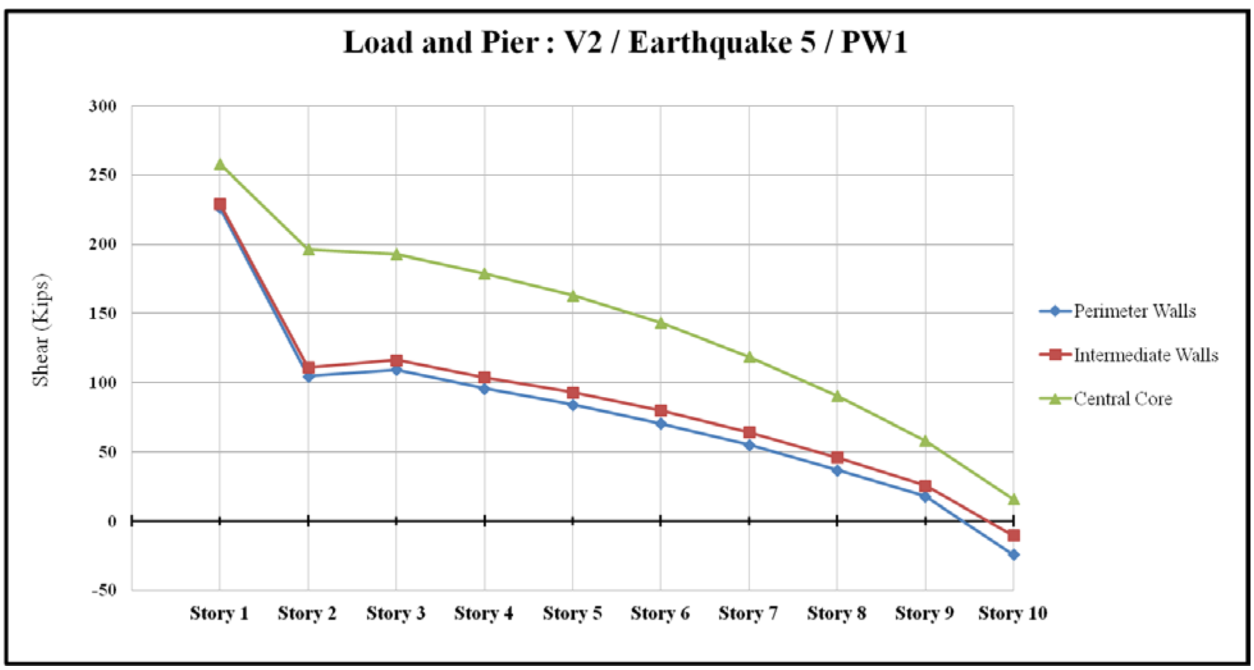


Figure 3.7: Shear on pier 1 comparison - medium rise

The same analysis has been performed to obtain moments in pier one for all three models and earthquake 5 is the most critical case for pier one. It must be considered that the forces that are obtained from the analysis also include the impact of frames on shear walls.

Table 3.6: Moment on pier 1 - medium rise

Stories	Moment on Pier 1 – Earthquake 5 (Kips - ft)		
	Perimeter Walls	Intermediate Walls	Central Core
Story 1	7,948.18	7,436.15	4,253.37
Story 2	5,288.08	4,815.30	3,302.33
Story 3	4,127.92	3,694.05	2,851.34
Story 4	2,941.78	2,570.49	2,351.09
Story 5	1,938.52	1,635.61	1,884.56
Story 6	1,086.50	853.31	1,440.89
Story 7	402.58	235.82	1,029.36
Story 8	-96.08	-201.52	657.30
Story 9	-378.17	-430.10	338.82
Story 10	-435.41	-425.92	51.24

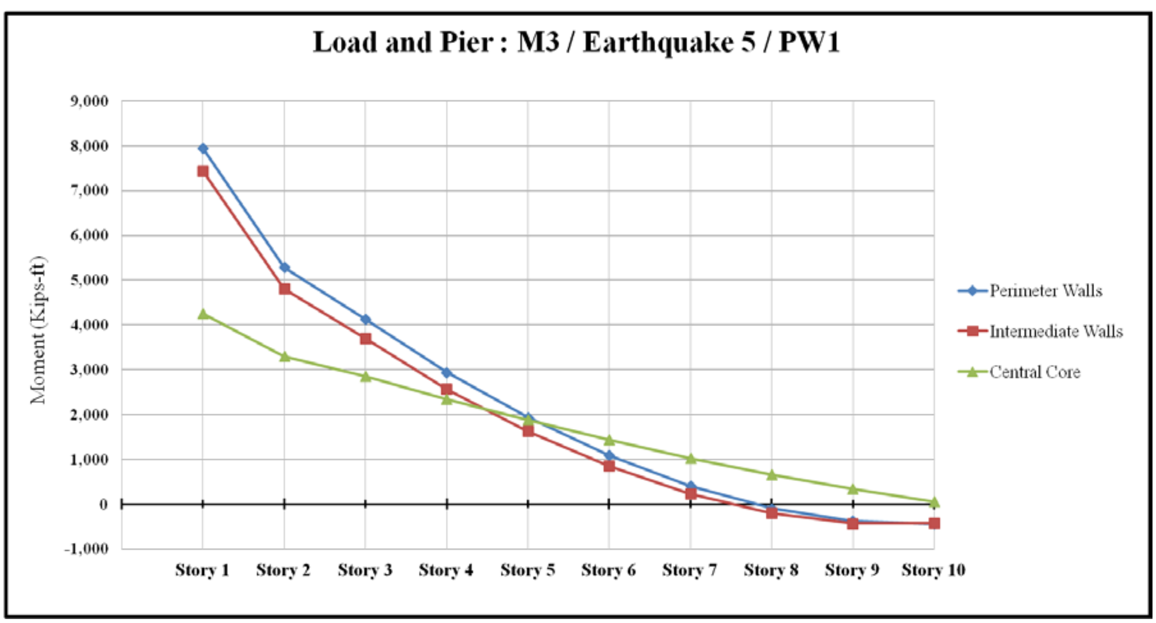


Figure 3.8: Moment on pier 1 comparison - medium rise

d) Fundamental Periods

Natural period of a structure is also in inverse relationship with stiffness ($T_n = 2\pi\sqrt{\frac{m}{k}}$) where T_n is the natural periods (sec), m is mass and k is the stiffness of a system. As stiffness increases, natural period decreases and vice versa. In the models that are created, mass (weight) is the same for all and the only parameter which changes is the lateral stiffness of the system. Therefore, it is expected that models show natural periods relative to their stiffness values. Since central core resulted into highest stiffness, lowest natural period is expected for this case. The same explanation predicts that perimeter wall will result into largest natural period.

Table 3.7: Natural periods - medium rise

Model	Natural Period (seconds)	%
Perimeter Walls	1.522	-
Intermediate Walls	1.425	- 6.4 %
Central Core	1.038	- 31.8 %

Based on the results, central core is showing a natural period of 1.038 seconds which is about 30% shorter than the natural period resulted from perimeter walls. Intermediate walls are also having a slightly smaller natural period than perimeter walls.

Two graphs are presented which display modal periods for modes 1 and 2 respectively. There is barely any difference between the two modal periods. Results presented in graphs below, better illustrate that the difference between natural period of perimeter walls and intermediate walls is not significant but central core is having a natural period that is about 0.5 seconds shorter than other two models which is a considerable amount.

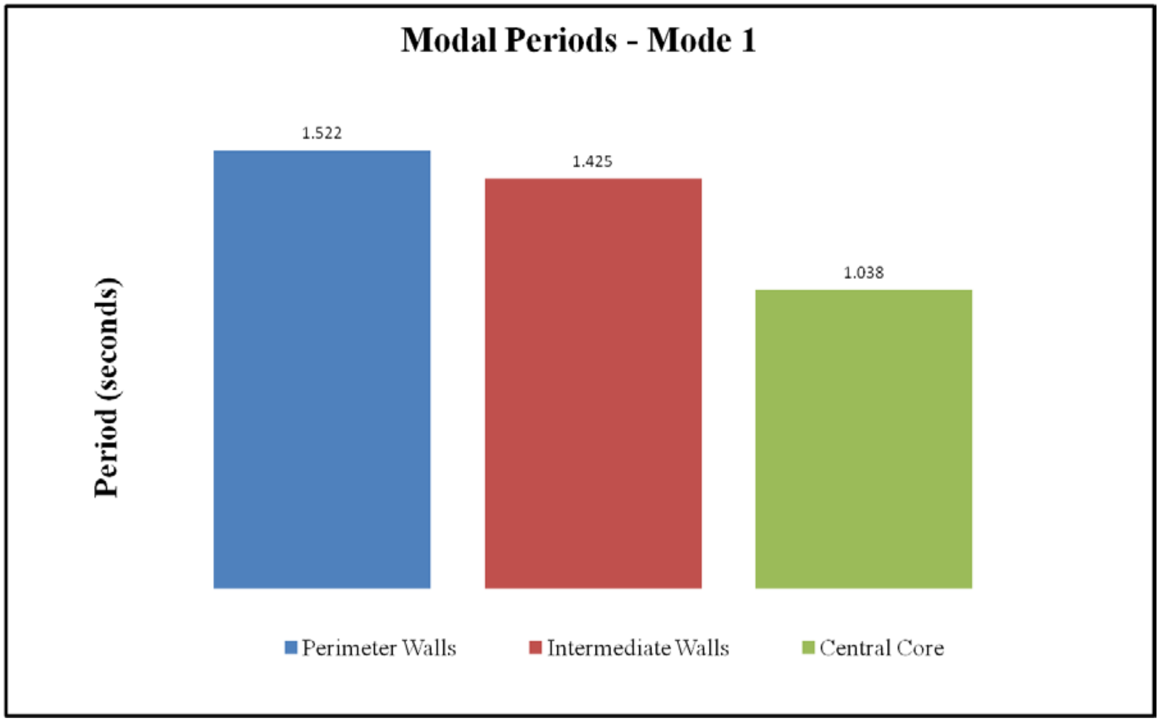


Figure 3.9: Modal periods in mode 1 - medium rise

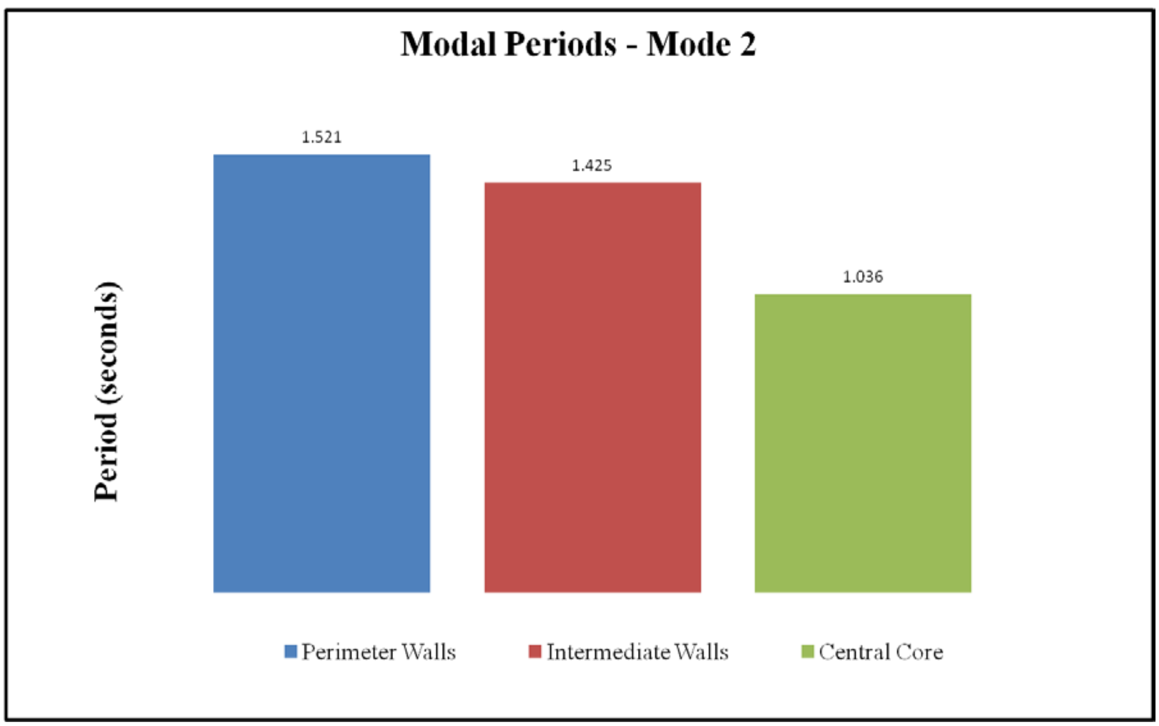


Figure 3.10: Modal periods in mode 2 - medium rise

3.2.3 Discussion

All of the results discussed in previous section follow a certain pattern. The model with a central concrete core is showing highest stiffness values, least lateral displacement, smaller drifts and a shorter natural period compared to other two models. Intermediate shear walls also showed higher stiffness than perimeter shear walls but in most cases, the results for these two models were fairly close to each other. The question is "what is the explanation behind this behavior?" Why is the model with a central core providing such high stiffness in the building while number of shear walls is the same in all three models?

The main reason for this behavior is the combined section that is formed when shear walls connect to each other at the center of the plan and create a concrete box. The new section has a much higher moment of inertia than shear walls that act individually. After shear walls form a box, all of the walls basically act as a single element. This element resists lateral forces by bending around a new neutral axis which is the axis of the box and shear walls no longer act individually. Therefore, walls that were perpendicular to direction of load are now acting as flanges for the new section which is created. The figure is a 3D model which shows how all the shear walls act together as a box to resist lateral forces (other elements are made invisible to display a better view of the central core).

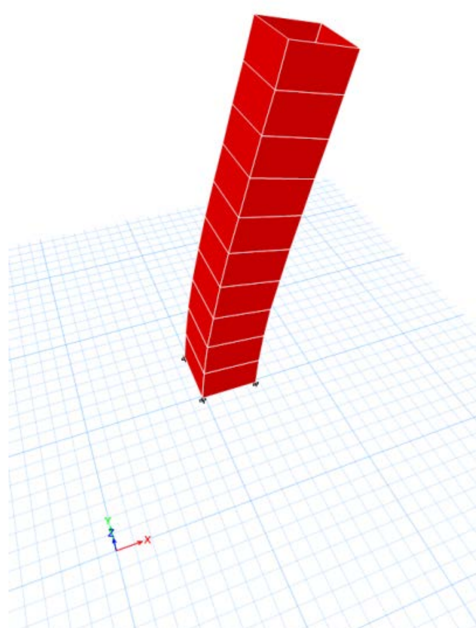


Figure 3.11: Deformed central core

This behavior is better observed when stress in shear walls is displayed. In pictures below, stress in model with a central core is compared to perimeter walls both for the same earthquake and loading condition. In the model with perimeter walls, shear walls parallel to direction of load are resisting the lateral forces and the influence of walls that are perpendicular to seismic forces are negligible (in chapter 5 this assumption is verified). As a result, walls that are not parallel to direction of load are showing small stress values (around zero). On the other hand, walls in central core act together to resist lateral loads and those which are perpendicular to direction of load, are acting as flanges for the section. High stress values, even in perpendicular walls, show the fact that these walls are all engaged in resisting lateral loads and act as a unit which creates a large moment of inertia and increases the stiffness by a significant amount. Note that these images are showing stress values on deformed shapes which are magnified and exaggerated.

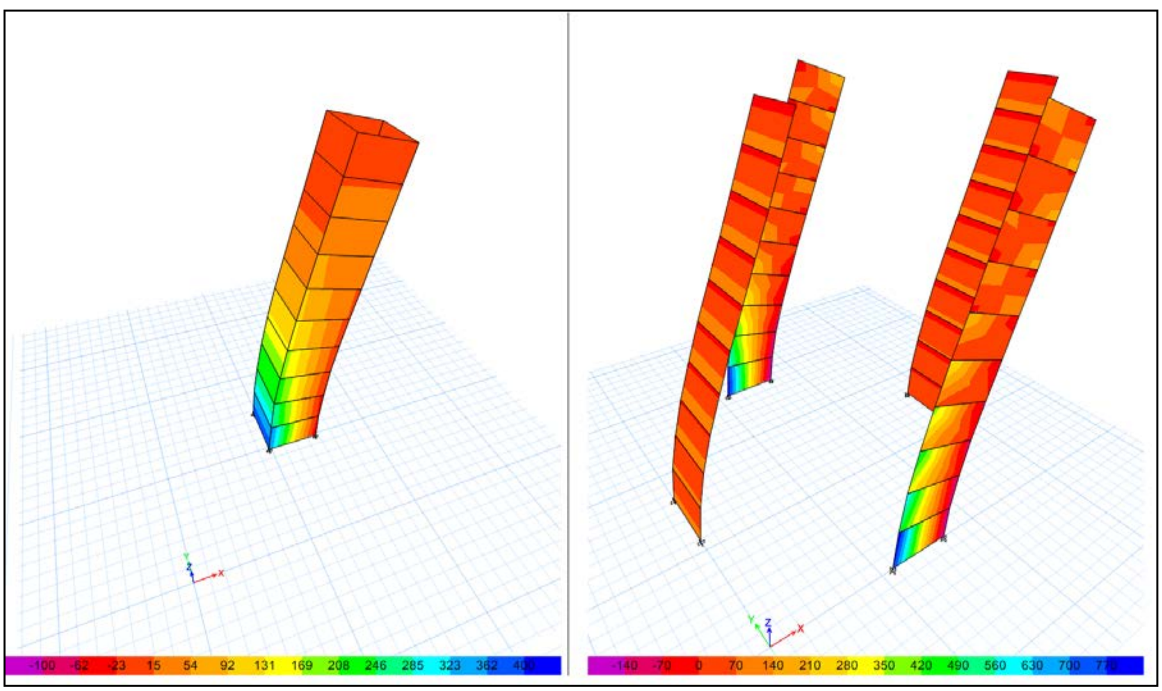


Figure 3.12: Stress in deformed shear walls - medium rise

Higher stiffness of central core is a great advantage for buildings in order to limit lateral displacements and drifts. However, there are some downsides in using a central core. As stated, walls that are perpendicular to direction of lateral loads in a box section, will act as flanges and help increase stiffness. As we know, flanges (such as those in I-shaped rolled beams) will be either in tension or compression when flexural loads are applied. The same concept is present in a box section formed by concrete shear walls. The walls that act as flanges will have to resist tension and compression caused by lateral loads in form of axial loads. These loads will then combine with other forces in standard combinations. Chart below displays the axial load in pier 2 caused by an earthquake which is perpendicular to pier 2 (Positive values represent tensile loads).

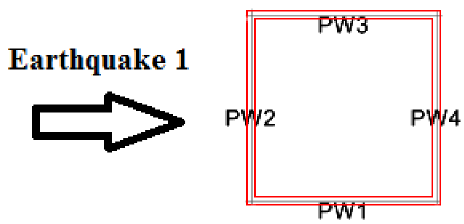


Figure 3.13: Load direction and pier labels

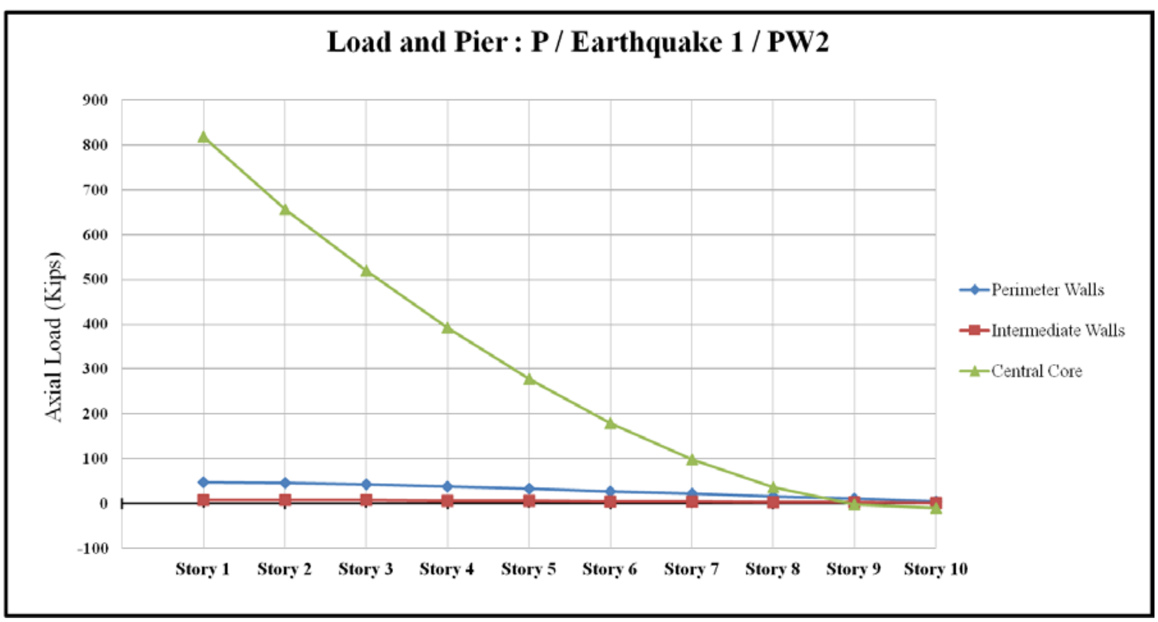


Figure 3.14: Axial loads caused in tension flange - medium rise

As seen in the graph, tension caused by earthquake in pier two is significant. It is important to know that this tensile force will be combined with other loads such as gravity loads and in this case they are opposite forces. Service gravity loads (dead + super dead + live loads) in pier 2 at base are about -2000 kips. However, on the other side of the box at pier 4, these forces will act in the same direction since pier 4 will be the flange which is in compression. Graph below shows the axial load caused by the same earthquake in pier 4. As expected, the chart is exactly the same as the one presented for figure 2 but with negative numbers since this wall is in compression.

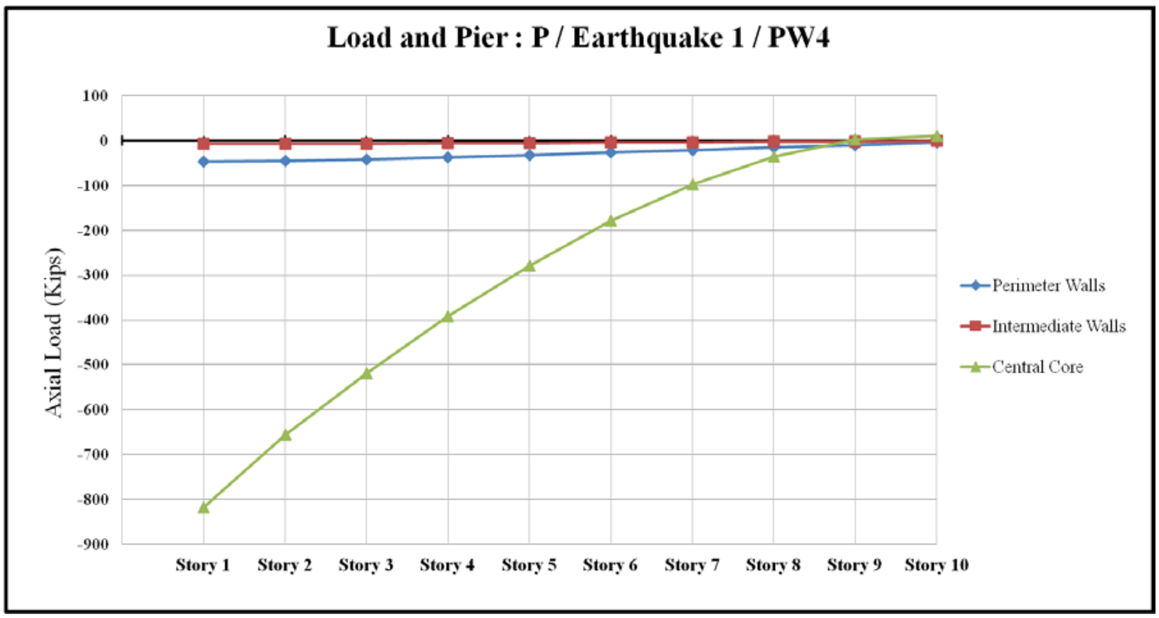


Figure 3.15: Axial loads caused in compression flange - medium rise

To put it simply, these results indicate that even though the central core is much stiffer than other models, it comes with a price which is higher loads to design for. In this case, gravity loads will add to the axial compression caused by lateral loads and since seismic forces are reciprocating loads, the same situation is expected in all piers. If only service loads are considered in this case, each pier will have to resist 2000 (gravity axial load) + 800 (seismic axial load) which is about 40% higher than the case where the only axial load is generated by gravity loads and seismic forces are ignored.

The reason why higher design loads are generally considered for stiffer models can also have another explanation based on fundamental periods and their impact on seismic forces. As it was observed in results section, the models with higher stiffness also showed shorter fundamental periods. The model with perimeter walls had a natural period of 1.52 seconds while central core showed a fundamental period of 1.04 seconds which is 30% shorter. As it was thoroughly explained in chapter 2 (general model information), lateral seismic forces are closely related to natural period of the structure. This relationship is better displayed in seismic response spectra constructed in chapter 2 where different values for T in horizontal axis, will result into different acceleration values and consequently, different seismic loads.

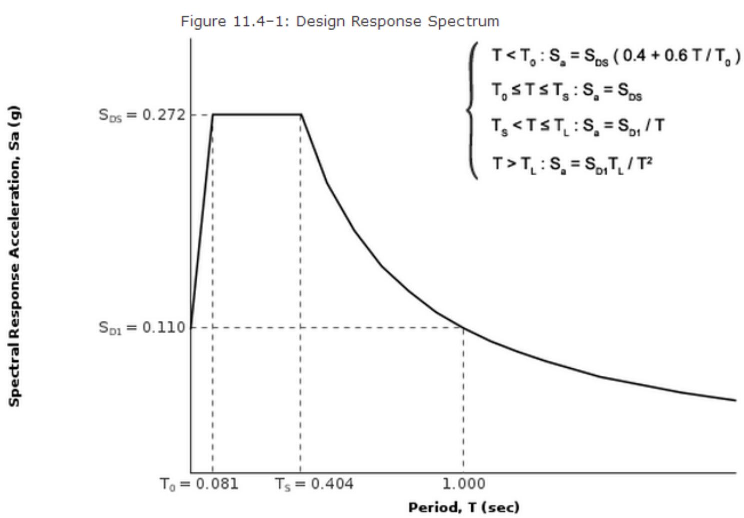


Figure 3.16: Design response spectrum

Based on the position where natural periods of models fall in response spectrum, it is expected to see higher seismic forces in central core compared to other models. Graphs shown provide more details regarding story forces (Story shear and moment). These forces are applied to each story which then will be distributed based on stiffness and layout of shear walls.

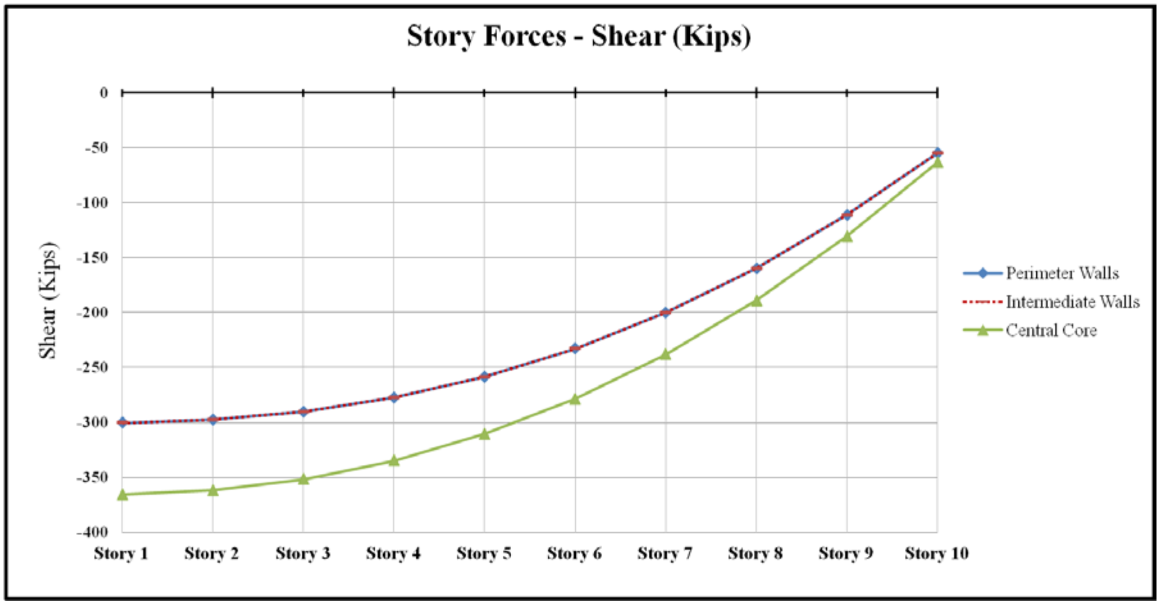


Figure 3.17: Story shear - medium rise

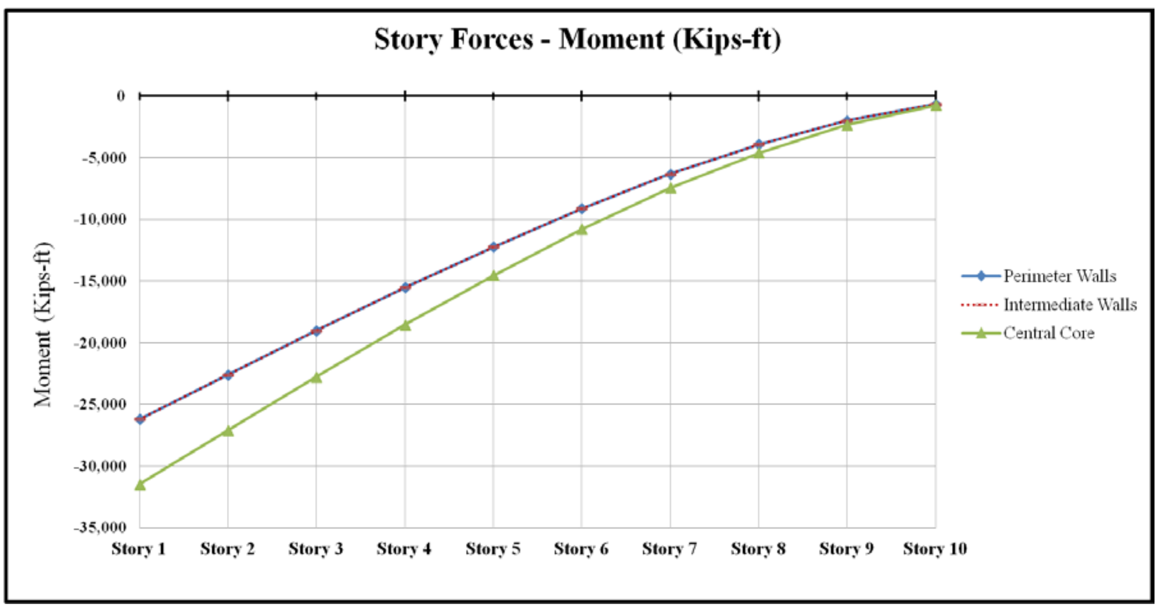


Figure 3.18: Story moment - medium rise

Results in both graphs agree with the expectations and indicate that the model with highest stiffness, because of shorter natural period, must resist higher story forces. These forces will then distribute among lateral load resisting systems which will eventually result into pier forces, used in design of shear walls.

Besides all these explanations however, there seems to be a discrepancy between what the forces were expected to look like and what is shown in the figures representing story forces (including base shear). Even though based on what was explained previously, we would expect the stiffer model between intermediate walls and perimeter walls to show higher story forces, that is not the case. Why are the forces in these two models exactly the same in all of the floors?

The reason these results are exactly the same is because of the difference that can be between fundamental period (natural period of the structure in first mode of vibration) and the period which is used in seismic analysis and design of a structure according to ASCE. These two values are not always the same. ASCE in section 12.8.2 states that "The fundamental period of the structure, *T*, in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis." However ASCE continues by talking about an upper limit on fundamental period of a structure. "The fundamental period, *T*, shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 12.8-1 and the approximate fundamental period, T_a." In section 12.8.2 ASCE states that the approximate fundamental period can be obtained using the equation 12.8-7

$$T_a = C_t h_n^x$$

Where h_n is the structural height and C_t and x are obtained from table 12.8-2. To summarize all the parameters described, table below is presented.

Table 3.8: Parameters of approximate fundamental period

Approximate Fundamental Period Parameters	**Value**
C_t (Table 12.8-2, All other structural systems)	0.02
x (Table 12.8-2, All other structural systems)	0.75
C_u (Table 12.8-1, $S_{D1} \approx 0.1$)	1.7

Based on the information obtained from ASCE, the approximate period in all of the models is calculated as below.

$$T_a = C_u C_t h_n^x = 1.7 \times 0.02 \times (10 \times 12')^{0.75} = 1.23\ seconds$$

This period is the same for all of the models while natural periods obtained from ETABS displayed different periods for each configuration since a "Modal Analysis" was performed. But ASCE states that the value based on calculations above is an upper limit on period of the structure. In section 12.9 (Modal Response Spectrum Analysis), it is mentioned that "Where the calculated fundamental period exceeds C_uT_a in a given direction, C_uT_a shall be used in lieu of T in that direction". Therefore, if any of the fundamental periods based on modal analysis obtained from ETABS is larger than 1.23 seconds, the period which must be used in design and analysis should be reduced to 1.23 seconds.

Table 3.9: Fundamental periods, upper limits and design periods

Model	Natural Period from Modal Analysis	Upper Limit	Period Used in Design & Analysis
Perimeter Walls	1.52 s	1.23 s	1.23 s
Intermediate Walls	1.43 s	1.23s	1.23 s
Central Core	1.04 s	1.23 s	1.04 s

It can now be seen that the period used in analysis of both perimeter walls and intermediate walls is the same because of the upper bound introduced by ASCE. To investigate how this limit will influence story forces, equations for story base shear must be considered. Equations 12.8-1 and 12.8-2 in ASCE determine story shear as below.

$$V = C_s W$$

$$C_s = \frac{S_{Ds}}{\frac{R}{I_e}}$$

In these equations, C_s is the seismic response coefficient and W is the effective seismic weight. In all of the models, response modification factor (R), importance factor (I_e) and effective seismic weight are the same and the only parameter which can make a difference between base shear values is the seismic response coefficient (C_s). However, this coefficient itself is limited to a minimum and maximum value based on ASCE equations 12.8-3 through 12.8-6. Table below summarizes the process for finding the adequate seismic response coefficient.

Table 3.10: Seismic respond coefficients

Model	**Seismic Respond Coefficient** $(C_s = \frac{S_{DS}}{\frac{R}{I_e}})$	**Upper Limit for C_s** $(C_s = \frac{S_{D1}}{T \times \frac{R}{I_e}})$	**Lower Limit for C_s** $(C_s = 0.044 S_{DS} I_e)$	**Final C_s**
Perimeter Walls	0.054	0.018	0.012	0.018
Intermediate Walls	0.054	0.018	0.012	0.018
Central Core	0.054	0.021	0.012	0.021

The fact that final seismic respond coefficient is the same for perimeter walls and intermediate walls proves why they are showing exactly the same results on story forces and have the same base shear. Basically the concept does not change, however ASCE is limiting the parameters to achieve a more conservative design. As a result, even though stiffness of the building was increased when intermediate walls were used compared to perimeter walls, the story forces and base shear did not change according to ASCE in this specific case.

In order to confirm the accuracy of results, it can be seen that the ratio between final seismic respond coefficient in central core to other two models is equal to 0.021/0.018 = 1.17 If based on the assumptions, all other parameters stay the same in these three models, the ratio should be approximately the same if base shears are compared. By looking at the figure for story forces, the ratio between base shear for central core to other two models is about 360/300=1.20 which is close enough to verify the discussion.

The reason why a model with intermediate walls is stiffer than the model with perimeter walls is another part of this discussion. Findings indicate that based on comparison of results, intermediate walls are showing a relatively higher stiffness and consequently a smaller lateral displacement than perimeter walls. Note that if these walls are connected through a rigid diaphragm and contribution of frames is ignored, then it is expected to see the same stiffness from lateral analysis of the two models. But as it is observed, even though the results are fairly close, there is a small difference between stiffness of intermediate walls and perimeter walls.

The explanation lies within contribution of frames to lateral stiffness of the structure. Results for both shear and moment in piers show that in top floors, forces are being reversed in perimeter walls and intermediate walls. Negative sign for values of shear imply that the wall is being pushed back and as a result, negative moments are created. In lateral analysis of shear wall-frame structures, walls and frames deform differently. In top floors if shear walls are not very stiff relative to the frame they are located within, they tend to show higher lateral displacement than frames and frames push back on the wall. This will cause reversed forces near the top floors and explains why shear and moment in perimeter and intermediate walls are showing negative values. Macgregor explains this behavior using the picture below.

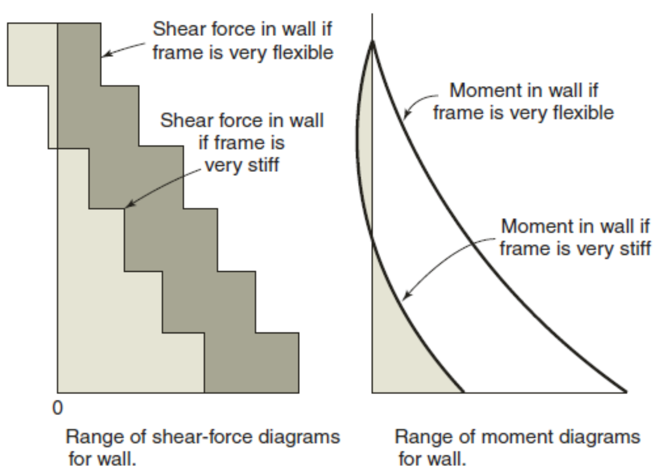


Figure 3.19: Effects of frame stiffness on shear walls
Source: Reinforced concrete mechanics & design (MacGregor)

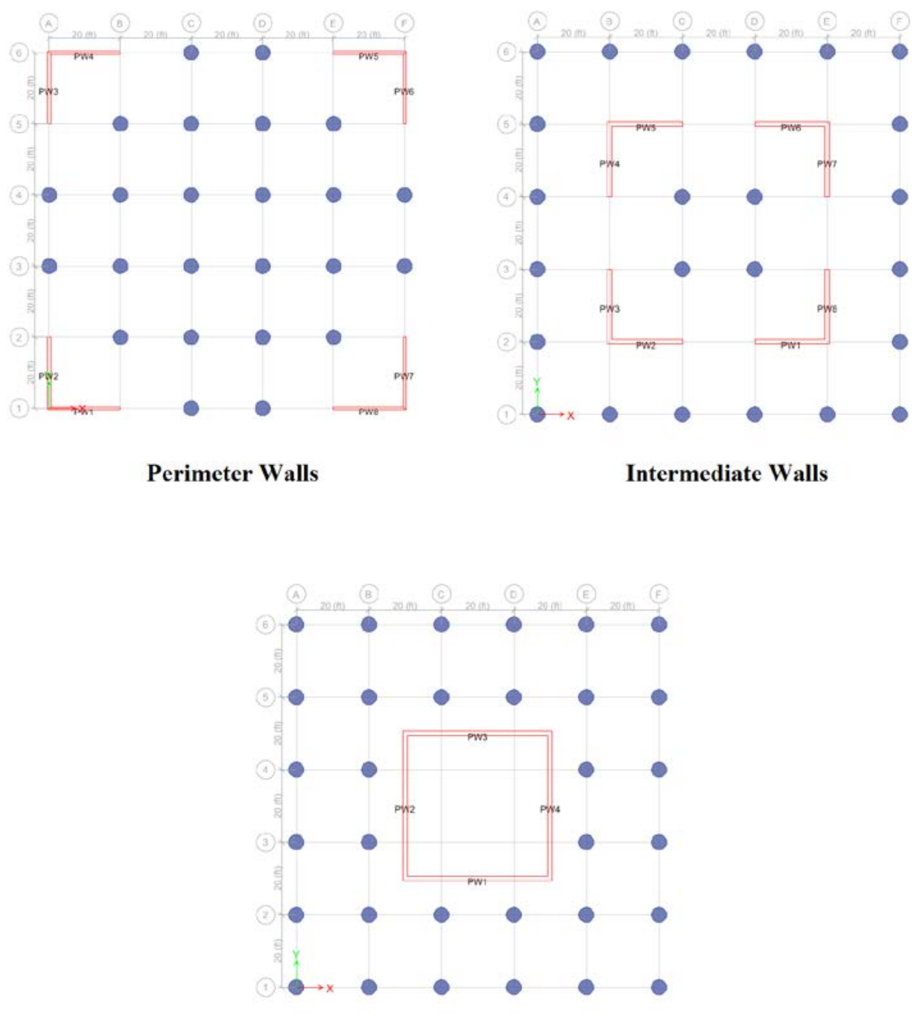
Explanations provided indicate that if a frame is stiff, it will push back on the wall at higher floors and the stiffer the frame is, the sooner the moment will change directions in height. The reason why intermediate walls are showing relatively higher stiffness than perimeter walls is the fact that intermediate walls are located in stiffer frames than perimeter walls and higher negative moments at top of intermediate walls verify this statement. Exterior frames in these models are not the same as interior frames and do not have the same stiffness. The reason is that column strip in exterior frames is smaller than interior ones and as a result, exterior frames are less stiff. Consequently, these exterior frames cannot push back on the wall as effectively as interior frames can and so in upper floors, walls that are located in perimeter of the plan can deflect easier than those located in interior frames. In other words, there is less resistance for perimeter walls to deflect in upper floors while interior frames make it harder for intermediate walls to show larger displacements at top floors.

The difference between the results for intermediate walls and perimeter walls however, are fairly close in almost all of the parameters under study. But even the small difference can show the importance of frames in shear wall-frame structures and how the interaction between the two can influence the behavior of a structure.

3.3 High Rise Tower

3.3.1 Model Description and Methodology

This model is very similar to the model described in medium rise section, especially in plan. Square footage of each floor has remained the same and a typical floor plan is 100ft x 100ft divided by columns into five 20ft spans. This building has 30 stories, spaced at 12ft and the total height of the structure is 360ft. Models that are created for this section follow the same pattern as in medium rise buildings. Figure below is a plan view of models that are studied in this chapter.



Perimeter Walls **Intermediate Walls**

Central Core

Figure 3.20: Shear wall configurations considered for high rise towers

3D models display the size of the towers and where shear walls are located within each plan. Same as medium rise buildings, shear walls have three different configurations as shown below.

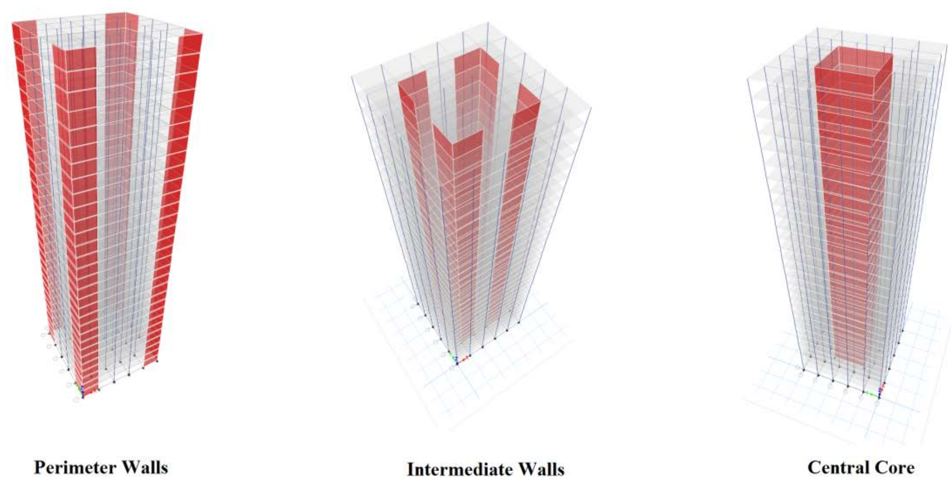


Figure 3.21: 3D view of all high rise models

As it is seen, the number of shear walls has increased from medium rise models. This is due to the fact that high rise towers can have much higher displacements if they are not stiff enough when resisting lateral loads compared to medium rise buildings because of the height of the structure. As a result, there is usually a higher demand for lateral force resisting systems that can both resist larger lateral loads and also reduce displacements especially at the top floors where it can be very troublesome. Since all of the models studied in this case must be able to resist the applied loads and do not fail under described loading situations, in each direction double the amount of shear walls that were modeled in medium rise buildings are provided. This extra stiffness can help the structure better resist large lateral deformations.

Material properties, gravity loads and seismic forces are the same as those described in general model information, chapter 2. Section properties of elements used in high rise models are presented in the table below.

Table 3.11: Section properties - high rise

Element	Dimensions
Flat Slab (4 ksi)	Thickness: 9 in.
Circular Columns (4 Ksi)	Diameter: 50 in.
Shear Walls (5 Ksi)	Thickness: 14 in. Width: 20ft

It is observed that other than providing more shear walls, the thickness of walls and diameter of columns have also increased. These dimensions are designed in a way the building does not fail under load combinations including gravity loads and seismic forces. As a result, for a high rise tower, these dimensions are usually bigger than medium rise buildings under the same loading.

For high rise models, the seismic force resisting system is also different than medium rise models. In this section, “Special Reinforced Concrete Shear Walls” are selected under building frame systems which is not limited in seismic design category B (refer to ASCE table 12.2-1). Site class, risk category and importance factors remain the same as medium rise building. Summary of this information is provided in table below.

Table 3.12: Seismic design coefficients and factors - high rise

Design Coefficients & Factors Obtained from ASCE	Value
Response Modification Coefficient (R)	6
Overstrength Factor (Ω_0)	2.5
Deflection Amplification Factor (C_d)	5
Site Class	D
Seismic Importance Factor (I_e)	1.00

Considerations for high rise towers in all of the models in this study are the same as those stated in medium rise section

- Symmetric plans with CM and CR on the center of the plan, small torsional forces are generated as a result
- Each shear wall is 20ft wide and does not include a boundary element
- Equal number of shear walls at each direction in all of the models. They all have a total number of 8 shear walls resisting lateral loads, 4 in each direction
- Floors are assumed to be rigid diaphragms in analysis of lateral loads
- Shear walls are fixed at the base and columns are pinned
- Dimensions of columns, slabs and walls do not change in height or in plan
- All of the models were analyzed and designed for a preliminary stage to verify that failure does not happen for any element in all models

Same as medium rise buildings, the attempt is to keep all of the parameters consistent and unchanged in all of the models and conclude that any change in the data provided from analysis is a result of different shear wall configurations in plan. The models are analyzed and compared in parameters such as stiffness, lateral displacements, drifts and fundamental periods. The intention is to see whether high rise towers will behave in the same pattern that medium rise buildings did or will there be difference between behaviors of the two.

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3.3.2 Results

To show the results of high rise building analyses, the same pattern as medium rise building is followed. Data is first presented in tables to show the quantity of the parameter under investigation. It is then followed by a graph which visually describes the effect of various factors and how much they influence the design. A similar behavior as medium rise buildings is expected for high rise towers as well. If assumptions and explanations in analysis of medium rise buildings stay true for this case, central core is expected to show the highest stiffness, least lateral displacement and shortest natural period.

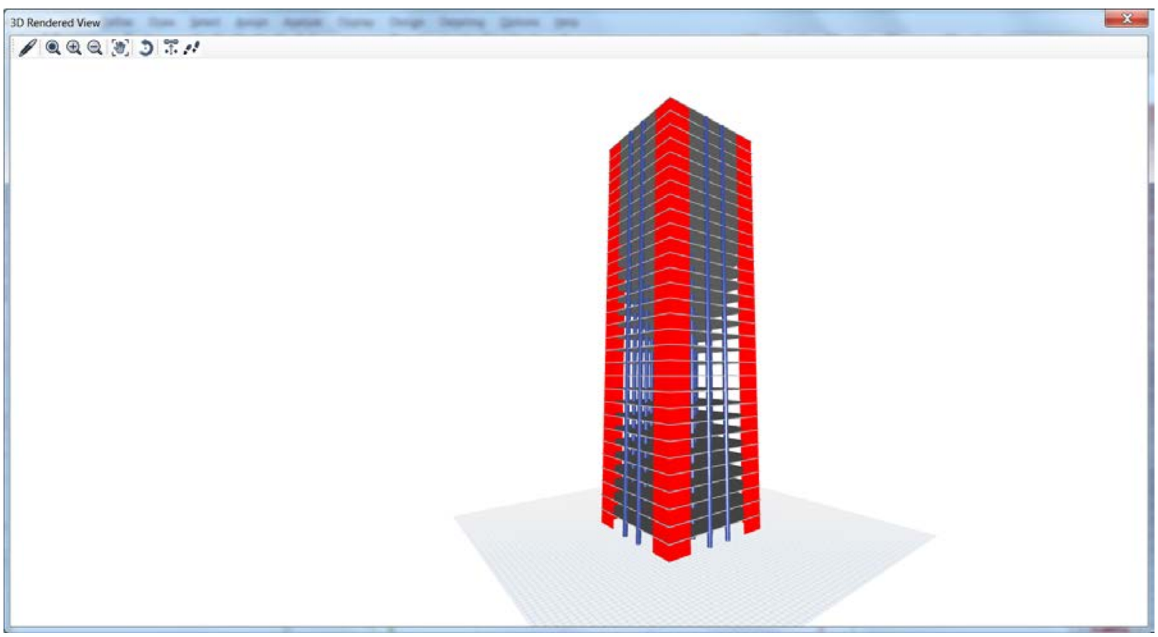


Figure 3.22: Rendered view of a high rise tower

For simplicity and conciseness, duplicate explanations are avoided since many concepts stay the same for medium rise buildings and high rise towers. Results are provided in tables followed by graphs, but if extra explanation is needed, refer to section 3.2 (medium rise buildings).

a) Lateral Stiffness

Table 3.13: Stiffness values - high rise

Stories	Stiffness (Kips/in.)		
	Perimeter Walls	**Intermediate Walls**	**Central Core**
Story1	27,306.19	34,580.54	82,803.96
Story2	11,839.30	15,577.18	47,090.21
Story3	7,651.91	10,285.40	32,905.00
Story4	5,797.40	7,923.67	25,690.49
Story5	4,746.21	6,582.11	21,351.20
Story6	4,071.50	5,722.40	18,442.54
Story7	3,602.65	5,127.57	16,362.07
Story8	3,258.27	4,693.55	14,796.44
Story9	2,994.50	4,364.04	13,572.80
Story10	2,785.50	4,105.75	12,585.42
Story11	2,615.01	3,897.63	11,766.53
Story12	2,472.19	3,725.60	11,070.11
Story13	2,349.45	3,579.74	10,463.63
Story14	2,241.25	3,452.70	9,923.12
Story15	2,143.33	3,338.79	9,430.22
Story16	2,052.31	3,233.33	8,970.33
Story17	1,965.35	3,132.28	8,531.38
Story18	1,879.95	3,031.91	8,103.02
Story19	1,793.87	2,928.64	7,676.03
Story20	1,704.92	2,818.81	7,241.86
Story21	1,611.01	2,698.56	6,792.40
Story22	1,510.00	2,563.71	6,319.71
Story23	1,399.73	2,409.70	5,815.89
Story24	1,277.99	2,231.48	5,272.98
Story25	1,142.56	2,023.67	4,682.98
Story26	991.26	1,780.60	4,037.99
Story27	822.09	1,496.80	3,330.05
Story28	633.39	1,167.57	2,552.40
Story29	423.93	789.74	1,697.45
Story30	194.05	364.21	764.91

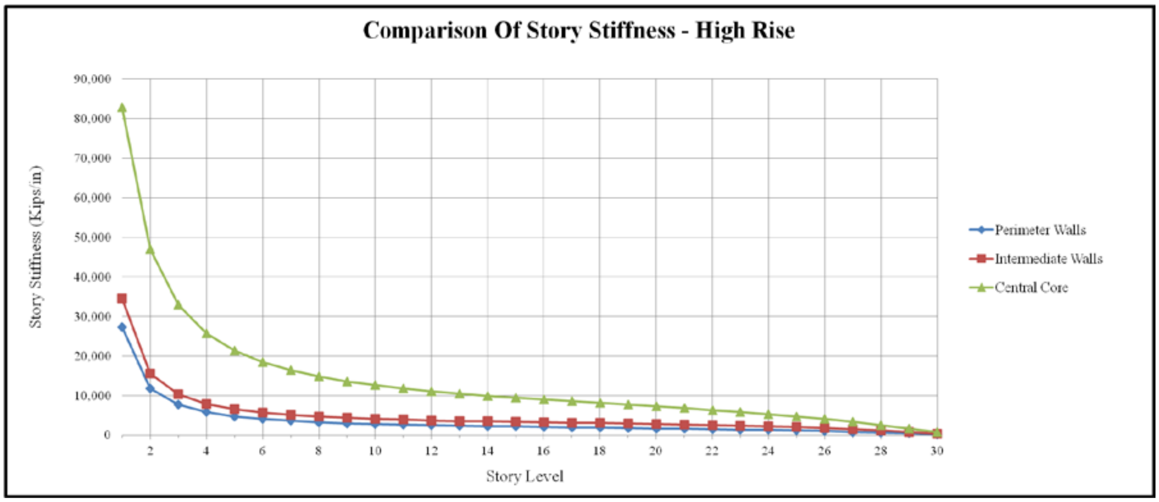


Figure 3.23: Stiffness comparison - high rise

Both the values in table and corresponding graph show that central core is having a much higher stiffness than the other two models as expected. Stiffness values at lower levels are much higher than in top levels. The reason for this is the difference between behavior of shear wall and frames in different levels and the effect that structural height can have on stiffness. Since frames tend to show more stiffness in higher levels than shear walls, top floors are controlled by stiffness of the frames but in lower levels, shear walls contribute much more to the stiffness of the high rise tower and in these stories (approximately up to first 12 floors) stiffness of the building is heavily influenced by shear walls. This being said, the stiffness at 30th floor in the model with central core is still almost double the stiffness of model with intermediate walls.

Same as the figures shown in medium rise models, the lines which represent perimeter walls and intermediate walls are moving very closely in almost all of the levels which indicate there is not a great difference between the two in terms of lateral stiffness. However, model with a central core, is way above the other two models in graph.

b) Lateral Displacement and Drift

Table 3.14: Lateral displacements and drifts - high rise

Stories	Lateral Displacement (in.)			Drift (in.)		
	Perimeter Walls	Intermediate Walls	Central Core	Perimeter Walls	Intermediate Walls	Central Core
Story1	0.03	0.02	0.01	0.03	0.02	0.01
Story2	0.09	0.07	0.03	0.06	0.05	0.02
Story3	0.19	0.15	0.06	0.10	0.07	0.02
Story4	0.32	0.25	0.09	0.13	0.09	0.03
Story5	0.48	0.37	0.13	0.15	0.11	0.04
Story6	0.67	0.51	0.17	0.18	0.13	0.04
Story7	0.88	0.66	0.22	0.20	0.14	0.05
Story8	1.11	0.83	0.28	0.22	0.15	0.05
Story9	1.36	1.01	0.34	0.24	0.16	0.06
Story10	1.62	1.20	0.40	0.25	0.17	0.06
Story11	1.90	1.39	0.47	0.27	0.18	0.07
Story12	2.19	1.60	0.54	0.28	0.19	0.07
Story13	2.50	1.80	0.61	0.29	0.19	0.07
Story14	2.81	2.01	0.68	0.30	0.19	0.07
Story15	3.12	2.22	0.76	0.30	0.20	0.08
Story16	3.45	2.44	0.84	0.31	0.20	0.08
Story17	3.77	2.65	0.92	0.31	0.20	0.08
Story18	4.10	2.86	1.00	0.31	0.19	0.08
Story19	4.43	3.07	1.08	0.31	0.19	0.08
Story20	4.76	3.28	1.16	0.31	0.19	0.08
Story21	5.09	3.48	1.25	0.31	0.19	0.08
Story22	5.41	3.68	1.33	0.31	0.18	0.08
Story23	5.74	3.88	1.41	0.31	0.18	0.08
Story24	6.06	4.07	1.49	0.30	0.17	0.08
Story25	6.37	4.25	1.57	0.30	0.17	0.08
Story26	6.68	4.43	1.65	0.30	0.17	0.08
Story27	6.99	4.61	1.73	0.29	0.16	0.08
Story28	7.29	4.78	1.81	0.29	0.16	0.08
Story29	7.60	4.94	1.89	0.29	0.15	0.08
Story30	7.89	5.11	1.97	0.28	0.15	0.08

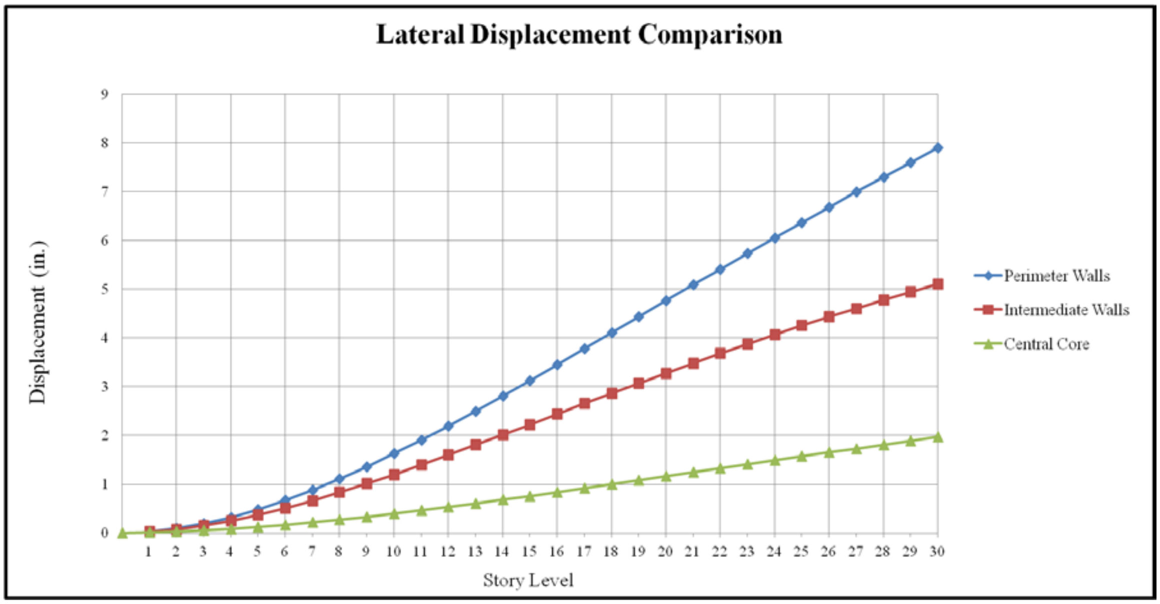


Figure 3.24: Displacement comparison - high rise

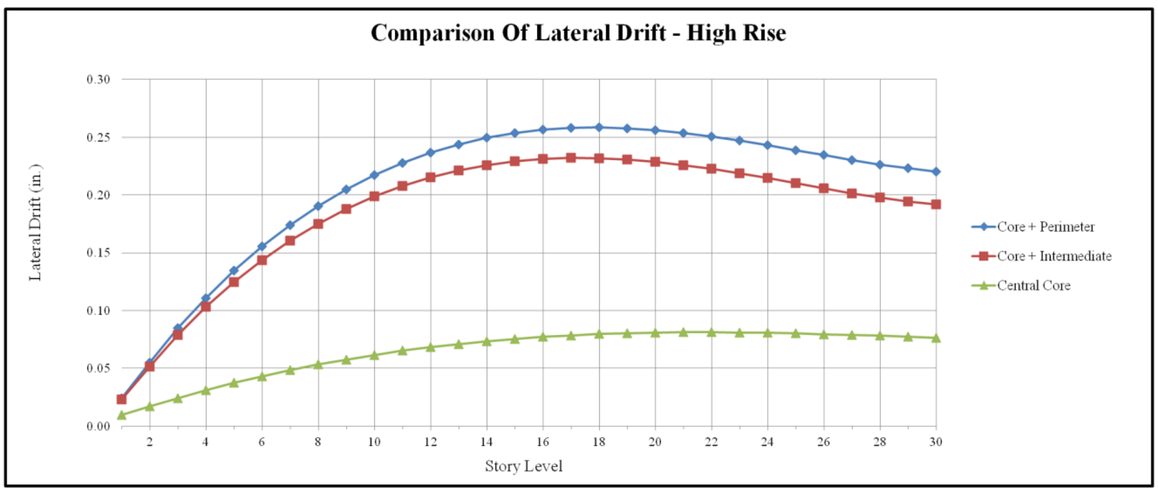


Figure 3.25: Drift comparison - high rise

Both lateral displacement and story drift values indicate that the model with central core is expected to have fewer problems in lateral movement of the structure. Maximum story displacement in model with a central core, which happens at 30^{th} floor, is only about 25% of the displacement in perimeter walls. Note that both story displacements and drifts in all models are below acceptable limits discussed in section 3.2.2

c) Fundamental Periods

Table 3.15: Natural periods - high rise

Model	Natural Period (Seconds)
Perimeter Walls	5.149
Intermediate Walls	4.151
Central Core	2.58

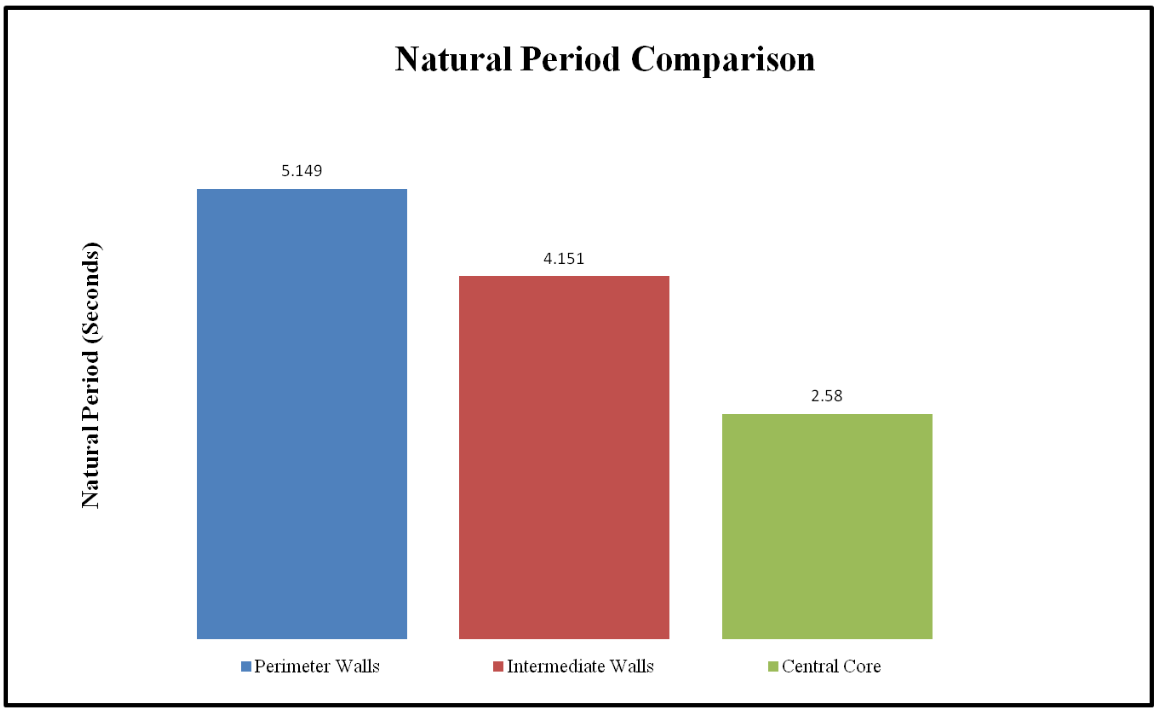


Figure 3.26: Comparison of Fundamental periods - high rise

Natural period, which is directly related to stiffness of the structure, is showing that period of first mode of vibration in central core is about 50% of shorter than natural period in perimeter walls and approximately 40% less than natural period in model with intermediate walls which is a significant difference.

3.3.3 Discussion

As expected, central core proves to be a much stiffer model against seismic loads than the other two models. Stiffness and lateral displacements in high rise towers are one of the most important considerations in design. Structural engineers tend to minimize displacements especially at top floors since these deformations can be very dangerous for safety of the structure. At the same time, large lateral displacements can cause issues with the serviceability of structures because people at top levels may feel the structure swinging from side to side. Even though the structure might be safe from strength point of view, the feeling is very unpleasant and disturbing for public. The fact that the model with a concrete core in the center of the plan is showing the smallest amount of lateral displacements explains why central cores are very popular amongst structural engineers when it comes to design of high rise towers.

It is also very important to know that dimensions of central core can play an important role in overall stiffness of the structure. To explain the importance of this issue, a quick analysis was performed. All of these models have a central core, but dimensions of these concrete boxes vary. The first two models have the same central core but with different wall layouts around the core. The third model however, is having one large concrete core at the center. Keep in mind that all of the models have the same properties, loading and number of shear walls in each direction and the only factor which is different is the distribution of these shear walls in plan.

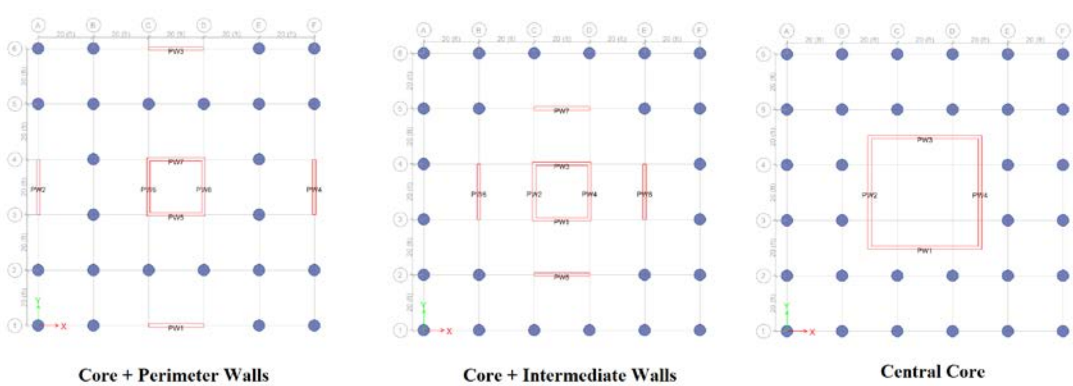
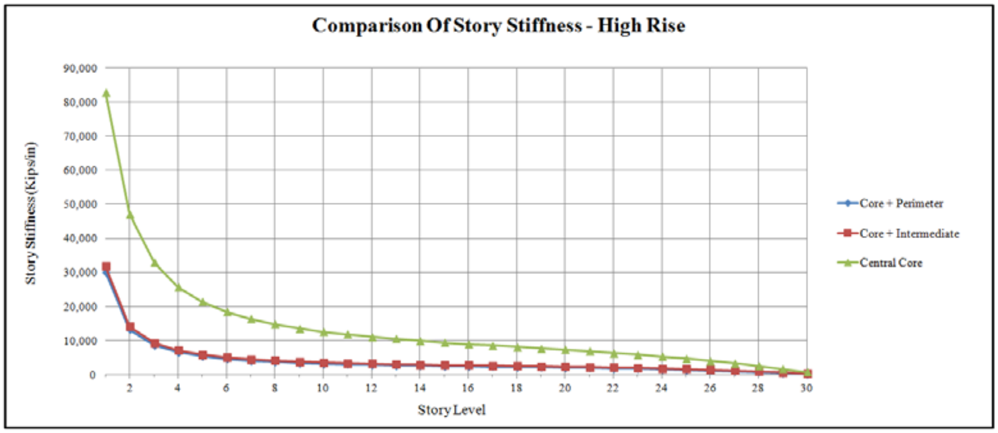
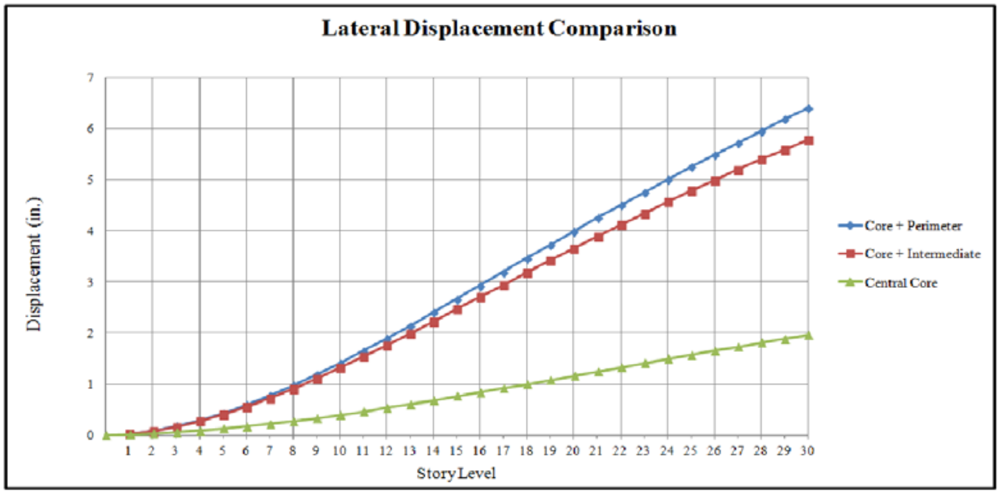
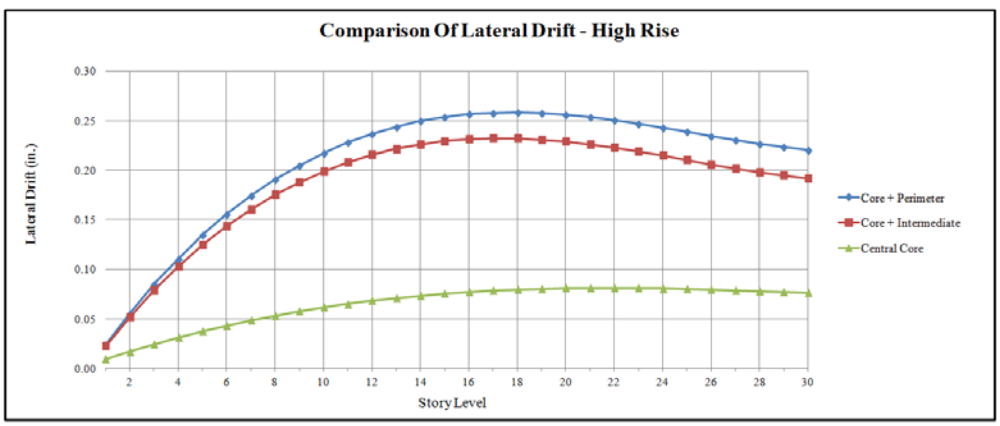


Figure 3.27: High rise models with different core sizes







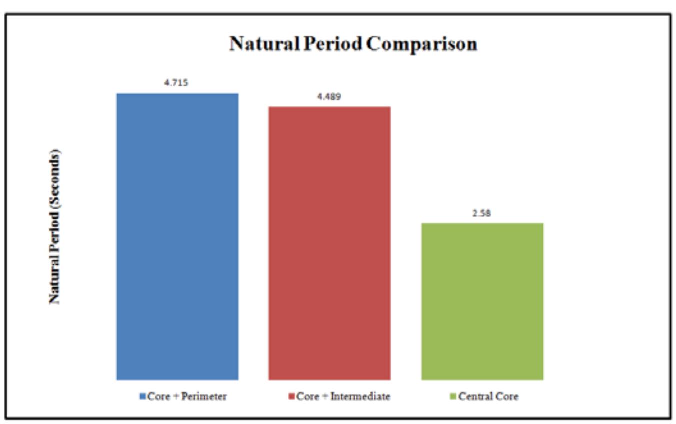


Figure 3.28: Comparison results of models with different core dimensions

Based on graphs that compare different central cores, it is observed that the model that forms one big box in the center of the plan is much more efficient in resisting lateral displacements by showing a high stiffness compared to other central cores. So even though all of these models have a concrete box in the middle acting as a core, results are very different. The fact that in this model all of the walls act together is the main reason why better performance is observed. The central core connects all of the shear walls together and forms a very stiff element at the center of the structure just as explained in medium rise buildings. Even though other models also have a core in the middle, the rest of the shear walls in these two models do not act together and therefore do not contribute as much to the overall stiffness of the system. Pictures below display how in the bigger central core, all of the walls act together and form flanges to resist lateral loads while on the model with core and perimeter walls, some of the individual walls do not contribute to the stiffness of the system.

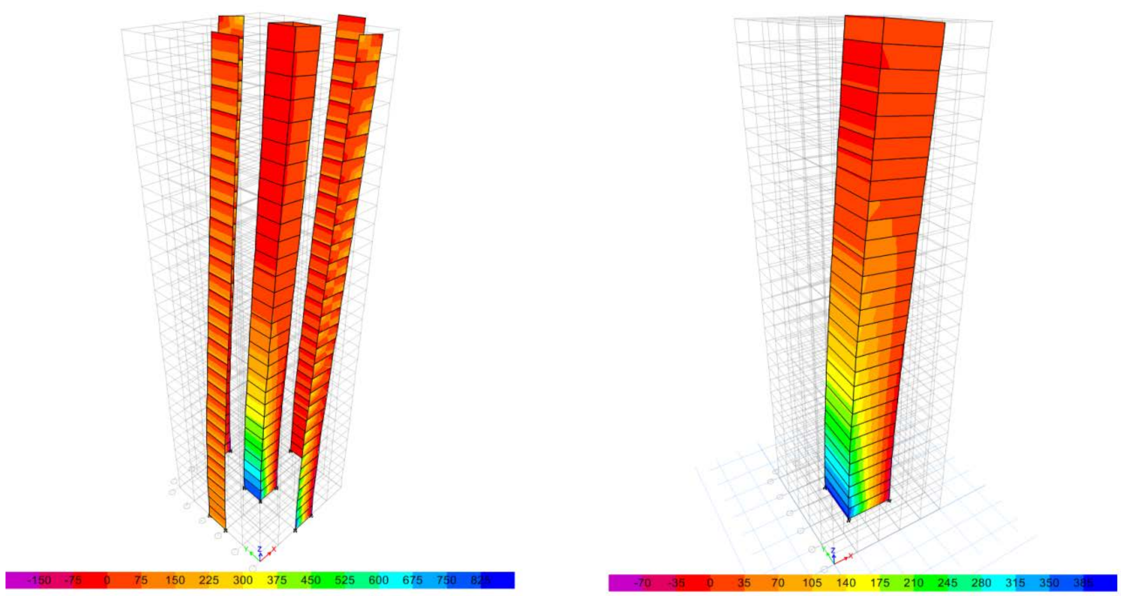


Figure 3.29: Stress in deformed shear walls - high rise

As explained in medium rise buildings, the same disadvantages in using stiffer system apply to high rise towers, which is larger design loads for the structure. Flanges will be in tension or compression and at the same time, stiffer building might be experiencing higher seismic forces. Figures below compare the story shears and moments in all models. The same discussions provided for medium rise buildings, are valid in this section and explain why perimeter walls and intermediate walls have the same story forces in all levels.

Figure 3.30: Story shear - high rise

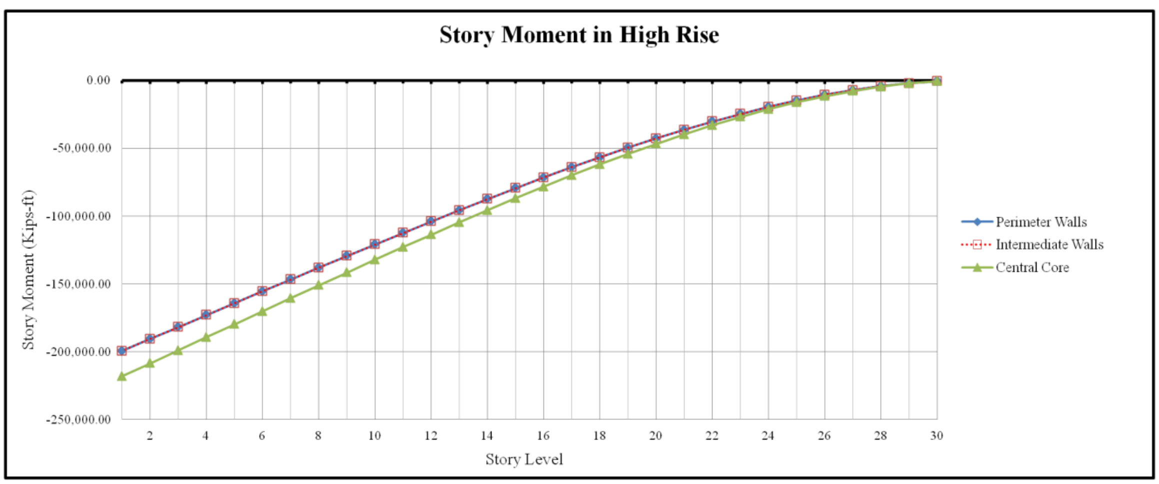


Figure 3.31: Story moment - high rise

Another phenomenon which is more observable in high rise towers is the influence of frames in shear wall-frame lateral load resisting systems. As explained in section 3.2, frames deform differently than shear walls and in top levels, they tend to show higher stiffness. The result is that walls which have smaller stiffness tend to deform more than the frame but since the system is acting together, frame pushes the wall back. In high rise towers, this behavior happens more often and it causes the wall to bend in different directions. First it bends in direction of loading in lower levels since the walls are having higher stiffness than the frame, but as the levels increase frames gradually become stiffer than the wall and push back on it, causing the wall to bend in the opposite direction of loading. Picture below, which shows an elevation from a shear wall in one of the high rise models, displays this behavior. In the first few floors (up to 12^{th} or 13^{th} floor) shear wall bends in the direction of loading, causing compression on the right side and tension on the left side of the wall shown in the picture. However in levels higher than that on the right side of the wall, yellow and orange colored stresses are slowly emerging which is a sign of tensile stresses. In other words, it means the wall is being pushed in the opposite direction and will bend the other way. It must be stated that this behavior highly depends on relative stiffness of walls and frames which can differ greatly from case to case.

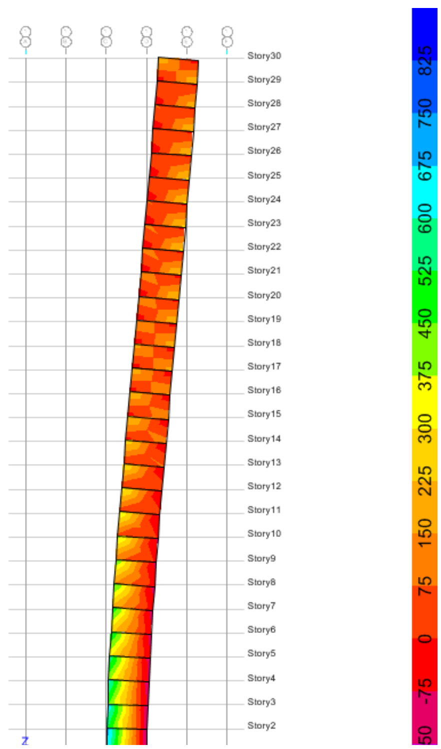


Figure 3.32: Effects of frames on shear walls at top floors

Chapter 4

Effect of Cracking on Stiffness of Shear Walls

4.1 Introduction

There is a famous adage about concrete which says “There are two guarantees about concrete. One, it will get hard and two, it will crack!” Cracking is an inevitable issue in concrete structures which can be a result of different factors such as applied loads, shrinkage, thermal loads and settlements in the structure. When concrete is in tension, if tensile stress of a particular element grows beyond rupture stress, cracking will occur and that element will not have the same stiffness as it used to have prior to cracking.

It would be ideal that member stiffnesses reflect the degree of cracking caused by applied loads to each member. However in reality, some of the complexities in assigning different stiffnesses make the analysis inefficient. ASCE 7-10 in chapter 12.7.3 states that the models that are created to analyze the forces and displacements in a structure, must consider the effects of cracked sections on stiffness properties of concrete and masonry elements. The reason is that the lateral deflection which a structure sustains under factored lateral loads might be substantially different from what is obtained using linear analysis. This is due to the fact that members show inelastic responses and a decrease in effective stiffness is inevitable. A simple way described in ACI code in order to estimate an equivalent nonlinear lateral deflection using linear analysis is to reduce the Stiffness of concrete members using stiffness modifiers.

ACI provision 8.8.2 states that lateral deflections of reinforced concrete building systems resulting from factored lateral loads shall be computed by linear analysis with member stiffnesses defined as below.

Table 4.1: Cracked stiffness modifiers

Element	Moment of Inertia, I
Columns	0.7Ig
Walls – Uncracked	0.7Ig
Walls –Cracked	0.35Ig
Beams	0.35Ig
Flat Plates and Flat Slabs	0.25Ig

In commentary (R.10.10.4.1), ACI explains that “If the factored moments and shears from an analysis based on the moment of inertia of a wall, taken equal to 0.70Ig, indicate that the wall will crack in flexure, based on the modulus of rupture, the analysis should be repeated with I = 0.35Ig in those stories where cracking is predicted using factored loads.” These stiffness reduction factors will result into larger lateral displacements and the overall stiffness of structure will drop. At the same time, lower stiffness of an element (due to cracking) means less force will be attracted by that element and the rest will be passed to adjacent members that are not cracked and have a higher stiffness.

In this chapter, main focus is on investigating the effects of cracking on stiffness and lateral displacement of shear walls using ACI provisions explained above. The attempt is to implement a more precise method in identifying elements that will crack under applied loads and study how these cracked elements, influence the behavior of shear walls.

4.2 Model Description and Methodology

Walls subjected to lateral loads, bend about their neutral axis. As a result, tension stresses are formed in one half of the section and compression stresses on the other half. Meanwhile, walls resist shear and axial loads as well. Therefore a small element in the wall undergoes stresses from axial loads, shear forces and bending at the same time. By transforming the stresses, principal stresses can be found. Once the structure is analyzed using ETABS, principal stresses can be obtained for all elements in shear walls. Based on ACI equation 9-10, modulus of rupture is calculated using the equation below.

$$f_r = 7.5 \times \sqrt{f'_c}$$

Where f_r is the modulus of rupture and f'_c is the compressive strength of concrete both in psi. If the stress that is obtained from analysis is larger than the stress obtained from equation above, it can be assumed that the element will crack under applied loads. Since the compressive strength that was used for shear walls in this study is 5000psi, modulus of rupture will be

$$f_r = 7.5 \times \sqrt{5000} \cong 530 \, psi$$

Based on the approach explained in introduction, it can be assumed that any element in the wall which has a tensile stress of more than 530 psi will crack and stiffness modifiers for cracked wall shall be applied to that element. This modifier based on ACI code is 0.35Ig. But if the stress in a particular element is less than 530 psi, that element can be assumed as uncracked wall and the stiffness modifier applied to that section is 0.7Ig. These modifiers will adjust the results and produce a more realistic value for displacements and element forces by considering the effects of cracking.

To be able to recognize which elements of the wall will crack and need stiffness multipliers, a fine mesh is applied. By meshing the walls into smaller segments, data is obtained for each of the elements individually and modifiers can be applied to that particular section. Otherwise if the wall is not properly meshed, data is generated for bigger elements which will not lead to a precise analysis. There is a downside to this approach however. Since there are hundreds of new elements created due to meshing, ETABS generates massive amount of data which requires strong computers in order to process all of the output. Therefore, it can take a long time for regular computers to analyze even medium sized structures. To overcome these obstacles, a simple model is required which is able to represent certain conditions.

A cantilevered shear wall is considered for this chapter. All of the geometric dimensions of the new model are the same as those used in medium rise building, a ten story shear wall with a total height of 120 ft, thickness of 10 inches and width of 20 feet. The wall is then meshed into 600 equal elements (2ft x 2ft) that are connected to adjacent members. A factored lateral load of 216 Kips is also distributed vertically (this is an arbitrary load which is selected to demonstrate cracking in the shear wall and can be assumed as the base shear).

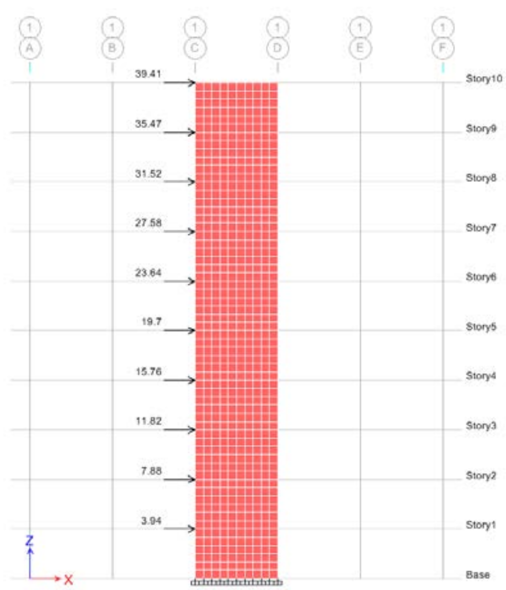


Figure 4.1: Meshed shear wall and applied loads

The base shear is distributed using equations below:

$$F_x = VC_{vx}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

Where F_x is the force at level x, V is the base shear which in this case is 216 kips, w_i is weight and h_i is the height at each level. After meshing the wall and distributing the vertical forces, the model is ready for analysis. It is important to know that all the other properties and variables that are not mentioned here, are the same as those described in general model information including the compressive strength and modulus of elasticity. Based on explanations above, it can be assumed that the model represents a shear wall which is used in a 10 story building and after performing the analysis, the forces that are applied to the wall at each level are obtained and then applied to the wall at a separate model.

In order to achieve a more precise analysis, redistribution of forces must be considered. When loads are applied to elements with equal stiffnesses, forces are also distributed equally among all of them. But when stiffness of one of the elements is smaller than the rest of memebers due to cracking, less force is attracted by cracked element and more is passed to the adjacent uncracked elements as shown in figure below.

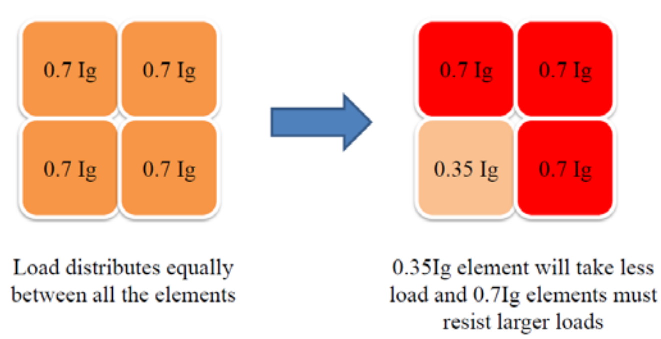


Figure 4.2: Redistribution of loads after cracking

Based on the discussion above, assume that in the first run certain elements are cracked and stiffness modifiers are applied. If the analysis is performed again, there is a chance that the elements adjacent to them (which were not cracked in the previous analysis) surpass the modulus of rupture and crack in the new analysis. The reason is that more force is being passed on by cracked elements to uncracked elements since they have double the stiffness and therefore, attract more loads. As a result, each time the model is analyzed and modifiers are applied, another round of analysis is performed to see how other elements, especially those adjacent to previously cracked elements would react. If the new tensile stress is higher than modulus of rupture, stiffness modifiers will also be applied to these newly cracked members and the wall will be analyzed again. This process continues until no new element is cracked due to redistribution of forces and results start to converge. Figure below describes the methodology used in this chapter.

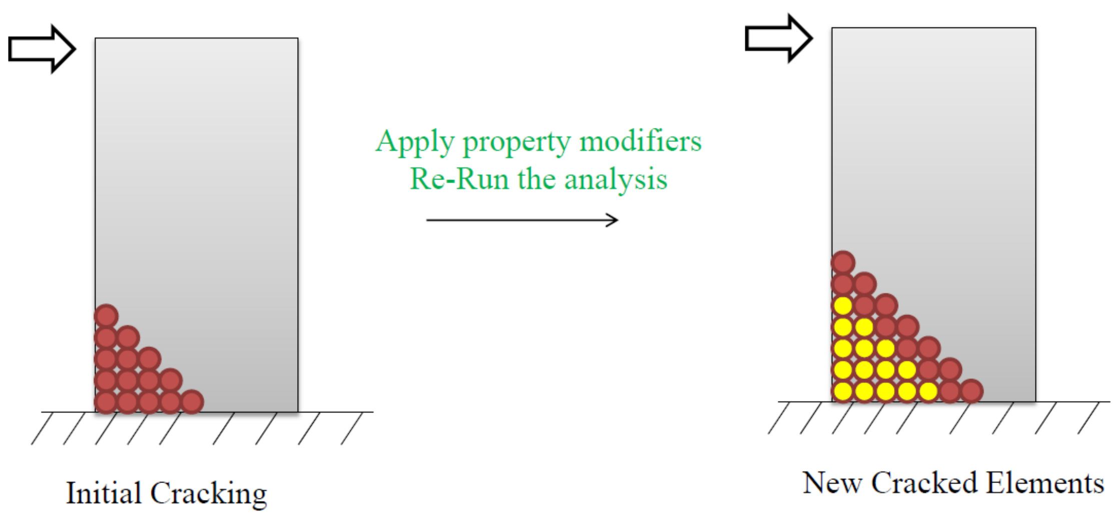


Figure 4.3: Methodology of cracked analysis

4.3 Results

The model described in previous section is analyzed in multiple steps until results converge and no new cracked element appears in analysis. The progress at every level is described and results are presented for each step until convergence is achieved.

4.3.1 No Cracking (initial analysis)

At first, the model is analyzed assuming that all of the elements are uncracked, so a stiffness multiplier of 0.7Ig is applied to all of the elements before analysis. Table below shows the results of lateral displacements for all stories in first analysis

Table 4.2: Story displacements - initial analysis

Stories	Story1	Story2	Story3	Story4	Story5	Story6	Story7	Story8	Story9	Story10
Displacement (in.)	0.09	0.31	0.64	1.06	1.55	2.10	2.68	3.28	3.89	4.51

In picture below, blue elements are those that have a stiffness multiplier of 0.7Ig (all the elements). The other picture illustrates stresses on deformed shape of the wall.

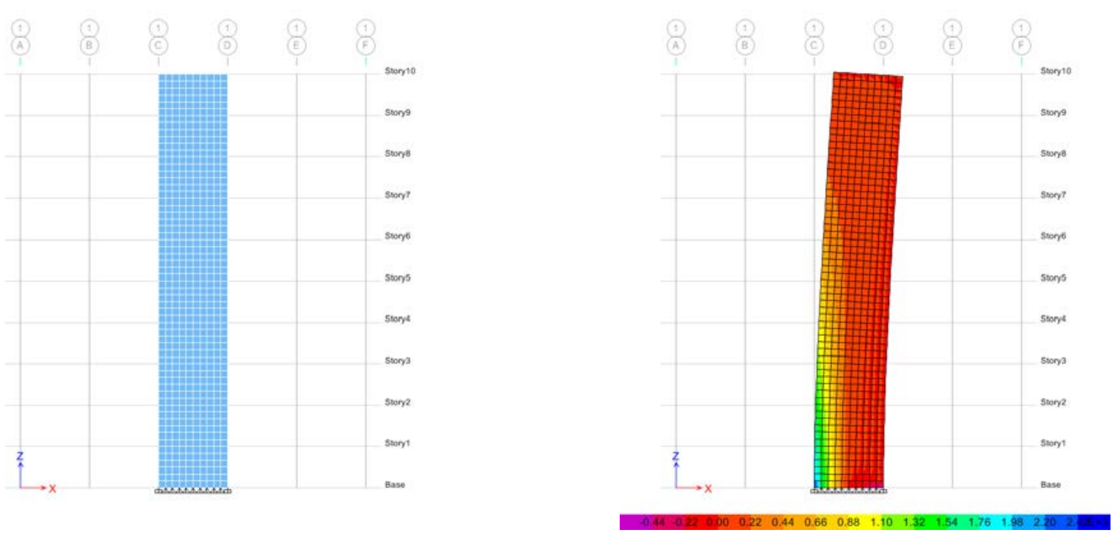


Figure 4.4: All uncracked elements and stresses on deformed shape - initial analysis

The information which is required is the maximum tensile stress on each of the 600 elements in the wall. Picture below shows the values of S_{max} which is obtained from ETABS and exported into Excel. Data is sorted using VBA in a way that each value is placed at the position of the element it’s representing. A conditional formatting is used with number 530 being the midpoint from green to red (530 psi represents the modulus of cracking). As it is seen, the pattern of stresses in the wall follows a logical order as it was expected. Maximum tensile stress is happening at the corner of the wall and it decreases with height. In the picture on the right, all the elements that have a tensile stress more than 530 psi are highlighted in red and the number of those elements is stated below that column. This way, it is known how many elements are cracked in the first round of analysis and where these elements are located.

190.05	25.64	12.88	5.98	4.83	3.29	2.51	1.45	0.71	0.43
160.79	30.49	27.09	16.19	14.05	11.57	10.15	6.89	4.02	0.74
87.88	33.47	16.77	18.28	16.07	17.16	14.24	11.38	6.78	1.78
73.34	34.11	12.02	16.7	17.89	18.8	18.33	13.81	9.17	2.76
74.64	26.25	21.93	18.7	19	22.33	19.83	16.13	10	3.37
73.5	59.75	36.28	28.37	26.99	24.76	22.43	16.72	10.66	3.96
87.92	77.12	52.55	42.24	33.93	29.47	23.64	17.74	10.68	4.25
98.18	65.71	64.2	48.83	40.27	32.19	25.67	17.91	10.89	4.83
107.44	76.17	58.78	51.52	42.53	35.06	26.46	18.54	11.01	5.11
121.99	88.55	62.82	53.08	45.44	36.47	27.84	18.76	11.3	5.79
144.32	81.51	76.02	61.62	49.24	39.26	28.76	19.43	11.71	6.16
132.52	124.57	93.28	72.84	56.3	42.31	30.46	20.03	12.19	6.93
156.25	151.25	112.55	86.03	63.53	46.15	31.98	20.9	12.83	7.42
205.48	140.91	125.89	93.93	69.31	49.11	33.66	21.71	13.44	8.25
224.36	168.19	126.15	98.24	72.41	51.53	34.83	22.53	14.17	8.83
247.85	187.37	136.48	103.11	75.69	53.18	36	23.27	14.81	9.7
276.83	190.36	155.13	113.9	80.38	55.56	37.07	24.04	15.58	10.35
262.94	235.83	175.75	127.11	87.58	58.54	38.45	24.89	16.28	11.26
296.18	265.08	197.02	141.43	95.25	61.91	39.92	25.77	17.13	11.99
361.33	262.93	213.12	151.28	101.34	64.79	41.32	26.69	17.89	12.92
387.66	297.2	219.4	158.25	105.38	66.97	42.44	27.51	18.76	13.71
418.33	321.96	235.42	166.58	109.47	68.62	43.39	28.33	19.54	14.68
453.05	333.56	258.4	179.84	115.01	70.69	44.36	29.1	20.42	15.52
448.79	380.81	282.05	194.83	122.53	73.4	45.49	29.99	21.24	16.51
486.07	412.48	305.25	210.43	130.51	76.34	46.79	30.87	22.18	17.41
557.1	418.66	324.44	222.26	137	79	47.99	31.83	23.06	18.44
589.98	457.66	336.26	231.97	141.86	80.95	49.03	32.69	24.03	19.38
626.77	487.16	356.95	243.36	146.81	82.5	49.88	33.57	24.93	20.44
666.62	506.17	383.15	258.8	153.05	84.31	50.78	34.4	25.91	21.42
673.34	554.45	409.36	275.24	160.91	86.68	51.76	35.34	26.84	22.52
714.35	588.52	434.37	292.08	169.1	89.24	52.94	36.26	27.88	23.52
786.98	602.49	456.56	305.79	176.02	91.6	54.02	37.27	28.85	24.66
825.46	644.75	473.36	318.25	181.62	93.36	55.02	38.19	29.92	25.69
867.33	678.19	497.7	332.05	187.4	94.76	55.84	39.16	30.9	26.87
911.46	703.56	526.2	349.29	194.22	96.33	56.7	40.03	32	27.91
928.94	752.06	554.44	366.97	202.39	98.34	57.61	41.05	32.98	29.13
973.18	788.28	581.06	384.77	210.67	100.54	58.7	41.99	34.14	30.18
1045.04	809.3	605.92	400.18	218.04	102.55	59.71	43.08	35.15	31.45
1088.14	853.68	627	415.11	224.28	104.14	60.69	44.02	36.34	32.5
1134.09	890.27	654.18	430.79	230.96	105.34	61.5	45.09	37.34	33.81
1181.57	921.04	684.16	449.4	238.08	106.73	62.36	45.99	38.57	34.85
1209.1	968.92	713.92	467.99	246.51	108.34	63.21	47.11	39.56	36.22
1255.92	1006.88	741.97	486.54	254.78	110.22	64.26	48.04	40.86	37.24
1325.33	1034.18	768.91	503.37	262.61	111.83	65.21	49.25	41.83	38.68
1372.05	1079.68	793.57	520.4	269.35	113.3	66.23	50.16	43.2	39.66
1421.11	1118.62	822.88	537.44	276.88	114.27	67.03	51.39	44.12	41.18
1471.02	1153.95	853.59	557.02	284.09	115.57	67.98	52.25	45.57	42.09
1507.82	1200.42	884.37	576.05	292.85	116.73	68.81	53.57	46.44	43.72
1556.58	1239.9	913.62	595.51	300.8	118.45	69.94	54.44	48.01	44.52
1622.39	1272.61	942.51	613.13	309.13	119.64	70.94	55.95	48.81	46.31
1671.98	1318.47	969.64	631.93	316.21	121.32	72.22	56.89	50.61	46.91
1723.5	1358.98	1000.67	649.92	324.62	122.21	73.46	58.71	51.31	49
1774.76	1398.19	1031.52	670.73	332.33	124.31	74.93	59.9	53.54	49.11
1819.46	1441.88	1063.73	690.65	342.32	125.49	76.78	62.03	53.98	51.76
1866.69	1482.84	1094.25	713.3	351.89	128.24	77.06	63.02	56.48	50.42
1922.62	1518.21	1128.25	733.75	362.41	128.65	75.86	60.98	55.22	53.74
1961.82	1565.05	1160.25	758.03	368.42	123.51	74.14	54.86	50.62	49.18
2000.99	1611.18	1208.97	765.89	362.7	111.89	55.01	53.99	43.07	41.87
2038.58	1728.47	1204.43	754.49	341.71	71.45	18.19	3.93	51.96	60.66
2444.67	1651.7	1167.19	720.73	300.39	11.88	-18.94	-17.33	-13.28	99.54

190.05	25.64	12.88	5.98	4.83	3.29	2.51	1.45	0.71	0.43
160.79	30.49	27.09	16.19	14.05	11.57	10.15	6.89	4.02	0.74
87.88	33.47	16.77	18.28	16.07	17.16	14.24	11.38	6.78	1.78
73.34	34.11	12.02	16.7	17.89	18.8	18.33	13.81	9.17	2.76
74.64	26.25	21.93	18.7	19	22.33	19.83	16.13	10	3.37
73.5	59.75	36.28	28.37	26.99	24.76	22.43	16.72	10.66	3.96
87.92	77.12	52.55	42.24	33.93	29.47	23.64	17.74	10.68	4.25
98.18	65.71	64.2	48.83	40.27	32.19	25.67	17.91	10.89	4.83
107.44	76.17	58.78	51.52	42.53	35.06	26.46	18.54	11.01	5.11
121.99	88.55	62.82	53.08	45.44	36.47	27.84	18.76	11.3	5.79
144.32	81.51	76.02	61.62	49.24	39.26	28.76	19.43	11.71	6.16
132.52	124.57	93.28	72.84	56.3	42.31	30.46	20.03	12.19	6.93
156.25	151.25	112.55	86.03	63.53	46.15	31.98	20.9	12.83	7.42
205.48	140.91	125.89	93.93	69.31	49.11	33.66	21.71	13.44	8.25
224.36	168.19	126.15	98.24	72.41	51.53	34.83	22.53	14.17	8.83
247.85	187.37	136.48	103.11	75.69	53.18	36	23.27	14.81	9.7
276.83	190.36	155.13	113.9	80.38	55.56	37.07	24.04	15.58	10.35
262.94	235.83	175.75	127.11	87.58	58.54	38.45	24.89	16.28	11.26
296.18	265.08	197.02	141.43	95.25	61.91	39.92	25.77	17.13	11.99
361.33	262.93	213.12	151.28	101.34	64.79	41.32	26.69	17.89	12.92
387.66	297.2	219.4	158.25	105.38	66.97	42.44	27.51	18.76	13.71
418.33	321.96	235.42	166.58	109.47	68.62	43.39	28.33	19.54	14.68
453.05	333.56	258.4	179.84	115.01	70.69	44.36	29.1	20.42	15.52
448.79	380.81	282.05	194.83	122.53	73.4	45.49	29.99	21.24	16.51
486.07	412.48	305.25	210.43	130.51	76.34	46.79	30.87	22.18	17.41
557.1	418.66	324.44	222.26	137	79	47.99	31.83	23.06	18.44
589.98	457.66	336.26	231.97	141.86	80.95	49.03	32.69	24.03	19.38
626.77	487.16	356.95	243.36	146.81	82.5	49.88	33.57	24.93	20.44
666.62	506.17	383.15	258.8	153.05	84.31	50.78	34.4	25.91	21.42
673.34	554.45	409.36	275.24	160.91	86.68	51.76	35.34	26.84	22.52
714.35	588.52	434.37	292.08	169.1	89.24	52.94	36.26	27.88	23.52
786.98	602.49	456.56	305.79	176.02	91.6	54.02	37.27	28.85	24.66
825.46	644.75	473.36	318.25	181.62	93.36	55.02	38.19	29.92	25.69
867.33	678.19	497.7	332.05	187.4	94.76	55.84	39.16	30.9	26.87
911.46	703.56	526.2	349.29	194.22	96.33	56.7	40.03	32	27.91
928.94	752.06	554.44	366.97	202.39	98.34	57.61	41.05	32.98	29.13
973.18	788.28	581.06	384.77	210.67	100.54	58.7	41.99	34.14	30.18
1045.04	809.3	605.92	400.18	218.04	102.55	59.71	43.08	35.15	31.45
1088.14	853.68	627	415.11	224.28	104.14	60.69	44.02	36.34	32.5
1134.09	890.27	654.18	430.79	230.96	105.34	61.5	45.09	37.34	33.81
1181.57	921.04	684.16	449.4	238.08	106.73	62.36	45.99	38.57	34.85
1209.1	968.92	713.92	467.99	246.51	108.34	63.21	47.11	39.56	36.22
1255.92	1006.88	741.97	486.54	254.78	110.22	64.26	48.04	40.86	37.24
1325.33	1034.18	768.91	503.37	262.61	111.83	65.21	49.25	41.83	38.68
1372.05	1079.68	793.57	520.4	269.35	113.3	66.23	50.16	43.2	39.66
1421.11	1118.62	822.88	537.44	276.88	114.27	67.03	51.39	44.12	41.18
1471.02	1153.95	853.59	557.02	284.09	115.57	67.98	52.25	45.57	42.09
1507.82	1200.42	884.37	576.05	292.85	116.73	68.81	53.57	46.44	43.72
1556.58	1239.9	913.62	595.51	300.8	118.45	69.94	54.44	48.01	44.52
1622.39	1272.61	942.51	613.13	309.13	119.64	70.94	55.95	48.81	46.31
1671.98	1318.47	969.64	631.93	316.21	121.32	72.22	56.89	50.61	46.91
1723.5	1358.98	1000.67	649.92	324.62	122.21	73.46	58.71	51.31	49
1774.76	1398.19	1031.52	670.73	332.33	124.31	74.93	59.9	53.54	49.11
1819.46	1441.88	1063.73	690.65	342.32	125.49	76.78	62.03	53.98	51.76
1866.69	1482.84	1094.25	713.3	351.89	128.24	77.06	63.02	56.48	50.42
1922.62	1518.21	1128.25	733.75	362.41	128.65	75.86	60.98	55.22	53.74
1961.82	1565.05	1160.25	758.03	368.42	123.51	74.14	54.86	50.62	49.18
2000.99	1611.18	1208.97	765.89	362.7	111.89	55.01	53.99	43.07	41.87
2038.58	1728.47	1204.43	754.49	341.71	71.45	18.19	3.93	51.96	60.66
2444.67	1651.7	1167.19	720.73	300.39	11.88	-18.94	-17.33	-13.28	99.54
35	31	25	15	0	0	0	0	0	0

Figure 4.5: Stresses in each element and cracked elements after initial analysis

4.3.2 First Cracked Analysis

As it is seen, a total number of 106 elements have cracked in the first round of analysis, having a tensile stress more than 530 psi. In the second round of analysis, all of the 106 elements will be assigned a stiffness modifier of 0.35Ig while the rest of the elements are having the same stiffness modifier of 0.7Ig. Picture below shows the elements that have been cracked in first round of analysis in red. Blue elements represent those with 0.7Ig. After re-analysis of the model with new properties, stress on deformed shape of the wall is presented below.

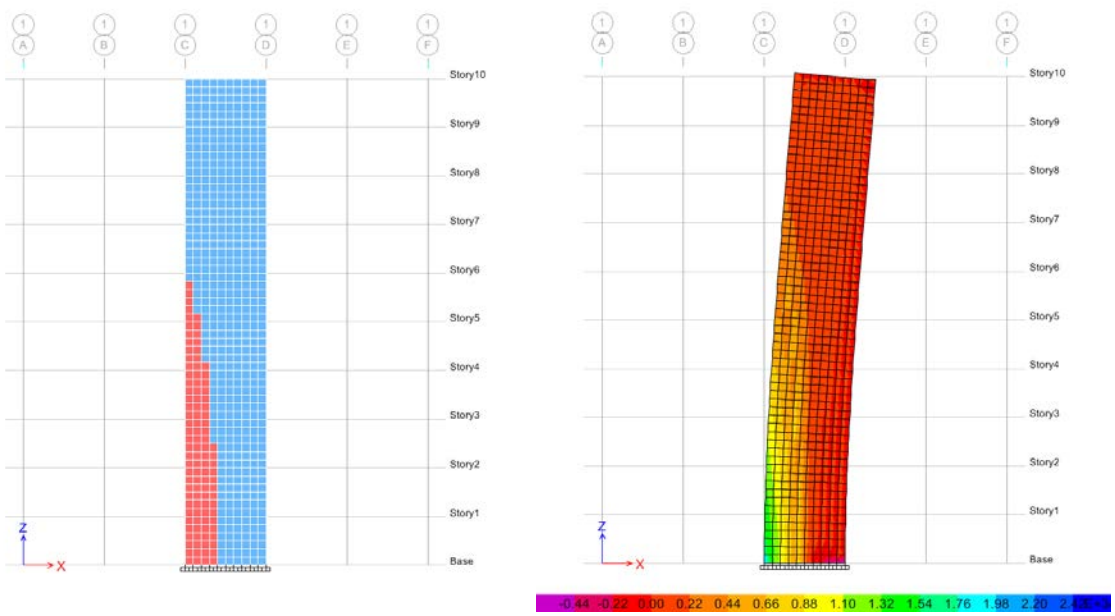


Figure 4.6: New cracked elements and stresses on deformed shape
- First cracked analysis

Table below summarizes lateral displacements in second analysis. It can be seen that lateral displacement has increased from first analysis to second as a result of cracked elements.

Table 4.3: Story displacements - first cracked analysis

Stories	Story1	Story2	Story3	Story4	Story5	Story6	Story7	Story8	Story9	Story10
Displacement (in.)	0.13	0.43	0.89	1.47	2.14	2.87	3.65	4.44	5.24	6.06

Stresses are obtained and sorted the same way it was explained in previous step. As expected, those elements which were cracked in first analysis are now taking less force and passing the load to adjacent elements which were not cracked previously. As a result, some of the elements which were not cracked in first round of analysis are now experiencing stresses higher than 530 psi. This explains why in the picture below, some of the elements right next to those cracked in first round, are now highlighted in red. It must be noted that the reason some of elements are not highlighted in red while they were cracked in the first round is because they are less stiff after cracking and do not take as much load as before, but these were cracked in the first round and their stiffness multiplier remains as 0.35Ig.

190.05	25.64	12.88	5.98	4.83	3.29	2.51	1.45	0.71	0.43
160.79	30.49	27.09	16.19	14.05	11.57	10.15	6.89	4.02	0.74
87.88	33.47	16.78	18.28	16.07	17.16	14.24	11.38	6.78	1.78
73.34	34.11	12.02	16.7	17.89	18.8	18.33	13.81	9.17	2.76
74.64	26.25	21.93	18.7	19	22.34	19.83	16.13	10	3.37
73.51	59.75	36.28	28.37	27	24.77	22.43	16.72	10.66	3.96
87.93	77.14	52.57	42.26	33.94	29.48	23.64	17.75	10.68	4.25
98.2	65.74	64.23	48.85	40.3	32.2	25.67	17.9	10.89	4.83
107.47	76.21	58.82	51.56	42.55	35.08	26.46	18.53	11.01	5.1
122.05	88.6	62.88	53.1	45.47	36.48	27.84	18.75	11.29	5.79
144.43	81.6	76.06	61.67	49.25	39.27	28.74	19.41	11.69	6.15
132.62	124.69	93.37	72.87	56.31	42.28	30.42	19.99	12.15	6.92
156.52	151.46	112.64	86.06	63.49	46.08	31.9	20.83	12.78	7.38
205.92	141.16	125.99	93.89	69.19	48.94	33.51	21.59	13.36	8.21
224.96	168.5	126.16	98.08	72.13	51.23	34.56	22.34	14.06	8.76
248.62	187.64	136.46	102.86	75.17	52.65	35.56	22.97	14.64	9.62
277.67	190.53	154.85	113.51	79.49	54.72	36.39	23.61	15.37	10.24
263.67	235.63	175.15	125.83	86.24	57.3	37.5	24.31	16.03	11.15
296.18	263.69	195.03	139.41	93.38	60.32	38.76	25.11	16.9	11.91
359.98	259.57	209.53	148.35	99.17	63.15	40.19	26.13	17.8	12.94
382.86	291.62	215.11	156.49	103.8	66.13	41.98	27.41	18.99	13.92
406.69	311.21	230.68	166.81	111.74	70.31	44.48	29.17	20.34	15.17
426.56	316.36	255.52	186.15	123.28	76.69	48.02	31.37	21.99	16.46
392.04	355.85	289.63	213.96	139.03	84.87	52.23	34	23.83	17.94
391.06	387.59	341.34	238.44	154.43	92.45	56.63	36.79	25.81	19.51
406.41	526.4	360.62	257.83	163.92	98.83	60.59	39.58	27.84	21.17
378.5	572.77	395.31	270.56	173.72	104.67	64.4	42.2	29.84	22.85
396.02	585.4	415.61	296.08	192.17	111.82	67.82	44.64	31.69	24.57
465.24	553.87	426.75	353.5	212.26	119.67	70.88	46.55	33.38	26.14
449.48	449.91	626.49	376.2	235.15	125.22	72.83	47.73	34.61	27.81
499.9	439.13	645.02	427.47	248.37	129.6	72.48	48	35.77	29.16
565.2	436.9	664.48	452.69	265.68	132.41	71.22	48.13	36.62	30.86
594.98	468.26	682.26	467.56	277.09	134.56	71.49	48.72	38.07	32.3
639.21	497.03	702.34	473.37	288	143.94	74.77	50.83	39.72	34.29
668.16	544.58	701.14	461.57	340.13	157.88	80.67	53.78	42.04	36
685.03	577.94	528.01	676.43	353.3	173.84	85.33	57	44.11	38.1
719.78	628.92	506.12	696.56	398.17	178.31	88.51	58.95	46.13	39.72
798.09	641.87	514.87	712.75	424.02	187.78	88.79	60.12	47.36	41.52
837.84	677.47	527.52	737.54	441.38	195.33	88.6	60.22	48.47	42.79
875.67	708.44	548.87	765.41	458.53	200.04	88.39	60.08	48.95	44.23
915.92	733.87	575.03	797.12	474.11	204.46	87.08	59.1	49.38	45.35
938.31	777.63	600.82	827.13	489.21	204.7	85.12	57.52	49.82	46.86
982.99	813.04	627.8	852.13	498.11	205.9	80.56	57.53	51.05	48.4
1045.08	842.9	658.48	868.79	495.16	198.3	82.53	60.06	53.3	50.72
1085.37	882.2	712.78	854.54	461.53	239.74	91.64	65.42	56.8	52.85
1125.58	925.01	730.87	621.18	676.65	244.28	102.9	70.81	59.92	55.66
1172.55	960.22	775.7	585.36	695.34	273.87	104.48	73.89	62.83	57.47
1207.59	1007.66	797.41	589.78	706.15	289.48	105.52	74.97	64.06	59.89
1256.66	1042.78	820.31	603.64	724.5	300.68	105	74.64	65.64	61
1316.79	1070.5	843.96	619.08	745.6	309.34	106.62	75.69	66.22	63.32
1357.94	1110.05	867.33	636.56	767.3	318.6	108.12	76.57	68.32	64.22
1399.27	1143.87	894.76	654.5	791.73	326.61	110.33	78.77	69.37	67.04
1439.12	1175.67	922.14	675.31	816.12	337.37	112.49	80.74	72.45	67.65
1471.14	1211.33	950.72	695.61	844.46	347.75	116.71	84.21	73.77	71.36
1504.84	1243.56	977.64	719.03	872.92	361.54	119.68	87.57	77.83	70.66
1544.85	1270.07	1007.7	741.3	905.7	373.32	124.19	88.52	78.57	75.44
1569.28	1307.41	1036.88	767.85	933.85	383.05	124.48	87.19	76.23	71.45
1596.09	1346.69	1079.57	780.01	954.8	385.88	108.79	87.4	72.88	64.07
1630.62	1446.64	1077.78	776.05	954.6	372.42	75.79	37.26	82.08	87.35
1988.85	1383.74	1046.68	747.32	953.09	325.48	-6.79	-19.77	-14.56	130.16

190.05	25.64	12.88	5.98	4.83	3.29	2.51	1.45	0.71	0.43
160.79	30.49	27.09	16.19	14.05	11.57	10.15	6.89	4.02	0.74
87.88	33.47	16.78	18.28	16.07	17.16	14.24	11.38	6.78	1.78
73.34	34.11	12.02	16.7	17.89	18.8	18.33	13.81	9.17	2.76
74.64	26.25	21.93	18.7	19	22.34	19.83	16.13	10	3.37
73.51	59.75	36.28	28.37	27	24.77	22.43	16.72	10.66	3.96
87.93	77.14	52.57	42.26	33.94	29.48	23.64	17.75	10.68	4.25
98.2	65.74	64.23	48.85	40.3	32.2	25.67	17.9	10.89	4.83
107.47	76.21	58.82	51.56	42.55	35.08	26.46	18.53	11.01	5.1
122.05	88.6	62.88	53.1	45.47	36.48	27.84	18.75	11.29	5.79
144.43	81.6	76.06	61.67	49.25	39.27	28.74	19.41	11.69	6.15
132.62	124.69	93.37	72.87	56.31	42.28	30.42	19.99	12.15	6.92
156.52	151.46	112.64	86.06	63.49	46.08	31.9	20.83	12.78	7.38
205.92	141.16	125.99	93.89	69.19	48.94	33.51	21.59	13.36	8.21
224.96	168.5	126.16	98.08	72.13	51.23	34.56	22.34	14.06	8.76
248.62	187.64	136.46	102.86	75.17	52.65	35.56	22.97	14.64	9.62
277.67	190.53	154.85	113.51	79.49	54.72	36.39	23.61	15.37	10.24
263.67	235.63	175.15	125.83	86.24	57.3	37.5	24.31	16.03	11.15
296.18	263.69	195.03	139.41	93.38	60.32	38.76	25.11	16.9	11.91
359.98	259.57	209.53	148.35	99.17	63.15	40.19	26.13	17.8	12.94
382.86	291.62	215.11	156.49	103.8	66.13	41.98	27.41	18.99	13.92
406.69	311.21	230.68	166.81	111.74	70.31	44.48	29.17	20.34	15.17
426.56	316.36	255.52	186.15	123.28	76.69	48.02	31.37	21.99	16.46
392.04	355.85	289.63	213.96	139.03	84.87	52.23	34	23.83	17.94
391.06	387.59	341.34	238.44	154.43	92.45	56.63	36.79	25.81	19.51
406.41	526.4	360.62	257.83	163.92	98.83	60.59	39.58	27.84	21.17
378.5	572.77	395.31	270.56	173.72	104.67	64.4	42.2	29.84	22.85
396.02	585.4	415.61	296.08	192.17	111.82	67.82	44.64	31.69	24.57
465.24	553.87	426.75	353.5	212.26	119.67	70.88	46.55	33.38	26.14
449.48	449.91	626.49	376.2	235.15	125.22	72.83	47.73	34.61	27.81
499.9	439.13	645.02	427.47	248.37	129.6	72.48	48	35.77	29.16
565.2	436.9	664.48	452.69	265.68	132.41	71.22	48.13	36.62	30.86
594.98	468.26	682.26	467.56	277.09	134.56	71.49	48.72	38.07	32.3
639.21	497.03	702.34	473.37	288	143.94	74.77	50.83	39.72	34.29
668.16	544.58	701.14	461.57	340.13	157.88	80.67	53.78	42.04	36
685.03	577.94	528.01	676.43	353.3	173.84	85.33	57	44.11	38.1
719.78	628.92	506.12	696.56	398.17	178.31	88.51	58.95	46.13	39.72
798.09	641.87	514.87	712.75	424.02	187.78	88.79	60.12	47.36	41.52
837.84	677.47	527.52	737.54	441.38	195.33	88.6	60.22	48.47	42.79
875.67	708.44	548.87	765.41	458.53	200.04	88.39	60.08	48.95	44.23
915.92	733.87	575.03	797.12	474.11	204.46	87.08	59.1	49.38	45.35
938.31	777.63	600.82	827.13	489.21	204.7	85.12	57.52	49.82	46.86
982.99	813.04	627.8	852.13	498.11	205.9	80.56	57.53	51.05	48.4
1045.08	842.9	658.48	868.79	495.16	198.3	82.53	60.06	53.3	50.72
1085.37	882.2	712.78	854.54	461.53	239.74	91.64	65.42	56.8	52.85
1125.58	925.01	730.87	621.18	676.65	244.28	102.9	70.81	59.92	55.66
1172.55	960.22	775.7	585.36	695.34	273.87	104.48	73.89	62.83	57.47
1207.59	1007.66	797.41	589.78	706.15	289.48	105.52	74.97	64.06	59.89
1256.66	1042.78	820.31	603.64	724.5	300.68	105	74.64	65.64	61
1316.79	1070.5	843.96	619.08	745.6	309.34	106.62	75.69	66.22	63.32
1357.94	1110.05	867.33	636.56	767.3	318.6	108.12	76.57	68.32	64.22
1399.27	1143.87	894.76	654.5	791.73	326.61	110.33	78.77	69.37	67.04
1439.12	1175.67	922.14	675.31	816.12	337.37	112.49	80.74	72.45	67.65
1471.14	1211.33	950.72	695.61	844.46	347.75	116.71	84.21	73.77	71.36
1504.84	1243.56	977.64	719.03	872.92	361.54	119.68	87.57	77.83	70.66
1544.85	1270.07	1007.7	741.3	905.7	373.32	124.19	88.52	78.57	75.44
1569.28	1307.41	1036.88	767.85	933.85	383.05	124.48	87.19	76.23	71.45
1596.09	1346.69	1079.57	780.01	954.8	385.88	108.79	87.4	72.88	64.07
1630.62	1446.64	1077.78	776.05	954.6	372.42	75.79	37.26	82.08	87.35
1988.85	1383.74	1046.68	747.32	953.09	325.48	-6.79	-19.77	-14.56	130.16
29	29	27	25	15	0	0	0	0	0

Figure 4.7: Stresses in each element and cracked elements after first cracked analysis

4.3.3 Second Cracked Analysis

By looking at the results from first cracked analysis, 34 new elements that were not cracked before are showing tensile stresses higher than 530 psi. This number is much smaller than 106 elements that cracked in the first analysis as expected. The same procedure is used for these elements and a stiffness multiplier of 0.35Ig is applied to newly cracked elements and analysis is repeated. Picture below displays the new elements that are cracked in orange. It clearly shows that all of these elements are next to those which were cracked in first analysis, displayed in red. Stresses on deformed shape are also provided.

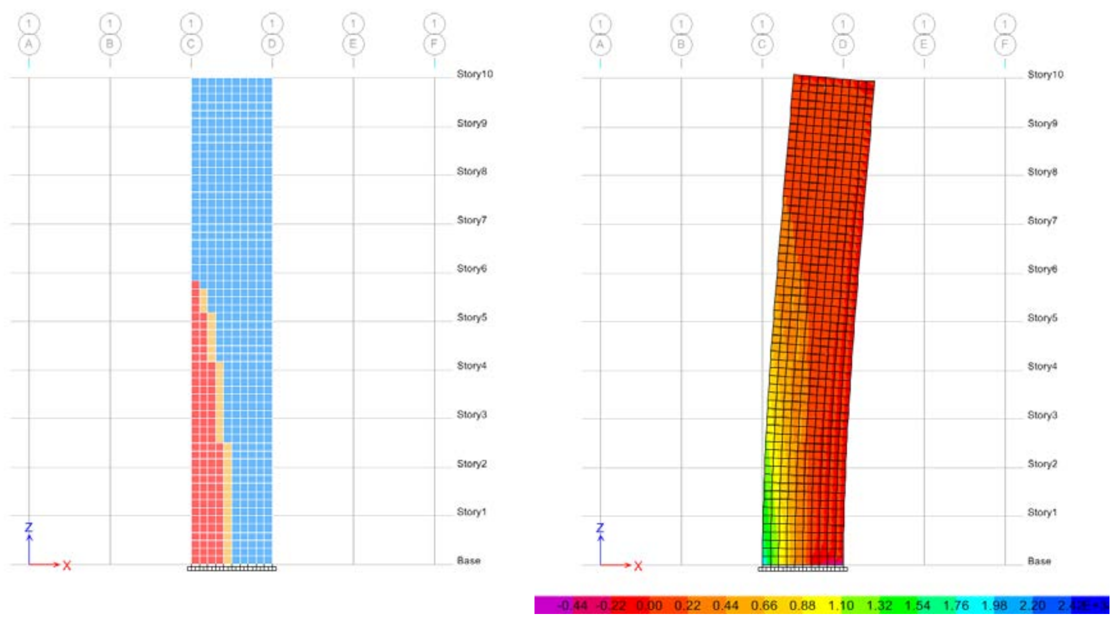


Figure 4.8: New cracked elements and stresses on deformed shape - second cracked analysis

As expected, lateral displacements keep increasing as analysis is refined. However, the increase in lateral displacement from first cracked analysis to second is not significant.

Table 4.4: Story displacements - second cracked analysis

Stories	Story1	Story2	Story3	Story4	Story5	Story6	Story7	Story8	Story9	Story10
Displacement (in.)	0.13	0.44	0.91	1.50	2.18	2.93	3.73	4.54	5.37	6.20

Analysis should continue until there are no more new cracked elements and it can be checked by reviewing stresses in each element after analysis is complete. The data tables below, show there are still a few elements that cracked after second cracked analysis. These, as expected, are adjacent to the elements that were cracked in previous analysis. But by comparing the number of new cracked elements at each step, it can be concluded that the analysis is reaching the convergence expected. From uncracked wall to first cracked analysis, 106 elements were cracked. From first cracked analysis to second cracked analysis, 34 elements were cracked but in the picture below, there are only 8 new elements that are highlighted in red and will crack from this analysis to next one.

190.05	25.64	12.88	5.98	4.83	3.29	2.51	1.45	0.71	0.43
160.79	30.49	27.09	16.19	14.05	11.57	10.15	6.89	4.02	0.74
87.88	33.47	16.78	18.28	16.07	17.16	14.24	11.38	6.78	1.78
73.34	34.11	12.02	16.7	17.89	18.8	18.33	13.81	9.17	2.76
74.64	26.25	21.93	18.7	19	22.34	19.83	16.13	10	3.37
73.51	59.75	36.28	28.38	27	24.77	22.43	16.72	10.66	3.96
87.93	77.14	52.57	42.26	33.94	29.48	23.64	17.74	10.68	4.25
98.2	65.74	64.23	48.86	40.3	32.2	25.67	17.9	10.89	4.83
107.48	76.21	58.84	51.57	42.55	35.08	26.46	18.53	11	5.11
122.07	88.62	62.88	53.11	45.47	36.48	27.84	18.75	11.29	5.78
144.46	81.61	76.08	61.66	49.25	39.26	28.74	19.4	11.68	6.15
132.64	124.72	93.37	72.88	56.3	42.27	30.41	19.98	12.15	6.91
156.59	151.5	112.66	86.05	63.47	46.06	31.88	20.82	12.77	7.38
206	141.21	125.98	93.86	69.14	48.9	33.48	21.56	13.34	8.2
225.06	168.52	126.14	98.01	72.05	51.15	34.5	22.3	14.03	8.75
248.72	187.66	136.4	102.83	75.08	52.53	35.47	22.91	14.61	9.6
277.74	190.47	154.78	113.38	79.3	54.54	36.25	23.52	15.33	10.22
263.65	235.48	174.94	125.57	85.97	57.04	37.32	24.19	15.99	11.14
295.86	263.2	194.58	139.01	93.03	60.02	38.49	24.96	16.86	11.89
359.17	258.71	208.78	147.82	98.75	62.75	39.91	25.96	17.77	12.98
381.23	290.27	214.32	156	103.71	65.75	41.65	27.29	19.06	13.98
403.75	309.24	229.44	166.46	111.54	69.9	44.39	29.28	20.56	15.39
422.23	313.68	253.74	185.02	122.54	76.92	48.65	32.03	22.58	16.77
387.63	352.41	284.33	209.77	139.41	87.42	54.44	35.49	24.78	18.58
381.5	379.44	321.68	234.76	162.37	98.59	60.49	39.13	27.31	20.28
459.34	472.64	320.6	281.75	176.72	107.11	65.31	42.72	29.68	22.31
420.73	388.54	493.67	288.82	188.11	110.28	69.73	45.73	32.09	24.14
478.84	376.91	513.66	324.33	196.16	119.64	73.14	48.52	34.12	26.1
507.14	401.39	514.25	326.73	239.31	128.33	76.96	50.51	35.91	27.81
507.68	435.35	391.4	487.23	257.09	140.2	78.55	51.71	37.07	29.38
538.34	478.7	381.64	510.23	292.23	144.28	78.61	51.21	37.63	30.77
614.27	485.97	392.35	527.3	312.15	149.84	76.57	49.43	37.94	32.02
652.21	522.15	404.07	545.73	321.8	152.5	73.31	48.44	38.4	33.45
686.14	556.15	427.67	563.56	324.39	147.07	72.38	49.25	39.8	35.07
722.97	574.88	474.68	570.9	300.48	173.54	78.67	52.67	42.02	36.96
732.06	624.34	496.34	424.29	444.73	181.8	87.39	56.45	44.54	38.99
772.62	657.66	530.1	404.35	468.47	199.76	89.87	59.26	46.73	40.85
841.16	679.76	548.43	410.26	482.69	212.6	90.1	60.51	48.26	42.63
882.58	719.89	564.23	421.87	499.05	221.59	92.12	61.02	49.49	44.07
924.27	752.48	587.86	436.14	517.47	228.55	93.56	61.79	50.45	45.58
966.01	779.55	615.54	454.9	536.81	234.83	94.3	62.32	51.35	46.64
987.64	823.58	644.31	474.5	557.39	239.79	94.01	61.73	51.3	48.09
1027.42	858.29	672.09	495.71	575.76	241.96	90.67	58.35	51.96	49.13
1088.59	881.52	701.81	519	592.69	237.47	84.3	58.86	52.95	51.23
1126.5	923.35	723.64	556.41	596.02	204.14	88.44	62.74	56.23	53.11
1167.68	959.17	755.77	568.68	430.33	297.65	100.03	69.77	59.49	55.99
1211.86	991.77	788.73	593.38	401.05	321.71	101.84	73.49	62.81	57.83
1244.23	1035.55	819.59	606.51	404.57	329.67	105.77	74.15	64.14	60.33
1288.61	1071.8	845.57	623.61	413.59	339.03	106.13	74.29	65.72	61.45
1348.78	1100.08	871.46	640.11	424.24	347.94	107.81	75.73	66.38	63.83
1390.87	1140.64	895.65	658.96	434.75	358.33	109.46	76.71	68.62	64.77
1433.22	1175.25	923.94	677.55	447.54	367.45	111.56	78.91	69.74	67.68
1473.92	1207.82	951.95	699.32	459.54	379.71	113.66	80.86	72.83	68.41
1506.66	1244.24	981.13	720.35	474.74	391.72	117.88	84.27	74.32	72.27
1541.29	1277.27	1008.43	744.03	489.27	407.55	121.23	88.06	78.64	71.81
1583.03	1304.75	1038.69	765.81	505.68	424.4	127.11	89.87	79.88	76.98
1609.89	1343.63	1068.02	790.98	516.77	439.19	129.79	89.82	78.43	73.35
1639.92	1384.43	1111.31	800.62	519.3	453.09	114.97	90.86	76.35	66.53
1677.3	1488	1108.55	794.29	504.7	443.29	85.83	42.41	85.52	90.36
2047.14	1423.5	1076.35	764.87	478.82	415.84	2.7	-20.54	-15.35	132.8

190.05	25.64	12.88	5.98	4.83	3.29	2.51	1.45	0.71	0.43
160.79	30.49	27.09	16.19	14.05	11.57	10.15	6.89	4.02	0.74
87.88	33.47	16.78	18.28	16.07	17.16	14.24	11.38	6.78	1.78
73.34	34.11	12.02	16.7	17.89	18.8	18.33	13.81	9.17	2.76
74.64	26.25	21.93	18.7	19	22.34	19.83	16.13	10	3.37
73.51	59.75	36.28	28.38	27	24.77	22.43	16.72	10.66	3.96
87.93	77.14	52.57	42.26	33.94	29.48	23.64	17.74	10.68	4.25
98.2	65.74	64.23	48.86	40.3	32.2	25.67	17.9	10.89	4.83
107.48	76.21	58.84	51.57	42.55	35.08	26.46	18.53	11	5.11
122.07	88.62	62.88	53.11	45.47	36.48	27.84	18.75	11.29	5.78
144.46	81.61	76.08	61.66	49.25	39.26	28.74	19.4	11.68	6.15
132.64	124.72	93.37	72.88	56.3	42.27	30.41	19.98	12.15	6.91
156.59	151.5	112.66	86.05	63.47	46.06	31.88	20.82	12.77	7.38
206	141.21	125.98	93.86	69.14	48.9	33.48	21.56	13.34	8.2
225.06	168.52	126.14	98.01	72.05	51.15	34.5	22.3	14.03	8.75
248.72	187.66	136.4	102.83	75.08	52.53	35.47	22.91	14.61	9.6
277.74	190.47	154.78	113.38	79.3	54.54	36.25	23.52	15.33	10.22
263.65	235.48	174.94	125.57	85.97	57.04	37.32	24.19	15.99	11.14
295.86	263.2	194.58	139.01	93.03	60.02	38.49	24.96	16.86	11.89
359.17	258.71	208.78	147.82	98.75	62.75	39.91	25.96	17.77	12.98
381.23	290.27	214.32	156	103.71	65.75	41.65	27.29	19.06	13.98
403.75	309.24	229.44	166.46	111.54	69.9	44.39	29.28	20.56	15.39
422.23	313.68	253.74	185.02	122.54	76.92	48.65	32.03	22.58	16.77
387.63	352.41	284.33	209.77	139.41	87.42	54.44	35.49	24.78	18.58
381.5	379.44	321.68	234.76	162.37	98.59	60.49	39.13	27.31	20.28
459.34	472.64	320.6	281.75	176.72	107.11	65.31	42.72	29.68	22.31
420.73	388.54	493.67	288.82	188.11	110.28	69.73	45.73	32.09	24.14
478.84	376.91	513.66	324.33	196.16	119.64	73.14	48.52	34.12	26.1
507.14	401.39	514.25	326.73	239.31	128.33	76.96	50.51	35.91	27.81
507.68	435.35	391.4	487.23	257.09	140.2	78.55	51.71	37.07	29.38
538.34	478.7	381.64	510.23	292.23	144.28	78.61	51.21	37.63	30.77
614.27	485.97	392.35	527.3	312.15	149.84	76.57	49.43	37.94	32.02
652.21	522.15	404.07	545.73	321.8	152.5	73.31	48.44	38.4	33.45
686.14	556.15	427.67	563.56	324.39	147.07	72.38	49.25	39.8	35.07
722.97	574.88	474.68	570.9	300.48	173.54	78.67	52.67	42.02	36.96
732.06	624.34	496.34	424.29	444.73	181.8	87.39	56.45	44.54	38.99
772.62	657.66	530.1	404.35	468.47	199.76	89.87	59.26	46.73	40.85
841.16	679.76	548.43	410.26	482.69	212.6	90.1	60.51	48.26	42.63
882.58	719.89	564.23	421.87	499.05	221.59	92.12	61.02	49.49	44.07
924.27	752.48	587.86	436.14	517.47	228.55	93.56	61.79	50.45	45.58
966.01	779.55	615.54	454.9	536.81	234.83	94.3	62.32	51.35	46.64
987.64	823.58	644.31	474.5	557.39	239.79	94.01	61.73	51.3	48.09
1027.42	858.29	672.09	495.71	575.76	241.96	90.67	58.35	51.96	49.13
1088.59	881.52	701.81	519	592.69	237.47	84.3	58.86	52.95	51.23
1126.5	923.35	723.64	556.41	596.02	204.14	88.44	62.74	56.23	53.11
1167.68	959.17	755.77	568.68	430.33	297.65	100.03	69.77	59.49	55.99
1211.86	991.77	788.73	593.38	401.05	321.71	101.84	73.49	62.81	57.83
1244.23	1035.55	819.59	606.51	404.57	329.67	105.77	74.15	64.14	60.33
1288.61	1071.8	845.57	623.61	413.59	339.03	106.13	74.29	65.72	61.45
1348.78	1100.08	871.46	640.11	424.24	347.94	107.81	75.73	66.38	63.83
1390.87	1140.64	895.65	658.96	434.75	358.33	109.46	76.71	68.62	64.77
1433.22	1175.25	923.94	677.55	447.54	367.45	111.56	78.91	69.74	67.68
1473.92	1207.82	951.95	699.32	459.54	379.71	113.66	80.86	72.83	68.41
1506.66	1244.24	981.13	720.35	474.74	391.72	117.88	84.27	74.32	72.27
1541.29	1277.27	1008.43	744.03	489.27	407.55	121.23	88.06	78.64	71.81
1583.03	1304.75	1038.69	765.81	505.68	424.4	127.11	89.87	79.88	76.98
1609.89	1343.63	1068.02	790.98	516.77	439.19	129.79	89.82	78.43	73.35
1639.92	1384.43	1111.31	800.62	519.3	453.09	114.97	90.86	76.35	66.53
1677.3	1488	1108.55	794.29	504.7	443.29	85.83	42.41	85.52	90.36
2047.14	1423.5	1076.35	764.87	478.82	415.84	2.7	-20.54	-15.35	132.8
30	27	24	19	5	0	0	0	0	0

Figure 4.9: Stresses in each element and cracked elements after second cracked analysis

4.3.4 Final Cracked Analysis

Significantly lower number of elements which were cracked in previous analysis indicate that this step can be assumed to be the final cracked analysis and there will be no new cracked elements after this step. In order to verify this assumption, the same procedure is followed. Eight new elements that have been cracked are displayed in yellow, right next to those that were cracked in second analysis. Stresses on deformed shape are also displayed after analysis is performed.

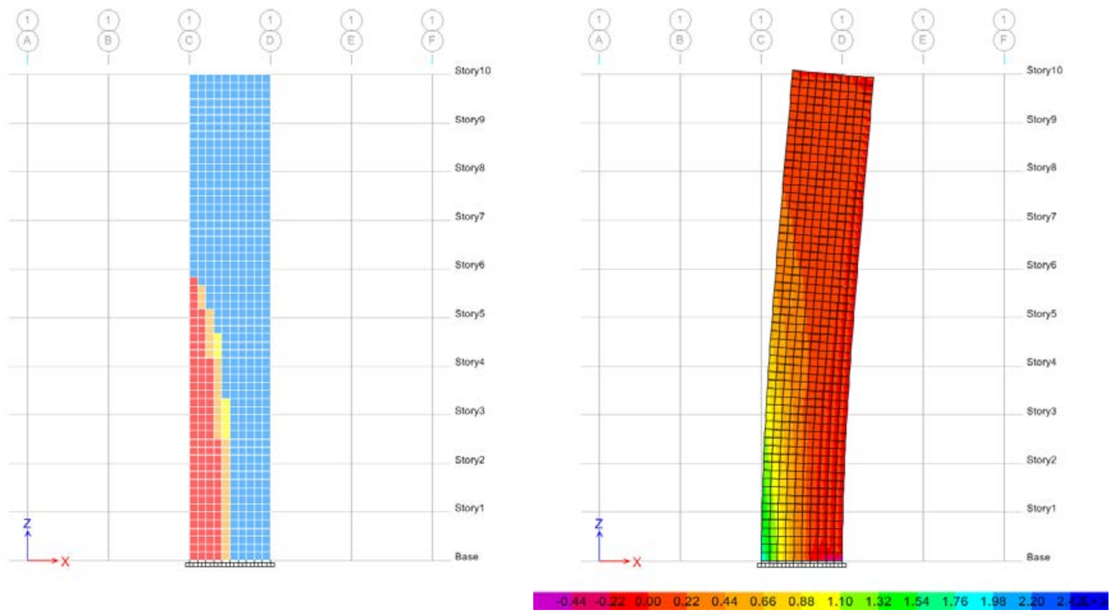


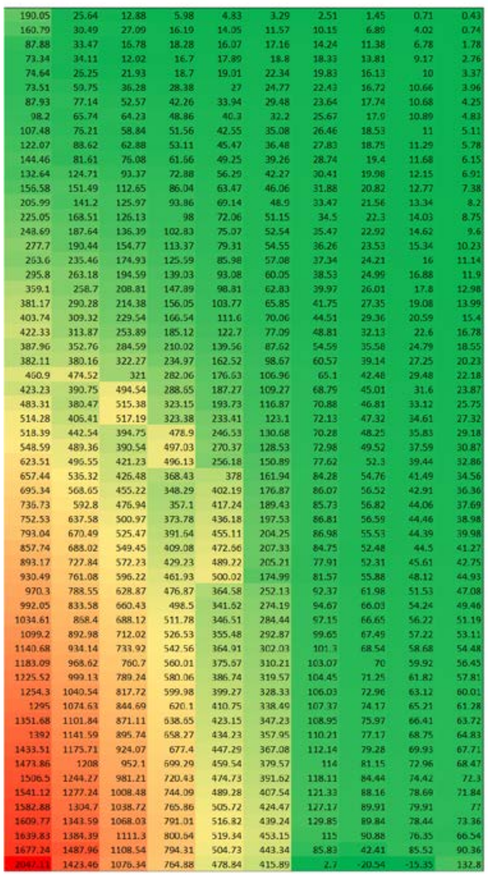
Figure 4.10: New cracked elements and stresses on deformed shape - final cracked analysis

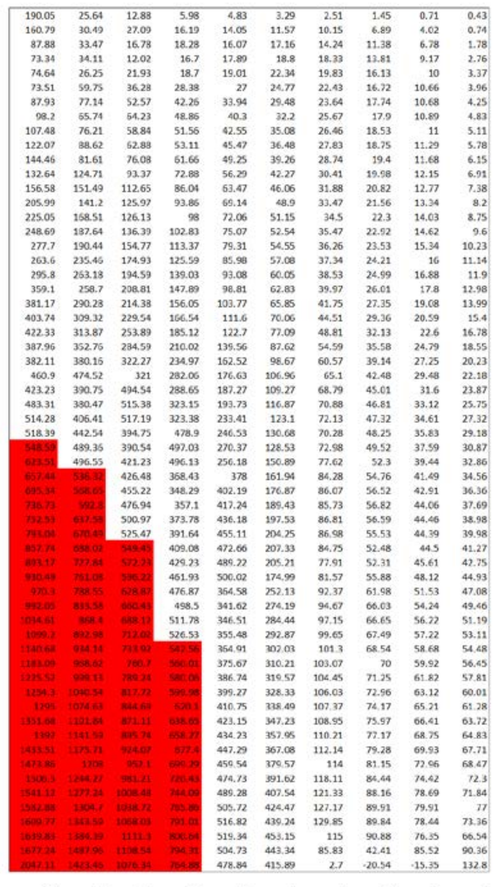
By comparing the results of lateral displacement to second cracked analysis, it is seen that the difference is very small and negligible.

Table 4.5: Story displacements - final cracked analysis

Stories	Story1	Story2	Story3	Story4	Story5	Story6	Story7	Story8	Story9	Story10
Displacement (in.)	0.13	0.44	0.91	1.50	2.19	2.94	3.74	4.55	5.38	6.22

As it was expected, figures below do not show any new cracked element and at this point it can be verified that this will be the final cracked analysis which is required. By looking at the stresses, it is obvious that previous cracked elements have increased the stress in some of the adjacent elements (yellow spots next to cracked elements), but none of the increased stresses are over 530 psi and therefore none of them will crack in this step of the analysis. As a result, it can also be concluded that the lateral displacement in this step will be the final lateral displacement of the shear wall. However the difference is so small compared to the previous step that it can easily be ignored.





190.05	25.64	12.88	5.98	4.83	3.29	2.51	1.45	0.71	0.43
160.79	30.49	27.09	16.19	14.05	11.57	10.15	6.89	4.02	0.74
87.88	33.47	16.78	18.28	16.07	17.16	14.24	11.38	6.78	1.78
73.34	34.11	12.02	16.7	17.89	18.8	18.33	13.81	9.17	2.76
74.64	26.25	21.93	18.7	19.01	22.34	19.83	16.13	10	3.37
73.51	59.75	36.28	28.38	27	24.77	22.43	16.72	10.66	3.96
87.93	77.14	52.57	42.26	33.94	29.48	23.64	17.74	10.68	4.25
98.2	65.74	64.23	48.86	40.3	32.2	25.67	17.9	10.89	4.83
107.48	76.21	58.84	51.56	42.55	35.08	26.46	18.53	11	5.11
122.07	88.62	62.88	53.11	45.47	36.48	27.83	18.75	11.29	5.78
144.46	81.61	76.08	61.66	49.25	39.26	28.74	19.4	11.68	6.15
132.64	124.71	93.37	72.88	56.29	42.27	30.41	19.98	12.15	6.91
156.58	151.49	112.65	86.04	63.47	46.06	31.88	20.82	12.77	7.38
205.99	141.2	125.97	93.86	69.14	48.9	33.47	21.56	13.34	8.2
225.05	168.51	126.13	98	72.06	51.15	34.5	22.3	14.03	8.75
248.69	187.64	136.39	102.83	75.07	52.54	35.47	22.92	14.62	9.6
277.7	190.44	154.77	113.37	79.31	54.55	36.26	23.53	15.34	10.23
263.6	235.46	174.93	125.59	85.98	57.08	37.34	24.21	16	11.14
295.8	263.18	194.59	139.03	93.08	60.05	38.53	24.99	16.88	11.9
359.1	258.7	208.81	147.89	98.81	62.83	39.97	26.01	17.8	12.98
381.17	290.28	214.38	156.05	103.77	65.85	41.75	27.35	19.08	13.99
403.74	309.32	229.54	166.54	111.6	70.06	44.51	29.36	20.59	15.4
422.33	313.87	253.89	185.12	122.7	77.09	48.81	32.13	22.6	16.78
387.96	352.76	284.59	210.02	139.56	87.62	54.59	35.58	24.79	18.55
382.11	380.16	322.27	234.97	162.52	98.67	60.57	39.14	27.25	20.23
460.9	474.52	321	282.06	176.63	106.96	65.1	42.48	29.48	22.18
423.23	390.75	494.54	288.65	187.27	109.27	68.79	45.01	31.6	23.87
483.31	380.47	515.38	323.15	193.73	116.87	70.88	46.81	33.12	25.75
514.28	406.41	517.19	323.38	233.41	123.1	72.13	47.32	34.61	27.32
518.39	442.54	394.75	478.9	246.53	130.68	70.28	48.25	35.83	29.18
548.59	489.36	390.54	497.03	270.37	128.53	72.98	49.52	37.59	30.87
623.51	496.55	421.23	496.13	256.18	150.89	77.62	52.3	39.44	32.86
657.44	536.32	426.48	368.43	378	161.94	84.28	54.76	41.49	34.56
695.34	568.65	455.22	348.29	402.19	176.87	86.07	56.52	42.91	36.36
736.73	592.8	476.94	357.1	417.24	189.43	85.73	56.82	44.06	37.69
752.53	637.58	500.97	373.78	436.18	197.53	86.81	56.59	44.46	38.98
793.04	670.49	525.47	391.64	455.11	204.25	86.98	55.53	44.39	39.98
857.74	688.02	549.45	409.08	472.66	207.33	84.75	52.48	44.5	41.27
893.17	727.84	572.23	429.23	489.22	205.21	77.91	52.31	45.61	42.75
930.49	761.08	596.22	461.93	500.02	174.99	81.57	55.88	48.12	44.93
970.3	788.55	628.87	476.87	364.58	252.13	92.37	61.98	51.53	47.08
992.05	833.58	660.43	498.5	341.62	274.19	94.67	66.03	54.24	49.46
1034.61	868.4	688.12	511.78	346.51	284.44	97.15	66.65	56.22	51.19
1099.2	892.98	712.02	526.53	355.48	292.87	99.65	67.49	57.22	53.11
1140.68	934.14	733.92	542.56	364.91	302.03	101.3	68.54	58.68	54.48
1183.09	968.62	760.7	560.01	375.67	310.21	103.07	70	59.92	56.45
1225.52	999.13	789.24	580.06	386.74	319.57	104.45	71.25	61.82	57.81
1254.3	1040.54	817.72	599.98	399.27	328.33	106.03	72.96	63.12	60.01
1295	1074.63	844.69	620.1	410.75	338.49	107.37	74.17	65.21	61.28
1351.68	1101.84	871.11	638.65	423.15	347.23	108.95	75.97	66.41	63.72
1392	1141.59	895.74	658.27	434.23	357.95	110.21	77.17	68.75	64.83
1433.51	1175.71	924.07	677.4	447.29	367.08	112.14	79.28	69.93	67.71
1473.86	1208	952.1	699.29	459.54	379.57	114	81.15	72.96	68.47
1506.5	1244.27	981.21	720.43	474.73	391.62	118.11	84.44	74.42	72.3
1541.12	1277.24	1008.48	744.09	489.28	407.54	121.33	88.16	78.69	71.84
1582.88	1304.7	1038.72	765.86	505.72	424.47	127.17	89.91	79.91	77
1609.77	1343.59	1068.03	791.01	516.82	439.24	129.85	89.84	78.44	73.36
1639.83	1384.39	1111.3	800.64	519.34	453.15	115	90.88	76.35	66.54
1677.24	1487.96	1108.54	794.31	504.73	443.34	85.83	42.41	85.52	90.36
2047.11	1423.46	1076.34	764.88	478.84	415.89	2.7	-20.54	-15.35	132.8
30	28	23	16	0	0	0	0	0	0

Figure 4.11: Stresses in each element and cracked elements after final cracked analysis

4.4 Discussion

To summarize all the steps and better understand how displacements are influenced over the entire process, table below is provided. Note that the % column represents the increase in lateral displacement of each step compared to previous step.

Table 4.6: Lateral displacements in cracked analysis

Stories	Lateral Displacements (in.)						
	No Cracking (0.7Ig)	First Cracked Analysis	%	Second Cracked Analysis	%	Final Cracked Analysis	%
Story 1	0.09	0.13	44.4 %	0.13	0 %	0.13	0 %
Story 2	0.31	0.43	38.7 %	0.44	2.3 %	0.44	0 %
Story 3	0.64	0.89	39.1 %	0.91	2.2 %	0.91	0 %
Story 4	1.06	1.47	38.7 %	1.5	2 %	1.5	0 %
Story 5	1.55	2.14	38.1 %	2.18	1.9 %	2.19	0.5 %
Story 6	2.1	2.87	36.7 %	2.93	2.1 %	2.94	0.3 %
Story 7	2.68	3.65	36.2 %	3.73	2.2 %	3.74	0.3 %
Story 8	3.28	4.44	35.4 %	4.54	2.3 %	4.55	0.2 %
Story 9	3.89	5.24	34.7 %	5.37	2.5 %	5.38	0.2 %
Story 10	4.51	6.06	34.4 %	6.2	2.3 %	6.22	0.3 %

By reviewing the displacements in the table and the increase percentage at each step, it is seen that there is a significant difference between final cracked analysis and the analysis without considering cracked sections (1.71 in. at top floor), but there is not a great difference between final cracked analysis and first cracked analysis. In other words, the difference between maximum lateral displacement on final analysis and first analysis is only about 0.16 inches or 2.7% which is not significant and if very precise analysis is not rquired, it can be ignored. This difference is even smaller when final and second cracked analyses are compared. In this case the difference in maximum lateral displacement is only 0.02 inches or 0.3% which is negligible.

Graph below demonstrates the rate of change in all of the steps above:

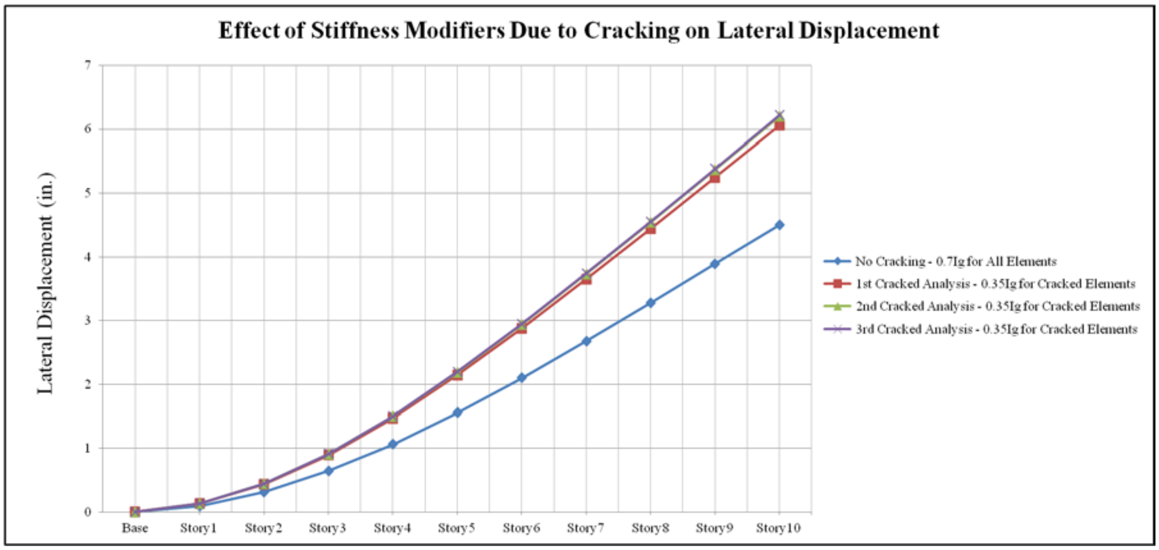


Figure 4.12: Comparison of displacements in cracked analysis models

As it is seen in the graph, the first model without cracking falls well below other analyses with a maximum displacement of about 4.5 inches. But first cracked analysis (displayed in red) is showing significant increase in lateral displacements with a maximum displacement of 6 inches. After first cracked analysis is performed, other results from second and final cracked models are so close to first analysis that the difference can hardly be seen in this graph.

Lastly, something that needs to be recognized when using this method is that most of the lateral loads such as seismic forces are considered at every direction in analysis. In other words, the same loads that were applied in positive X direction to this shear wall may be applied at negative X direction as well and as a result, cracking will occur on the other corner of the shear wall too.

To account for this phenomenon, it can be assumed that the same elements that were cracked in procedure described in this chapter will crack on the other side of the shear wall, forming a symmetrical shape of cracked elements as shown below. In this graph, all of the blue elements are those that have not been cracked and any other color describes a cracked element.

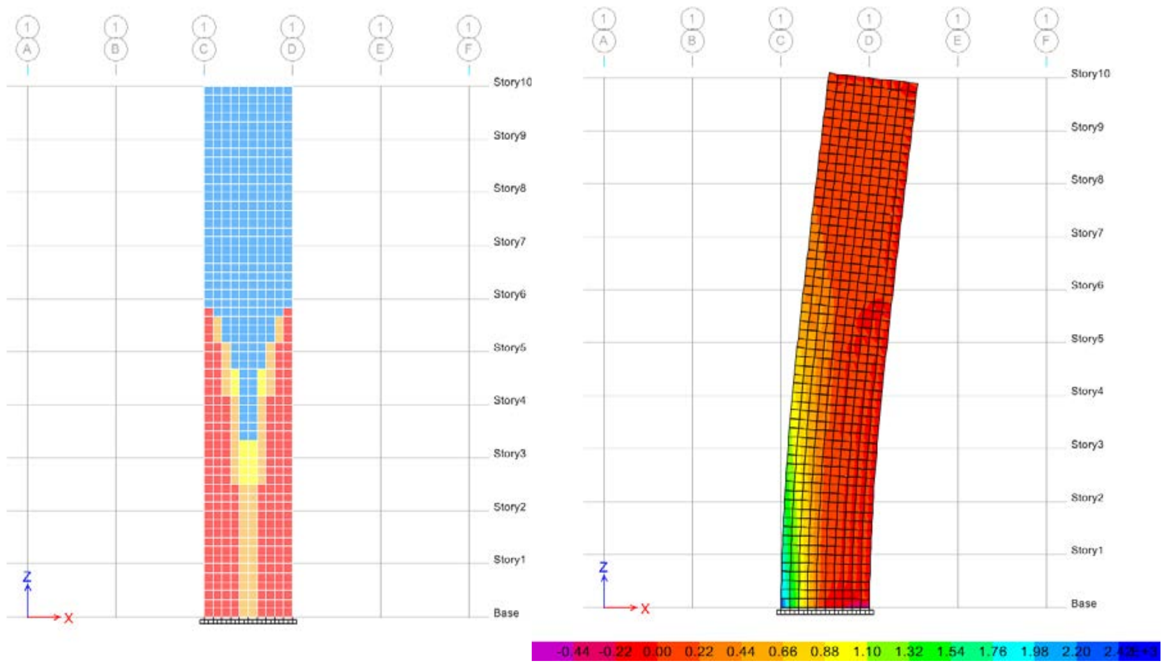


Figure 4.13: Symmetric cracking as a result of loads in both directions

It is expected to have higher lateral displacements since more elements have been cracked and the results in table below agree with the expectations.

Table 4.7: Story displacements - symmetric cracked elements

Stories	Story1	Story2	Story3	Story4	Story5	Story6	Story7	Story8	Story9	Story10
Displacement (in.)	0.18	0.62	1.28	2.12	3.10	4.16	5.27	6.41	7.55	8.70

For a better look at how analysis is affected, table below summarizes the results from three different analyses: No cracked elements, cracked elements on both directions, all cracked elements. The first two models were described in previous sections. All cracked analysis is a new model which applies cracked stiffness modifiers (0.35Ig) to all 600 elements in the wall.

Table 4.8: Lateral displacements of symmetric cracked elements and all cracked elements

Stories	Lateral Displacements (in.)				
	No Cracking (0.7Ig)	Cracked Elements on Both Sides (Symmetric Model)	%	All Cracked Elements (0.35Ig)	%
Story 1	0.09	0.18	100 %	0.18	0 %
Story 2	0.31	0.62	100 %	0.62	0 %
Story 3	0.64	1.28	100 %	1.28	0 %
Story 4	1.06	2.12	100 %	2.12	0 %
Story 5	1.55	3.1	100 %	3.11	0.3 %
Story 6	2.1	4.16	98.1 %	4.19	0.7 %
Story 7	2.68	5.27	96.6 %	5.35	1.5 %
Story 8	3.28	6.41	95.4 %	6.55	2.2 %
Story 9	3.89	7.55	94.1 %	7.77	2.9 %
Story 10	4.51	8.7	92.9 %	9.01	3.6 %

Increase percentages for each model in table above are based on increase in lateral displacements compared to its previous model. The reason why displacements in all Cracked model are 100% more than no cracking model is because 0.7Ig is 100% bigger than 0.35Ig and as a result the displacements on 0.35Ig model are also 100% bigger than displacements in 0.7Ig.

It is important to keep in mind that all of the models in this chapter are arbitrary models and are analyzed to illustrate how each of the steps along the procedure affects the results. Concepts and behavior of structures remain the same but other case studies might provide different amount of cracking and therefore, different displacement increases. Values provided in this chapter are for comparison only and the focus is mainly on behavior rather than numbers and percentages.

In the end, the model and methodology used in this chapter can be extended to other cases in real projects. The essential requirements would be the loads that are applied to the wall at each floor (both lateral loads and gravity loads). Once these information are obtained from analysis, the shear wall can be analyzed under the loads separately in order to identify which areas of shear wall will crack. When those cracking zones are identified to a certain precision, cracking modifiers can be applied to those areas in the general model. Then it is expected from the analysis to show more realistic displacements due to lateral loads because of cracked sections. It is important to know that if more powerful computers are used, these steps can be processed within the original model of the entire structure (in this case, the medium rise building discussed in chapter 3) without a need for a separate file. Also, for a conservative and quick approach, cracking modifiers can be applied to all shear wall elements and expect an upper limit for lateral displacements.

Chapter 5

Openings, Skewed Walls and Flexural Capacity

5.1 Introduction

Stiffness of shear walls can be influenced by many other parameters that at first might not seem to be crucial but it is very important for structural engineers to know how each factor can have an impact on stiffness of shear walls. In some situations, engineers might assume that ignoring the effects of a certain parameter is safe and it does not have a great impact on final results. This assumption can save time and increase efficiency in design procedures, but it is critical to know what the limit is in which ignoring a certain factor can still be considered safe. In other words, how much is too much to ignore? Some of these parameters are investigated in this chapter including

1. Openings in Shear Walls
2. Skewed Shear Walls

Since most of the studies up until this chapter were mainly about lateral stiffness of structures, it can also be helpful to cover parameters that can affect flexural capacity of shear walls. Last section of this study investigates the impact of wall thickness and vertical reinforcement on flexural strength of shear walls.

To better arrange these different studies, they are presented in multiple sections starting with “Effects of Openings on Stiffness of Shear Walls” followed by “Skewed Walls and Stiffness of structures”. “Flexural Capacity of Wall Sections” is also investigated in the end of this chapter.

5.2 Effects of Openings on Stiffness of Shear Walls

5.2.1 Introduction

There are many cases in which structural engineers need to provide openings in shear walls for different purposes. Most of the time, it is inevitable to have shear walls without any openings. These openings may be required for architectural purposes such as windows and doors, or for mechanical, electrical and plumbing reasons. When shear walls are used as a central core for example, they are built around elevator shafts or staircases, meaning that there must be openings provided to access different areas in a building. But the size of these openings is different from case to case. When there’s a small opening required for mechanical purposes, the effect that is has on stiffness of the wall is expected to be different than the effect that an opening with the size of a window or a door can have.

Structural engineers, based on their engineering judgment, may decide not to include some of the openings in their analysis since small openings might not have a remarkable impact on stiffness of a wall and it’s not worth the time to model all of them in the analysis. Detailing around these openings, on the other hand, must follow certain regulations that cannot be ignored and are addressed in ASCE and ACI. But the question that this study is trying to answer is: Which openings should be considered and which can safely be ignored? In other words, what ratio of openings compared to the size of the wall is large enough to have a significant impact on stiffness of the wall? This study, just like all other chapters in this research, is focused on stiffness of the structure and other parameters affected by the change in stiffness. Shear and flexural capacity used in design of shear walls are also influenced by openings which require further investigation and it is not covered in this section.

5.2.2 Model Description and Methodology

For this study, the medium rise building model (described in chapter 3) with central core is used. The same material properties, section properties and loadings presented in general model information are applied. The approach is to increase opening dimensions at each step and investigate the effects on different parameters at every ratio. Openings are provided at each face of the central core and at every level. Models that have been considered in this comparison are:

1. Medium rise building (Central Core) with no openings in shear walls
2. Medium rise building (Central Core) with 2ft x 2ft openings at each face in all levels
3. Medium rise building (Central Core) with 4ft x 4ft openings at each face in all levels
4. Medium rise building (Central Core) with 6ft x 6ft openings at each face in all levels
5. Medium rise building (Central Core) with 8ft x 8ft openings at each face in all levels
6. Medium rise building (Central Core) with 10ft x 10ft openings at each face in all levels

To have a better sense of how large the openings are relative to the size of the wall, a ratio of opening to wall is used. Since height of each floor is 12ft and each span in the plan is 20ft, the ratio of the opening will be

$$\frac{Area\ of\ Openings}{Area\ of\ wall}$$

So for example a 2ft x 2ft opening will have an opening ratio of:

$$\frac{2\ \times 2}{12 \times 20} = 0.167\ \rightarrow 1.67\%$$

By using the same calculation for each opening dimension, table below is obtained.

Table 5.1: Opening dimensions and ratios

Opening Dimension	**Opening Ratio**
No Opening	0%
2ft x 2ft	1.67%
4ft x 4ft	6.67%
6ft x 6ft	15%
8ft x 8ft	26.67%
10ft x 10ft	41.67%

Openings are provided at the center of each wall in each story and at all four sides of the core. The position of these openings stay the same in all models and the only difference between them is the size of square shaped opening. Pictures below show a rendered view and an elevation of openings in the wall. Other elements in the medium rise building model have been made invisible in order to display a better view of the central core alone.

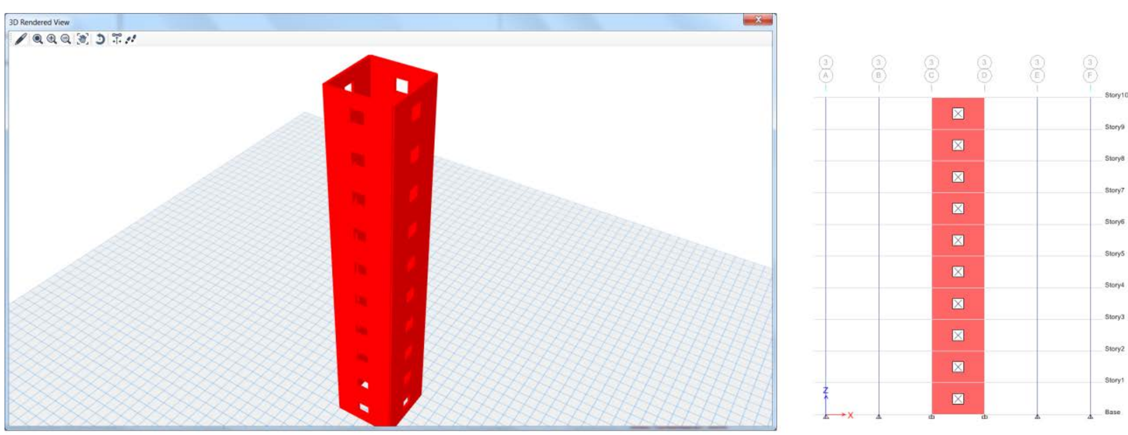


Figure 5.1: Rendered view and elevation of openings in central core

In this comparative study, all the models mentioned above, are analyzed and the results are compared in these categories.

1. Story Stiffness
2. Story Maximum Displacements and Drifts
3. Modal Periods
4. Story Shear

Since the main model, loading and position of openings are all symmetric, the properties and results will be the same in both X & Y direction. Therefore, all of the results displayed in this chapter can represent both directions of the structure. As stated, all of the parameters stay the same and the only parameter which changes at each step is the dimension of opening. This way we can conclude that any change in the results of analysis is due to variations in dimension of openings. Figure below is the model used in this study which includes all the elements. Openings are displayed on the central core.

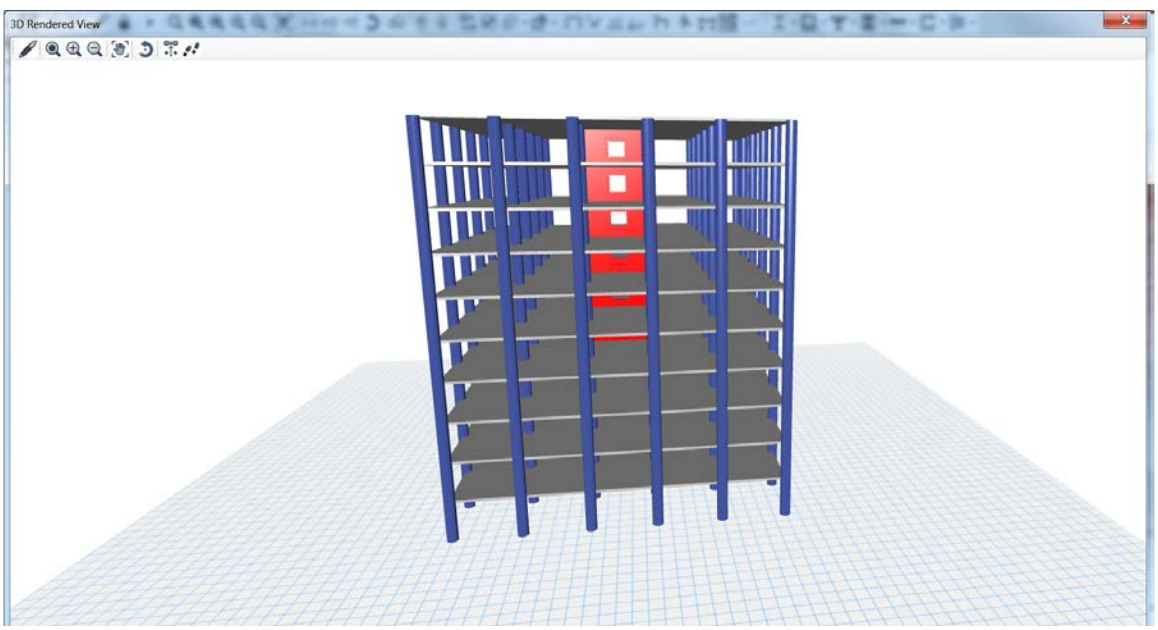


Figure 5.2: Rendered view of opening model

5.2.3 Results and Discussion

a) Story Stiffness

Openings are expected to affect the stiffness of a wall since they reduce the total area which resists lateral loads. Moment of inertia will not be the same throughout the entire height of the shear wall because of openings. Consequently, As the opening ratio in a shear wall increases, rigidity and stiffness of the wall is expected to decrease. At the same time, shear and flexural capacities are also affected by openings. All six models explained in model description were analyzed and the result for story stiffness in each case is provided in the table below.

Table 5.2: Stiffness values - openings

Stories	Story Stiffness (Kips/in)										
	No openings	2x2 (1.67%)	%	4x4 (6.67%)	%	6x6 (15%)	%	8x8 (26.67%)	%	10x10 (41.67%)	%
Story1	23,996.52	21,463.18	-10.56 %	19,491.15	-18.78 %	16,180.26	-32.57 %	12,071.63	-49.69 %	7,526.86	-68.63 %
Story2	13,102.38	14,376.49	9.72 %	13,671.08	4.34 %	11,987.71	-8.51 %	9,396.57	-28.28 %	6,030.76	-53.97 %
Story3	9,109.69	9,364.96	2.8 %	8,942.86	-1.83 %	8,001.57	-12.16 %	6,538.17	-28.23 %	4,444.13	-51.22 %
Story4	7,141.82	7,149.17	0.1 %	6,830.25	-4.36 %	6,169.21	-13.62 %	5,171.91	-27.58 %	3,689.52	-48.34 %
Story5	5,885.34	5,804.28	-1.38 %	5,552.21	-5.66 %	5,049.57	-14.2 %	4,316.30	-26.66 %	3,214.31	-45.38 %
Story6	4,926.65	4,810.66	-2.35 %	4,608.80	-6.45 %	4,216.81	-14.41 %	3,663.06	-25.65 %	2,829.78	-42.56 %
Story7	4,073.90	3,950.91	-3.02 %	3,792.19	-6.92 %	3,490.09	-14.33 %	3,076.67	-24.48 %	2,461.22	-39.59 %
Story8	3,210.72	3,098.65	-3.49 %	2,977.87	-7.25 %	2,755.30	-14.18 %	2,466.03	-23.19 %	2,050.97	-36.12 %
Story9	2,247.91	2,164.17	-3.73 %	2,086.44	-7.18 %	1,944.76	-13.49 %	1,769.83	-21.27 %	1,531.43	-31.87 %
Story10	1,118.28	1,069.59	-4.35 %	1,037.08	-7.26 %	977.50	-12.59 %	907.05	-18.89 %	815.48	-27.08 %

The “%” column compares the results from each model to the model without openings. As seen in the table, the effect of openings on the stiffness is more considerable in higher opening ratios. Overall, each increment in size of opening has resulted into loss of stiffness.

The graph below can help better illustrate the effect of openings on stiffness at each level

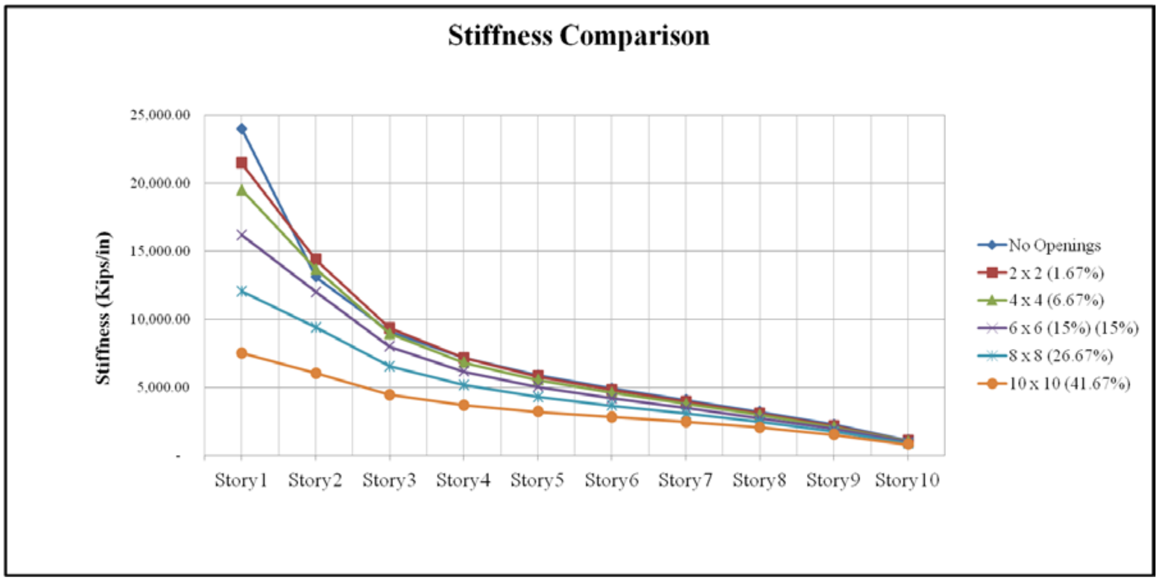


Figure 5.3: Stiffness comparison - openings

The results show that higher opening ratios can reduce the stiffness at different levels up to 68% which can have a great impact on design and analysis of the structure. This influence is deteriorated as the levels increase and at top levels story stiffness is not affected as much as lower levels. Also as expected, smaller openings do not have a great overall impact and can safely be ignored if the engineer decides to (base on other conditions in a project). In the graph shown, it can be observed that the line representing a model with no openings, is moving very closely with those representing 2x2 and 4x4 openings (both below 10% opening ratio) but when the ratio has reached 15% or more , the results start to show more difference in values compared to the model with no openings. This difference is much more significant in opening ratios over 25% and in the graph it can be seen that the two lines representing 8x8 and 10x10 openings (27% and 42% opening ratios) are falling well below other models.

b) Story Maximum Displacements and Drifts

As opening ratio increases, the stiffness of shear walls decrease and less stiffness will result into higher lateral displacements. Maximum story displacement is always one of the major considerations in structural design and it's very important to consider how much openings in shear walls can play a role in lateral displacement of buildings. In this case study, the model is analyzed for different opening ratios and the result of maximum story displacement in provided in the table below.

Table 5.3: Maximum story displacements - openings

Stories	Maximum Story Displacement (in.)									
	No openings	**2x2 (1.67%)**	**% increase**	**4x4 (6.67%)**	**% increase**	**6x6 (15%)**	**% increase**	**8x8 (26.67%)**	**% increase**	**10x10 (41.67%)**	**% increase**
Story1	0.022	0.024	6.50%	0.026	14.50%	0.030	32.10%	0.038	68.30%	0.060	169.20%
Story2	0.057	0.060	4.60%	0.063	10.80%	0.071	25.10%	0.089	55.50%	0.140	145.00%
Story3	0.102	0.106	3.70%	0.111	9.00%	0.124	21.10%	0.151	47.20%	0.231	126.20%
Story4	0.155	0.160	3.20%	0.168	7.80%	0.184	18.50%	0.220	41.50%	0.329	112.00%
Story5	0.214	0.220	2.90%	0.229	7.00%	0.250	16.60%	0.294	37.20%	0.430	100.90%
Story6	0.276	0.283	2.60%	0.294	6.50%	0.318	15.20%	0.369	33.80%	0.529	91.90%
Story7	0.339	0.347	2.40%	0.359	6.00%	0.386	14.10%	0.444	31.00%	0.624	84.30%
Story8	0.401	0.410	2.30%	0.423	5.60%	0.454	13.10%	0.516	28.60%	0.713	77.70%
Story9	0.462	0.472	2.10%	0.486	5.20%	0.518	12.20%	0.584	26.40%	0.793	71.70%
Story10	0.521	0.531	2.00%	0.546	4.90%	0.580	11.40%	0.648	24.40%	0.866	66.40%

Same as story stiffness, the "% increase" column displays the difference between results of each case and the model with no openings (highlighted in orange).

Displacements shown in table above indicate that in higher opening ratios, lateral displacement can be influenced significantly. For instance in 26.7% opening ratio, lateral displacement at the top floor is increased by about 25%. This percentage increases to about 67% in the model with 42% opening which can influence the design and analysis of a structure. But small openings, such as those up to the ratio of about 10%, do not make a remarkable impact on lateral displacement. An opening ratio of 6.7% increases the maximum story displacement at top level by about 5% which can be ignored in engineering decisions.

The other important parameter in lateral analysis which is similar to displacement is drift in each story. Values of strory drifts for each model are provided in the table below. Note that all the story drifts are below the allowable drift calculated in chapter 3.

Table 5.4: Story drifts - openings

Stories	Story Drift (in.)										
	No openings	**2x2 (1.67%)**	**%**	**4x4 (6.67%)**	**%**	**6x6 (15%)**	**%**	**8x8 (26.67%)**	**%**	**10x10 (41.67%)**	**%**
Story1	0.015	0.017	9.21 %	0.018	17.11 %	0.020	33.55 %	0.025	66.45 %	0.040	164.47 %
Story2	0.028	0.025	-11.23 %	0.025	-9.06 %	0.027	-1.45 %	0.032	16.67 %	0.050	80.07 %
Story3	0.039	0.037	-4.92 %	0.037	-3.11 %	0.040	2.85 %	0.045	17.1 %	0.066	70.47 %
Story4	0.047	0.046	-2.56 %	0.047	-0.43 %	0.049	4.69 %	0.055	16.42 %	0.076	61.41 %
Story5	0.053	0.052	-0.95 %	0.053	0.95 %	0.056	5.68 %	0.061	15.53 %	0.081	53.41 %
Story6	0.057	0.057	0 %	0.058	1.94 %	0.060	6.18 %	0.065	14.31 %	0.083	46.47 %
Story7	0.059	0.059	0.85 %	0.060	2.56 %	0.062	6.32 %	0.066	13.16 %	0.082	40 %
Story8	0.059	0.060	1.53 %	0.061	3.23 %	0.063	6.62 %	0.066	11.88 %	0.079	33.28 %
Story9	0.058	0.059	1.72 %	0.060	3.26 %	0.062	6 %	0.064	9.78 %	0.073	25.73 %
Story10	0.057	0.058	2.65 %	0.059	3.7 %	0.060	5.47 %	0.061	7.58 %	0.067	18.87 %

Graphs below illustrate the impact of openings on maximum stroy displacements and story drifts respectively.

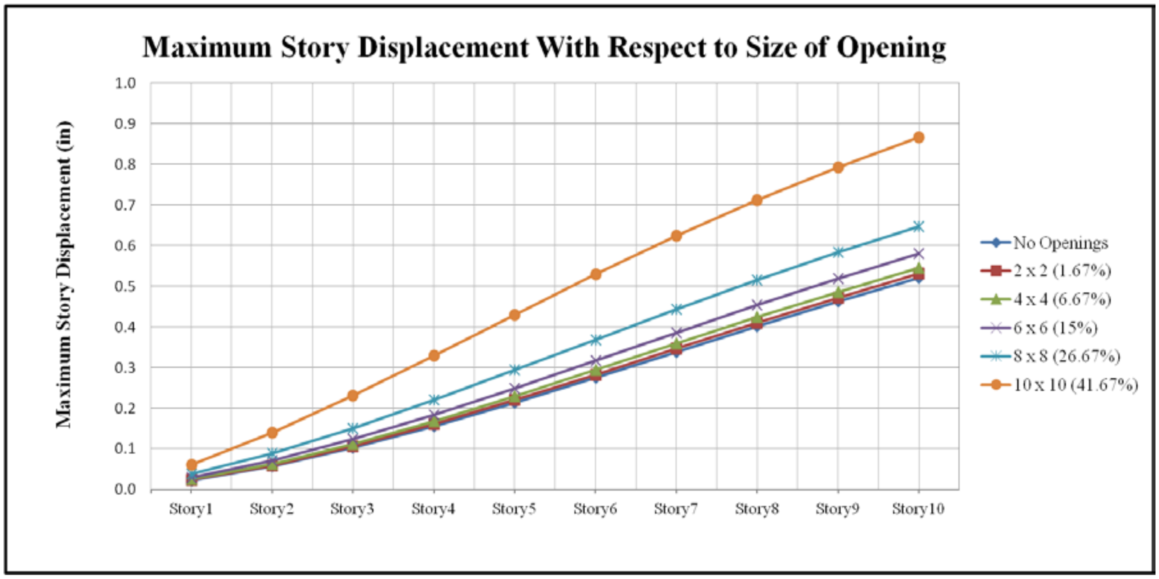


Figure 5.4: Displacement comparison - openings

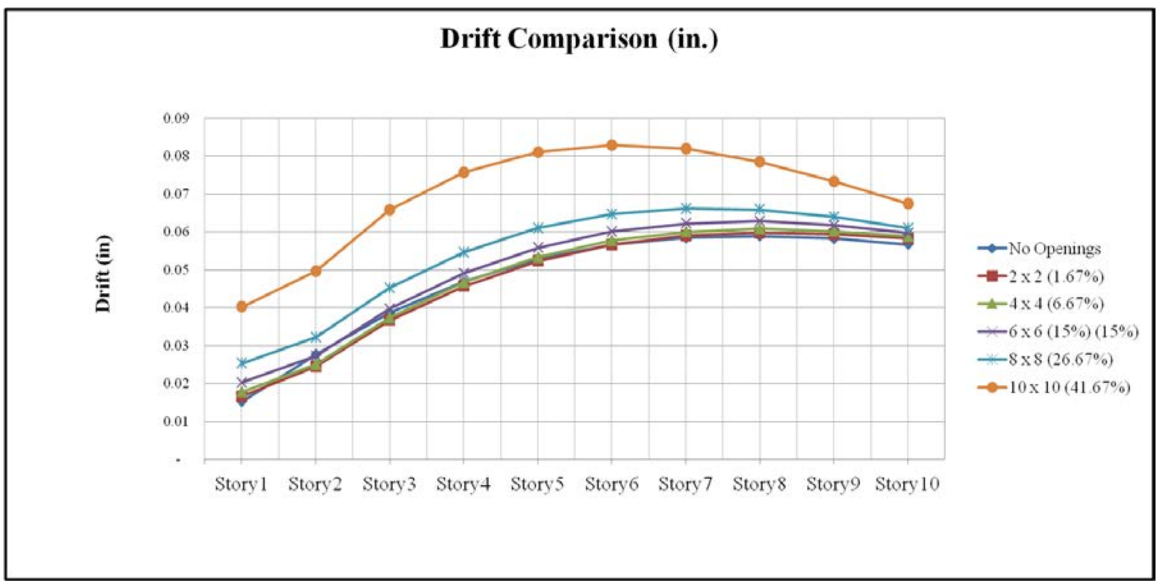


Figure 5.5: Drift comparison - openings

The graphs shown above agree with the general idea that increased opening ratio can cause a higher lateral displacement and story drift. As seen in the graph of maximum displacements, the first few lines which represent no openings, 1.67% and 6.67% opening ratios are very close to each other in every level. It can be concluded that opening ratios of up to about 10% may not have a significant effect on lateral displacement. But opening ratios of 15% and more are showing much larger displacements than the original model without any openings. At the same time, if effects of cracking (as studied in chapter 4) are combined with the results in here, the maximum story displacement would be even higher than what is presented in this chapter. The reason is that usually around corners of openings stress concentration can cause stresses higher than the rupture value and as a result, crack the concrete. This is why special requirements should be considered in reinforcement around openings. These cracked areas will not provide the same stiffness as uncracked elements for the reasons described in chapter 4 and therefore reduce the stiffness of the structure even more than what is provided in this chapter. Story drifts are also following the same pattern. Openings in the shear wall are causing larger drift values and the higher these opening ratios, the larger the story drifts.

It is important to note that all of the results in this category are based on the earthquake described in general model information plus an accidental eccentricity (earthquakes 3,4,5 and 6) These load cases result into the highest lateral displacements and thus are the most critical loading conditions when lateral displacements and story drifts are considered. Results from other earthquake conditions (earthquake 1 and 2) show the same pattern except lateral displacements are less if accidental eccentricity is not considered. Due to symmetry, the results are identical in X and Y directions and so all of the results provided in this section represent maximum story displacements in both directions.

c) Modal Periods

Since natural period of a structure is in relation with stiffness of the structure and openings can affect total stiffness of shear walls (and consequently the entire structure), it can be concluded that modal periods are also subject to change as opening ratios increase. Comparison of modal periods can show how much a structure is affected by these openings. Results from the analysis of all six models for this study on natural periods are presented in the table below.

Table 5.5: Natural Periods - openings

Opening Size	Natural Period (seconds)	Increase Percentage
No Opening (0%)	1.038	-
2ft x 2ft (1.67%)	1.06	↑ 2.22 %
4ft x 4ft (6.67%)	1.106	↑ 6.55 %
6ft x 6ft (15%)	1.243	↑ 19.75 %
8ft x 8ft (26.67%)	1.515	↑ 45.95 %
10ft x 10ft (41.67%)	1.968	↑ 89.60 %

As shown in the table above, natural period of the structure is not heavily affected in the first two models which include 2x2 and 4x4 openings. An increase of about 6.5%, which represents a 0.07 second increase in natural period of the structure, proves this argument. When 6x6 opening model is analyzed however, the results indicate that the structure is having a relatively higher natural period which compared to the model without any openings the natural period has increased by about 0.2 seconds or 20%. Last two models (26.67% and 41.67% opening ratios) clearly show that openings are playing a major role in modal periods of the structure. Natural periods are increased by about 50% and 90% respectively which is significant. Results indicate the fact that in cases where ratios of openings are higher, the impact that opening have on stiffness and natural period must be considered and cannot be ignored.

The graph below helps illustrate how increased ratios of openings influence the behavior of a structure under lateral loads.

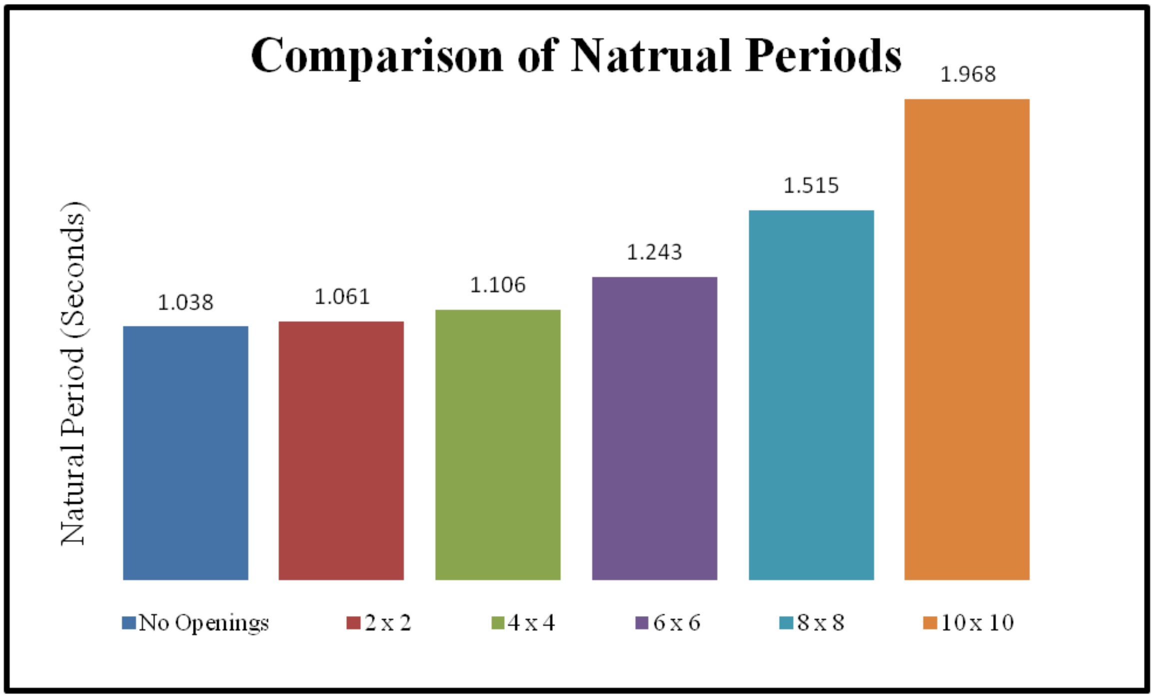


Figure 5.6: Natural period comparison - openings

It is better observed in the graph that the first three models (no openings, 2x2 and 4x4) show results that are fairly close to each other and the difference between the height of columns representing them in the graph is not easily seen. However, the next three models are showing a different behavior and the difference in the results between these periods and the first three is clearly visible. The pattern remains the same as the opening ratios increase to the point that natural period in the final model is almost double the natural period of model with no openings. From the graph, it can be verified that in this particular project, an opening ratio of 10% can be recognized as the threshold of openings that may not have significant influence on the results but it might not be safe to ignore opening ratios higher than 10%.

d) Story Shear

It is expected that reduced stiffness results into less shear at each story based on structural dynamics methods. Since openings decrease the stiffness of the structure at each level and natural period is increased, less shear is applied to the structure. Analysis results in the table below agree with these expectations.

Table 5.6: Story shear - openings

Stories	Story Shear (Kips)									
	No openings	2x2 (1.67%)	%	4x4 (6.67%)	%	6x6 (15%)	%	8x8 (26.67%)	%	10x10 (41.67%)	%
Story1	365.76	356.87	-2.43 %	347.41	-5.02 %	329.26	-9.98 %	305.75	-16.41 %	302.65	-17.25 %
Story2	361.60	352.90	-2.41 %	343.64	-4.97 %	325.87	-9.88 %	302.83	-16.25 %	299.76	-17.1 %
Story3	351.57	343.26	-2.36 %	334.40	-4.88 %	317.39	-9.72 %	295.33	-16 %	292.34	-16.85 %
Story4	334.81	327.06	-2.31 %	318.79	-4.78 %	302.92	-9.52 %	282.32	-15.68 %	279.47	-16.53 %
Story5	310.66	303.65	-2.26 %	296.16	-4.67 %	281.77	-9.3 %	263.09	-15.31 %	260.45	-16.16 %
Story6	278.61	272.50	-2.19 %	265.95	-4.55 %	253.37	-9.06 %	237.04	-14.92 %	234.69	-15.77 %
Story7	238.23	233.15	-2.13 %	227.70	-4.42 %	217.25	-8.8 %	203.67	-14.51 %	201.68	-15.34 %
Story8	189.13	185.22	-2.07 %	181.03	-4.28 %	172.98	-8.54 %	162.53	-14.07 %	160.98	-14.88 %
Story9	130.97	128.35	-2 %	125.55	-4.13 %	120.18	-8.24 %	113.20	-13.57 %	112.19	-14.33 %
Story10	63.43	62.22	-1.91 %	60.95	-3.91 %	58.50	-7.78 %	55.30	-12.82 %	54.93	-13.4 %

Comparison of story shear at different levels once again indicates the fact that small openings may not have a significant role in the forces which will be applied to the structure. For openings up to the ratio of 6.67%, maximum shear difference is only 5% at first level which will not make a remarkable effect. In this case, shear at first level drops by about 18 kips which can be ignored. However, higher opening ratios can affect story shear more severely and appropriate considerations are required.

All of the information in this table can be summarized into a graph that shows the pattern in which story shear applies to the structure based on increased opening ratios. This graph also shows that low percentage of openings does not make a great impact while higher percentages require more attention.

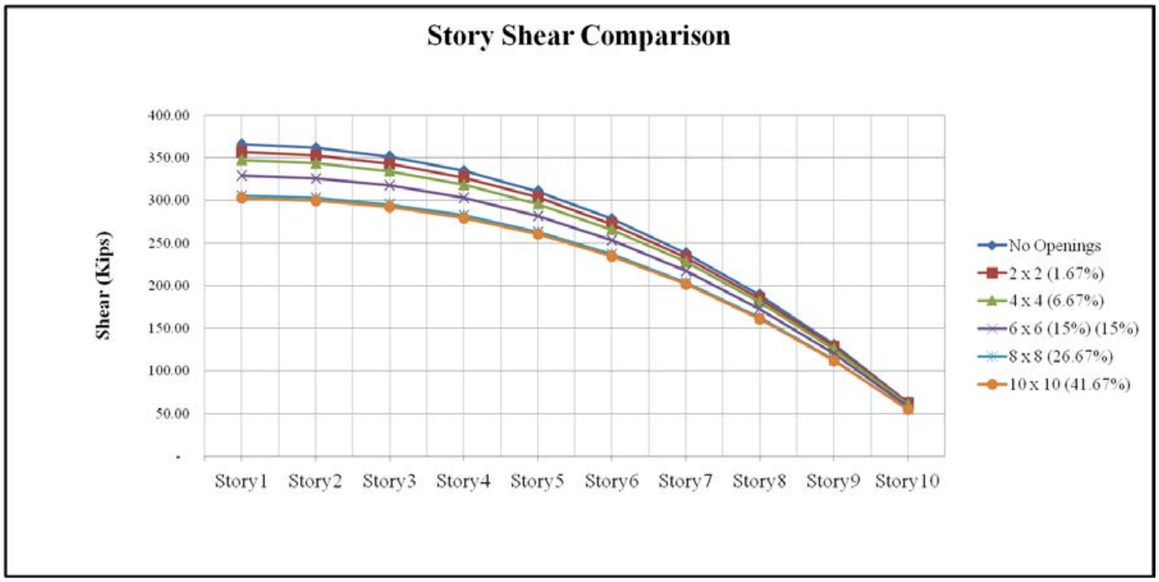


Figure 5.7: Story shear comparison - openings

As seen in the graph above, story shear is less affected by openings in the upper levels of the structure. At level 10, almost all of the models are on the same point. Based on information from the table, the difference between original model and the model with 10x10 opening is only about 13% at the top level. But the first few levels are more influenced by dimensions of opening. It is also be observed that in story shear, the first three models are moving very close to each other in the graph indicating that opening ratios up to 10% in this project, do not contribute as much to the final results. However, openings with ratios higher than 10% start to show results that are considerably different. In the case of story shear, the line representing 15% opening is well below the first three models.

As stated before, this study was mainly focused on stiffness and factors related to stiffness of a structure since it is the main topic of this research. However, it is very important to note that openings ,besides their effects on stiffness which was covered in this study, can also influence the shear and flexural capacity which is very critical and special requirements must be taken into account. ACI code addresses this issue in chapter 21, Earthquake-resistant structures. In provision 21.9.5 (design for flexure and axial loads) the code states that "For walls with openings, the influence of the opening or openings on flexural and shear strengths is to be considered and a load path around the opening or openings should be verified. Capacity-design concepts and strut-and tie models may be useful for this purpose".

Besides the issues with capacity, certain requirements are also provided for the reinforcement around openings in shear walls. ASCE code in chapter 12 (seismic design requirements for building structures) talks about design and detailing requirements and in provision 12.14.7.2 it states that "Openings in shear walls, diaphragms, or other plate-type elements, shall be provided with reinforcement at the edges of the openings or reentrant corners designed to transfer the stresses into the structure. The edge reinforcement shall extend into the body of the wall or diaphragm a distance sufficient to develop the force in the reinforcement.". ACI code provides more details related to reinforcement around openings in chapter 14 (walls). In provision 14.3.7 ACI dictates that "In addition to the minimum reinforcement required by 14.3.1, not less than two No. 5 bars in walls having two layers of reinforcement in both directions and one No. 5 bar in walls having a single layer of reinforcement in both directions shall be provided around window, door, and similar sized openings. Such bars shall be anchored to develop F_y tension at the corners of the openings."

5.3 Skewed Walls and Stiffness of Structures

5.3.1 Introduction

Ordinary shear walls are usually simple rectangular sections with one dimension much bigger than the other. As a result, moment of inertia about one axis of the wall is much greater than the weaker axis. Therefore, it is a common practice that engineers usually ignore the effect and stiffness of shear walls about their weak axis which results into a more conservative design. Structural analysis programs (such as ETABS) also provide the option to ignore stiffness of shear walls about the weaker axis since less equations will be required to solve and analyze the model, besides the fact that the contribution is very small compared to strong axis of shear walls.

For all of the reasons mentioned above, as it is seen in most of the shear wall-frame structures, shear walls are placed in two orthogonal directions and are analyzed separately in both directions. For instance, when the structure is being analyzed in X-direction, only the walls parallel to that direction are considered and the rest of the walls are ignored. But how will the structure react if walls are not placed in two orthogonal directions and skewed walls are expected to resist lateral loads? What will be the contribution of skewed walls to stiffness of a building in both directions?

The purpose of this study is to investigate these questions and compare how different configurations of shear walls relative to each other can affect stiffness in X and Y directions. Behavior and stiffness of shear walls at different angles is considered and the results are compared with each other using tables and graphs. To be able to investigate the effects of walls that are not parallel to loading, settings in ETABS must be set to account for both out of plane and in plane stiffness.

5.3.2 Model Description and Methodology

For this part of the research, the medium rise building model with intermediate walls is considered (described in chapter 3). Two of the walls which are parallel to X axis will rotate and will no longer be parallel to direction of loading, then the analysis is performed. All of the material properties and loading conditions are those described in general model information. The walls are 20ft wide and 10in thick and all other dimensions are those used in creating the medium rise building model in chapter 3.

To make data easier to interpret and also better illustrate the effects of skewed shear walls, the load is constantly applied in positive X direction and the walls rotate at 15 degree increments. As a result, the first model is the same as medium rise building with intermediate walls parallel to X. Second model has all the same properties and geometry, except the walls that were parallel to X axis are now having a 15 degree angle with direction of lateral loads. Therefore, not all the load will be carried by the strong axis and weaker axis also gets involved with some of the forces. It can be observed that in small angles, even though majority of the load will still be applied to strong axis, some of it will be applied to weak axis of the wall. The portion of the load applied to weaker axis increases as the angle is increased and since the weak axis is not nearly as stiff as the strong axis of the wall, stiffness is expected to decline in higher angles. To investigate the behavior of structure in different situations, walls are modeled at 0, 15, 30, 45, 60, 75 and 90 degrees from direction of lateral loads.

Another expectation is that even though stiffness in X direction is supposed to decline as angle increases, stiffness in Y will have to increase since strong axis of walls will be closer to being parallel to loads in Y direction in higher angles. Therefore in 90 degrees, all four walls will be parallel to loads in Y direction, and it is expected to increase the stiffness considerably while at this case, stiffness in X must be the lowest of all.

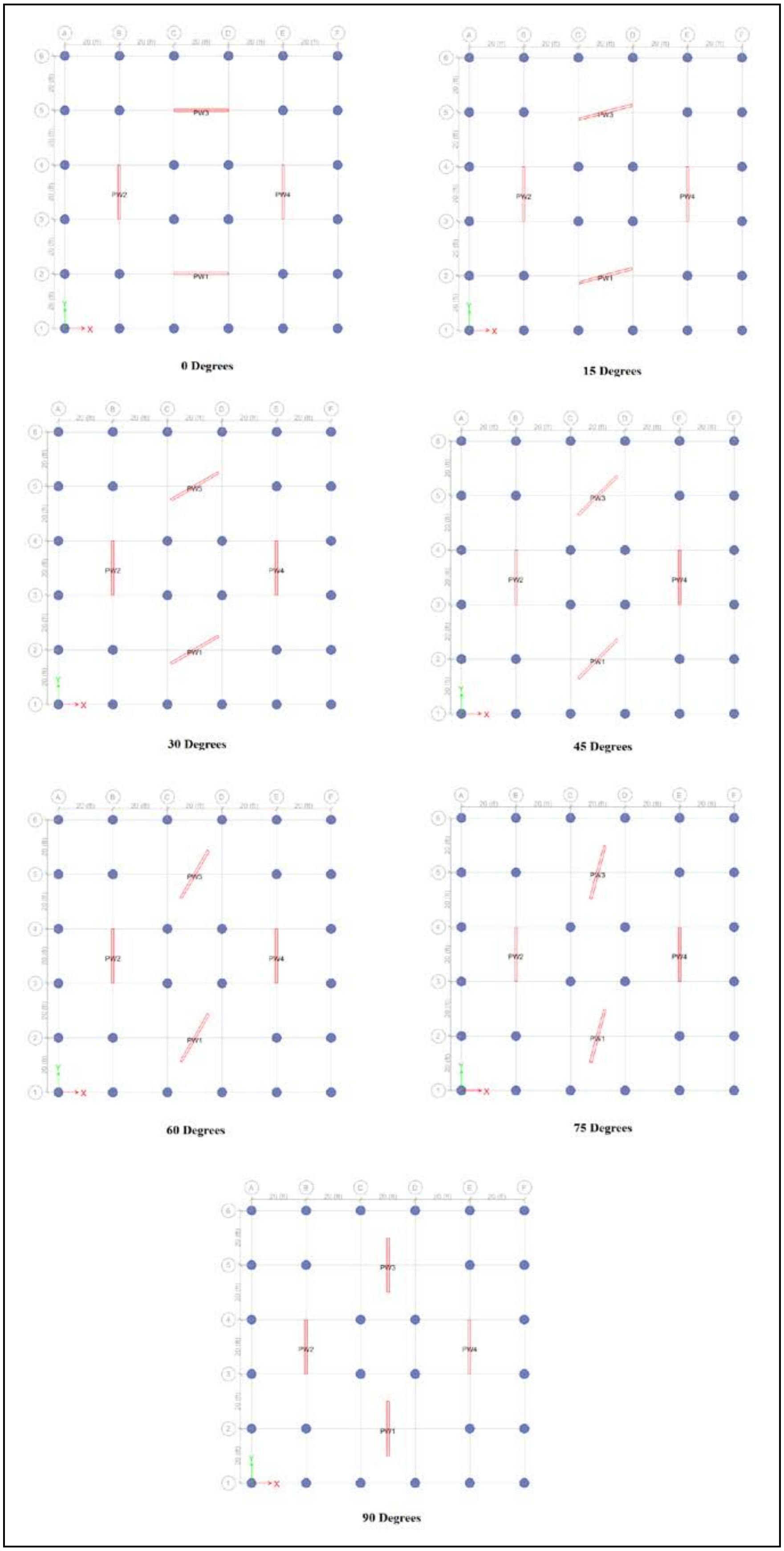


Figure 5.8: All of the models used in analysis of skewed walls

5.3.3 Results and Discussion

After running the analysis for all of the models described in section 5.3.2, results for two parameters are investigated

1. Stiffness (k/in.)
2. Drift (in.)

These two parameters are directly related to each other and they can describe the behavior of shear walls at each angle. Table below summarizes the results from all the models that are skewed relative to direction of loading.

Table 5.7: Stiffness values in X direction - skewed walls

Stiffness in X Direction	Stiffness Obtained for Each Angle (Kips/in.) - X Direction						
	0 ^{o}	15 ^{o}	30 ^{o}	45 ^{o}	60 ^{o}	75 ^{o}	90 ^{o}
Story1	12,792.42	11,281.56	8,407.66	5,379.92	2,979.35	1,457.59	943.42
Story2	6,224.53	5,610.15	4,511.87	3,272.16	2,164.47	1,303.35	943.10
Story3	4,310.72	3,900.26	3,205.43	2,425.08	1,733.35	1,186.53	942.72
Story4	3,450.19	3,128.20	2,608.82	2,026.45	1,514.15	1,116.67	943.26
Story5	2,927.99	2,658.85	2,245.94	1,783.61	1,378.51	1,069.37	940.86
Story6	2,536.41	2,306.20	1,972.00	1,598.95	1,273.19	1,027.37	930.64
Story7	2,176.60	1,981.31	1,715.24	1,420.34	1,164.66	973.51	902.72
Story8	1,781.83	1,623.64	1,423.51	1,204.96	1,019.10	882.73	836.50
Story9	1,293.20	1,179.54	1,047.20	906.56	792.20	712.55	690.29
Story10	663.02	605.02	542.94	479.24	430.75	400.02	394.78

Results for stiffness in X direction show that as expected, stiffness declines when walls are skewed and the bigger the angle gets, the more the stiffness is decreased. It is observed that this reduction of stiffness is more severe in lower levels since shear walls tend to govern the stiffness at bottom floors. Even a small angle such as 15 degrees has resulted in a loss of stiffness of about 1,500 kips/in at first floor which is about 12% reduction in stiffness. In higher angles such as 45 degrees this reduction percentage is closer to 60% which is significant.

As the angle of walls which were parallel to X increases, the stiffness in Y direction is expected to increase since skewed walls can contribute in both directions. Table below provides the results for stiffness in Y as walls are placed in different angles.

Table 5.8: Stiffness values in Y direction - skewed walls

Stiffness in Y Direction	**Stiffness Obtained for Each Angle (Kips/in.) - Y Direction**						
	0 ^{o}	**15 ^{o}**	**30 ^{o}**	**45 ^{o}**	**60 ^{o}**	**75 ^{o}**	**90 ^{o}**
Story1	12,795.54	13,033.45	13,326.33	13,961.60	15,331.37	18,874.71	24,349.13
Story2	6,226.39	6,396.36	6,657.09	7,170.02	8,069.75	9,587.42	10,953.02
Story3	4,312.16	4,444.60	4,649.03	5,048.33	5,733.99	6,759.23	7,468.79
Story4	3,451.45	3,566.27	3,740.22	4,074.64	4,638.96	5,431.41	5,903.13
Story5	2,929.16	3,033.56	3,188.14	3,479.57	3,962.10	4,607.52	4,952.69
Story6	2,537.51	2,633.94	2,773.18	3,030.09	3,447.11	3,981.25	4,242.56
Story7	2,177.62	2,265.69	2,389.59	2,612.91	2,967.88	3,403.67	3,599.59
Story8	1,782.73	1,859.37	1,964.42	2,149.13	2,436.42	2,774.08	2,913.01
Story9	1,293.90	1,352.80	1,431.67	1,566.85	1,772.63	2,003.96	2,090.52
Story10	663.40	695.14	736.66	805.89	909.50	1,021.86	1,060.44

Results indicate that stiffness in Y direction is not heavily influenced in the first 45 degrees. In this range, stiffness at first floor has only increased by 9% but in higher degrees, the impact can be significant. When walls are rotated by 90 degrees, as expected, stiffness in Y direction is almost doubled because there are four shear walls parallel to Y direction and there are no walls to resist lateral loads in X direction.

Figure 5.9: Stiffness comparison in X direction - skewed walls

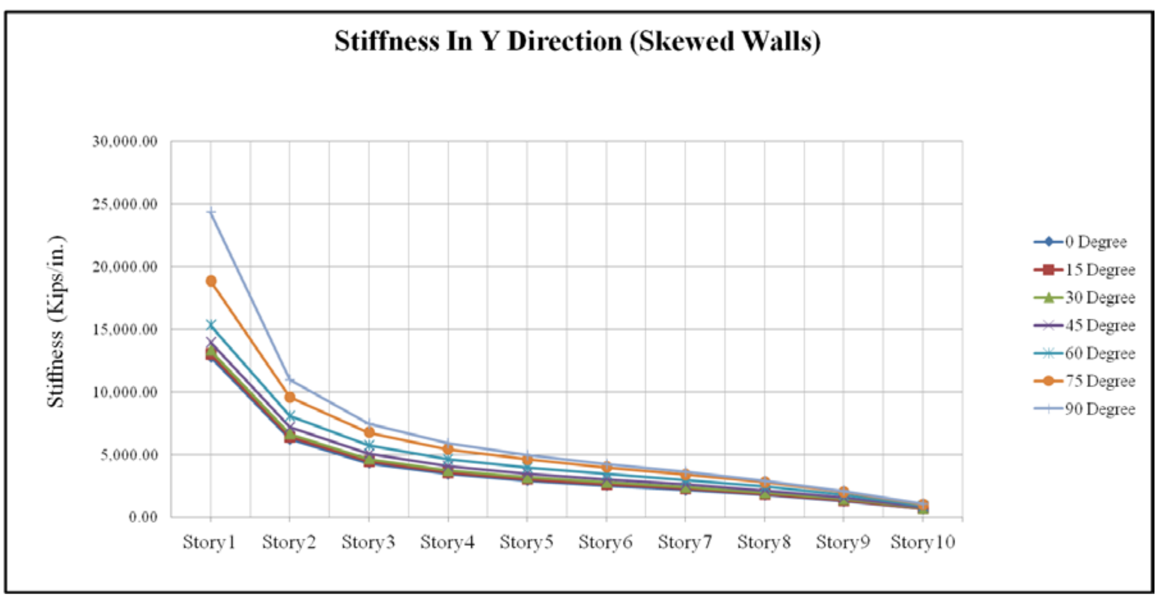


Figure 5.10: Stiffness comparison in Y direction - skewed walls

Graphs and figures clearly display how stiffness in both directions is affected when skewed walls are present. In first figure (stiffness in X direction) it can be observed that stiffness is declining considerably with each increment in angle of walls. Lines representing small angles such as 15 and 30 are falling well below the line for 0 degrees and the difference between stiffness values in angles higher than 30 are significantly more. In this figure, it can be easily verified that walls perpendicular to direction of load have such small contribution to stiffness that can be ignored in analysis and design of structures. The line representing a 90 degree rotation is almost flat at the lowest stiffness values and it's stiffness in X direction is only 7% of what it is in a wall which is not skewed at first level. However it is seen that in higher levels, the difference between stiffness values of different models is decreased since shear walls and frames tend to interact differently.

On the other hand, stiffness in Y direction is not greatly increased in the first 45 degrees. The graph shows that the first few lines (0, 15, 30 and 45 degrees) are moving very close to each other in all levels. Even though a 45 degree angle can reduce the stiffness in X direction by a considerable amount, it does not contribute to stiffness in Y direction significantly. However, the influence of 60 and 75 degrees on stiffness in Y direction is more observable in the graph. In 90 degrees, basically all of the walls are parallel to each other and the direction of loading in Y direction. So it can be assumed that number of shear walls is doubled and as a result the stiffness is also expected to increase significantly. In this case, there are no shear walls resisting lateral loads in X direction and the stiffness is extremely lower than the case where walls are not skewed. To better understand the difference between stiffness in X and Y directions when all of the walls are parallel to each other, the line representing 90 degrees can be compared in both figures. This line in first graph (Stiffness in X) is far below other lines and represents lowest stiffness values of all. On the other hand, the 90 degrees line in second figure (Stiffness in Y) shows the highest stiffness values and is placed well above all the other lines.

The same behavior is observed when drifts are compared. In X direction, since skewed walls are reducing the stiffness, drift values are increasing as a result. On the other hand, because of contribution of skewed walls to stiffness in Y direction, drift values are declining. These impacts are better illustrated in figures below.

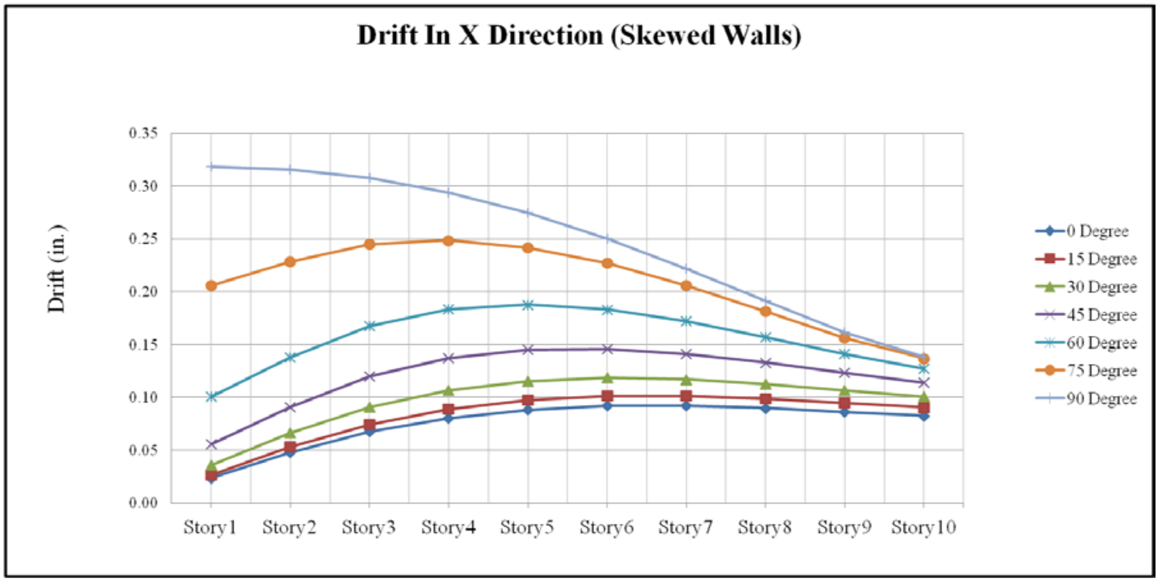


Figure 5.11: Drift comparison in X direction - skewed walls

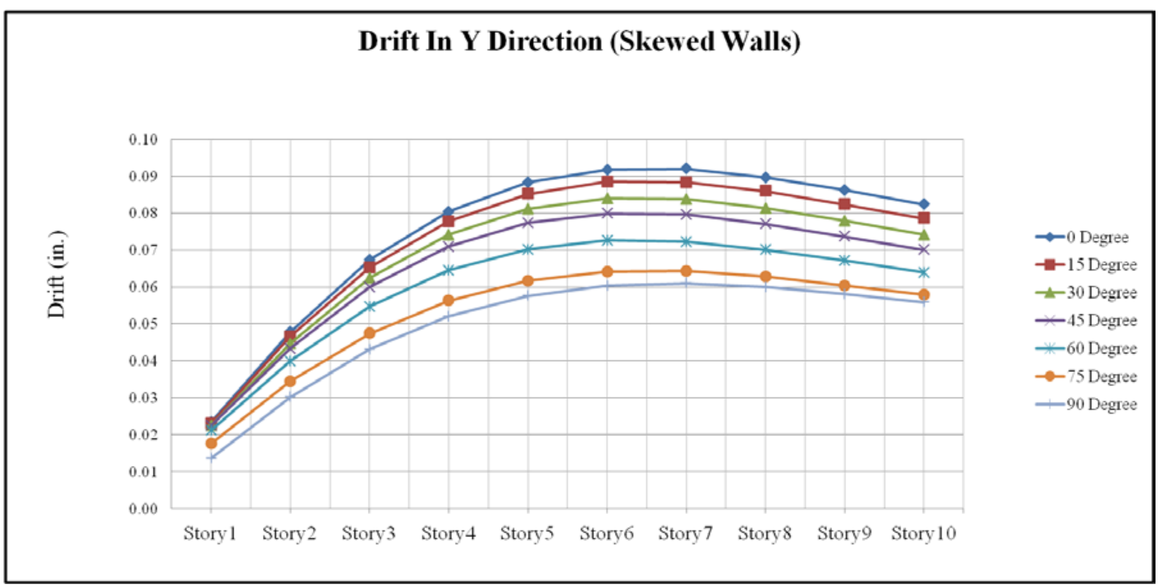


Figure 5.12: Drift comparison in Y direction - skewed walls

Based on the results obtained, it can already be confirmed that shear walls are not effective about their weak axis at all and the stiffness of walls in this direction can be ignored. This is due to the fact that the moment of inertia about weak axis is significantly smaller than the strong axis. If the model used in this chapter is considered, the moment of inertia in X & Y directions are calculated using equations below.

$$I_X = \frac{bh^3}{12} = \frac{10 \times 240^3}{12} = 1{,}152 \times 10^4$$

$$I_Y = \frac{hb^3}{12} = \frac{240 \times 10^3}{12} = 2 \times 10^4$$

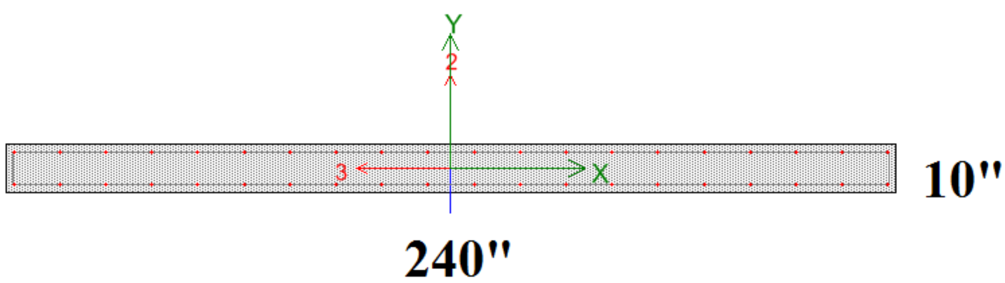


Figure 5.13: Typical shear wall section used in analysis of skewed walls

The moment of inertia about strong axis based on calculations is much higher than in weak axis. One of the main parameters involved in stiffness of a section in a particular direction is the geometry characteristics. When there is such a great difference between moment of inertia in two directions, it can be expected that the stiffness in two directions (and directions in between X and Y) can be very different from each other. But since the highest stiffness is obtained when the wall is parallel to direction of the load, it can be assumed that in order to optimize the design, it might be better to put shear walls in two orthogonal directions and avoid having skewed walls resist lateral loads.

It is important to note that the term “Skewed Walls” refers to the positioning of shear walls relative to each other and not necessarily the angle between shear walls and global coordinates. In other words in this study if walls are placed in two orthogonal directions, they will not be considered as skewed walls since forces can be distributed to strong axis of walls regardless of the direction of loading. But if shear walls are not placed in two orthogonal directions, then distribution of loads can be different. Picture below shows that in this model, even though the walls seem to be skewed relative to global coordinates, the stiffness in X and Y is not influenced the way it was explained in this section since the walls are placed in two orthogonal directions regardless of their position in the plans.

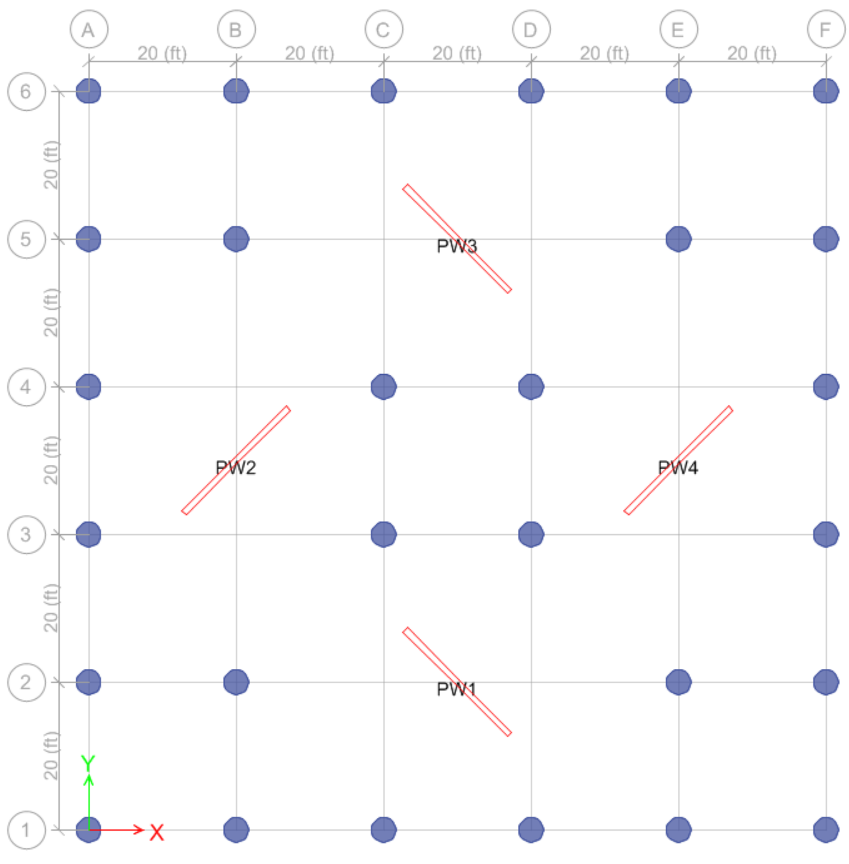


Figure 5.14: Shear walls placed in two orthogonal directions but not parallel to X and Y

5.4 Flexural Capacity of Wall Sections

5.4.1 Introduction

Shear walls are designed to resist lateral loads such as earthquake and wind, thus it is important to design these elements such that

1. Story drifts and maximum story displacements are limited to allowable amounts
2. Capacity of walls (both in flexure and shear) is higher than factored forces caused by the combination of lateral and gravity loads

Most of the discussions up until this chapter were focused on stiffness of the structure and how different parameters can affect the behavior of a building under lateral loads. This section however, is mainly focused on parameters which influence the flexural capacity of a section used as a shear wall.

The strength and behavior of short shear walls (one or two story buildings) are generally controlled by shear. These walls are also called squat walls. But if the wall is more than three or four stories, usually flexural forces control the design. Since the buildings studied in this research are mainly buildings over three or four stories, only flexural capacity is considered.

There are two factors that play a critical role in determining flexural capacity of a section, vertical reinforcement and thickness of the shear wall. There are times that structural engineers do not have many options regarding the size of the wall because of architectural reasons. However, if increasing the thickness of shear walls is an option, engineers must decide whether extra thickness is needed or additional reinforcement can provide required capacity. Budget plays an important role in such decisions, but it is also critical to know how certain strength can be achieved without excessive use of material by having a good understanding about behavior of shear walls.

5.4.2 Model Description and Methodology

Shear walls are concrete members which resist a combination of lateral and gravity loads. They can be assumed as cantilevers fixed at bottom which resist shear, bending and axial loads at the same time. To summarize, shear walls can be treated as beam-columns. Shear can be very well taken care of, using adequate concrete section and shear reinforcement, but combination of axial load and bending moment is critical in design of the wall. It is a common practice among engineers to analyze shear walls like reinforced concrete columns when the wall is simple in shape and estimate the flexural capacity of shear walls. The same approach is used in this analysis and interaction curves are developed similar to how columns are analyzed.

It is recommended to set the minimum thickness of a shear wall to 1/20th of the unsupported height of wall and preferably to 1/15th (MacGregor). Since in both medium rise and high rise models described in this research, unsupported height of wall is 12ft, the minimum thickness of wall will be

$$\frac{1}{15} \times (12' \times 12) = 9.6'' \rightarrow \text{say } 10''$$

As a result, three thicknesses are considered for this research.

1. 10 inch wall
2. 11 inch wall
3. 12 inch wall

Based on ACI code section 14.3 minimum ratio of vertical reinforcement area to gross concrete area shall be 0.0012 for bars not larger than no.5 with specified yield strength not less than 60 ksi. Also, maximum spacing between rebars shall not be further apart than smaller of three times thickness of the wall and 18 inches. Since the smallest thickness is 10 inches, maximum spacing will be 18″.

The same provision in ACI indicates that since the thickness of wall is more than 10 inches, two curtains of reinforcement (each consisting of not less than half of the total minimum reinforcement) are required. Three different reinforcement formations are considered for this research.

1. #3 @ 12″ (As = 0.11 in^2/ft)

2. #4 @ 12″ (As = 0.20 in^2/ft)

3. #5 @ 12″ (As = 0.31 in^2/ft)

The model used for this study is relatively simple. A section of a typical rectangular shear wall used in medium rise buildings is considered. All of the material properties are the same as those described in general model information. Since each span is 20ft, width of all of the models is considered to be 240″. In one series of models the thickness is set to 10″ and reinforcement increases, in the other series reinforcement is set to #3@12″ and thickness increases. The section is then analyzed as a column and interaction curves are produced. Picture below shows the geometry and different conditions considered in analyses.

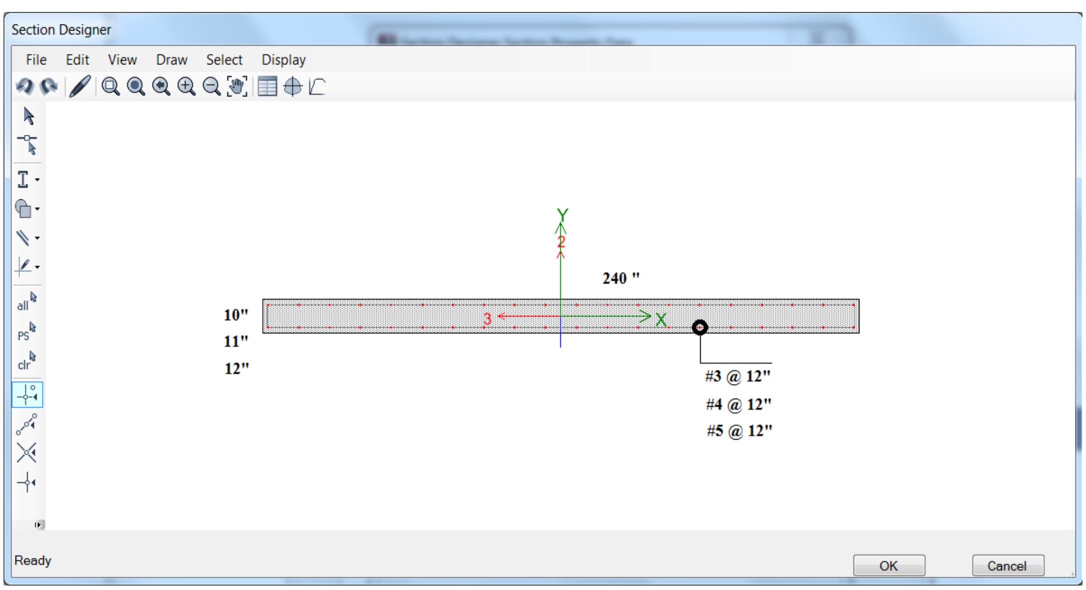


Figure 5.15: Sections and reinforcements used in flexural capacity models

5.4.3 Results and Discussion

This chapter will be divided into two parts: a) Reinforcement b) Thickness

a) Reinforcement

As stated, three models are analyzed for this part of the research.

1. 10in. wall thickness with #3 @12″ (As = 0.11 in^2/ft)
2. 10in. wall thickness with #4 @ 12″ (As = 0.20 in^2/ft)
3. 10in. wall thickness with #5 @ 12″ (As = 0.31 in^2/ft)

Corner rebars are spaced slightly closer to each other to provide enough room for side covers, but overall there will be 40 rebars (20 in each curtain). ETABS analyzes the section and provides capacities for 11 points in interaction curves. By connecting these points together, interaction curves are created. Table below summarizes the results of analyses.

Table 5.9: Points on interaction diagram - vertical reinforcement

Point	Vertical Reinforcement					
	#3 @ 12"		#4 @ 12"		#5 @ 12"	
	P (kips)	M (kips-ft)	P (kips)	M (kips-ft)	P (kips)	M (kips-ft)
1	5431.56	0.00	5535.92	0.00	5663.48	0.00
2	5431.56	6561.44	5535.92	6785.11	5663.48	7057.81
3	5322.89	11410.57	5404.43	11678.37	5504.17	12004.84
4	4602.45	14786.62	4670.01	15114.36	4752.70	15514.05
5	3875.28	16697.30	3923.36	17107.12	3982.25	17607.27
6	3135.31	17163.35	3153.40	17694.34	3175.63	18342.83
7	2886.13	18993.23	2870.88	19694.05	2852.33	20550.13
8	2512.67	19574.49	2452.28	20389.47	2378.49	21384.85
9	1598.54	14836.94	1495.57	15517.14	1369.68	16347.66
10	684.271	8337.324	538.59	8765.678	360.421	9287.897
11	-237.6	0	-432	0	-669.6	0

Table above shows how different points in interaction diagram react to increase in vertical reinforcement, but to be able to better understand the pattern that these values follow, a graph can be more helpful. All of the data from the table is illustrated in figure below.

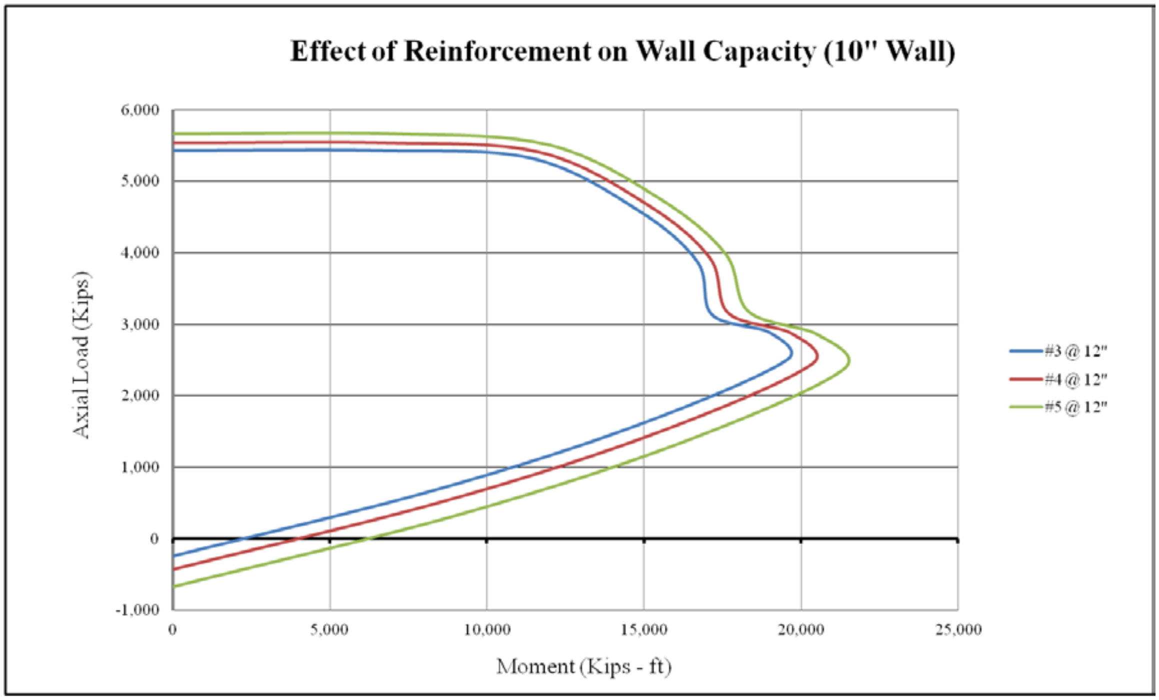


Figure 5.16: Effect of vertical reinforcement on interaction diagram

As expected, total capacity of section is increased as the reinforcement ratio increases. Maximum axial load capacity is increased only by 4.3% and maximum moment capacity at point 8 is only increased by 9.3% while reinforcement is increased by about 182% (from model 1 to model 3). By looking at the graph, it is clear that all three models are providing values very close to each other and the difference between these three curves is not great. This is while reinforcement has increased significantly from model 1 to model 2 (almost double the reinforcement) and the same from model 1 to model 3.

b) Thickness

Besides reinforcement, other parameter that can play an important role in capacity of a section (both in shear and flexure resistance) is thickness of the wall. As mentioned earlier, sometimes due to architectural reasons, thickness cannot be increased. But if there is room to increase the depth of the wall, it is very important to know how much it can help the design and whether it's significant or not. Three different models are analyzed for this research

1. 10in. wall thickness with #3 @12″
2. 11in. wall thickness with #3 @12″
3. 12in. wall thickness with #3 @12″

The goal of the study is to find how increments in thickness of a shear wall can influence the flexural capacity of that section. Reinforcement is kept constant in all three models to #3 @ 12″

Table 5.10: Points on interaction diagram - thickness

Point	Wall Thickness					
	10" Wall		11" Wall		12" Wall	
	P (kips)	M (kips-ft)	P (kips)	M (kips-ft)	P (kips)	M (kips-ft)
1	5431.56	0.00	5961.96	0.00	6492.36	0.00
2	5431.56	6561.44	5961.96	7190.35	6492.36	7819.27
3	5322.89	11410.57	5845.18	12519.07	6367.46	13627.57
4	4602.45	14786.62	5054.38	16225.38	5506.30	17664.13
5	3875.28	16697.30	4256.85	18316.98	4638.42	19936.67
6	3135.31	17163.35	3446.52	18814.64	3757.73	20465.94
7	2886.13	18993.23	3176.50	20806.54	3466.87	22619.84
8	2512.67	19574.49	2771.21	21431.78	3029.76	23289.07
9	1598.54	14836.94	1770.90	16236.94	1943.27	17636.95
10	684.27	8337.32	770.45	9118.24	856.64	9899.15
11	-237.60	0.00	-237.60	0.00	-237.60	0.00

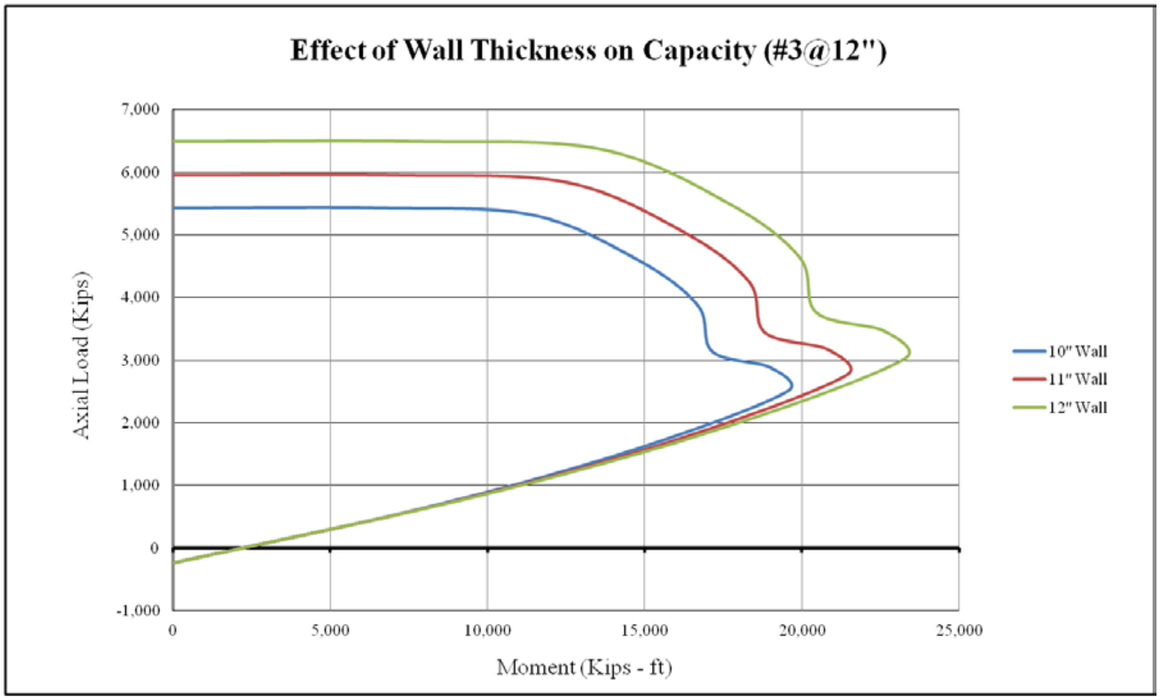


Figure 5.17: Effects of section thickness on interaction diagram

Figures in the graph above follow the logical order that was expected. Thickest wall (12″) is showing the highest capacity. The difference between maximum axial loads in 12″ wall to 10″ wall is about 20%. Maximum moment at point 8 has also increased by more than 18%. Since the reinforcement is the same in all three models (#3@12″), point 11 yields the same results for all cases. Point 11 is the resistance of section to pure tensile forces and since rebars are the only elements resisting tension, the values are the same for all three models.

By putting together the results from thickness and reinforcement increases, it is seen that the section capacity is influenced in different ways. Additional reinforcement is helping the section in tension-controlled zones, while contributes very little to the compression-controlled behavior. On the other hand, increase in thickness of the section helps capacity in compression-controlled zone, but strength of the section in tension remains exactly the same since reinforcement (the only element that resists tension) has not changed in three models.

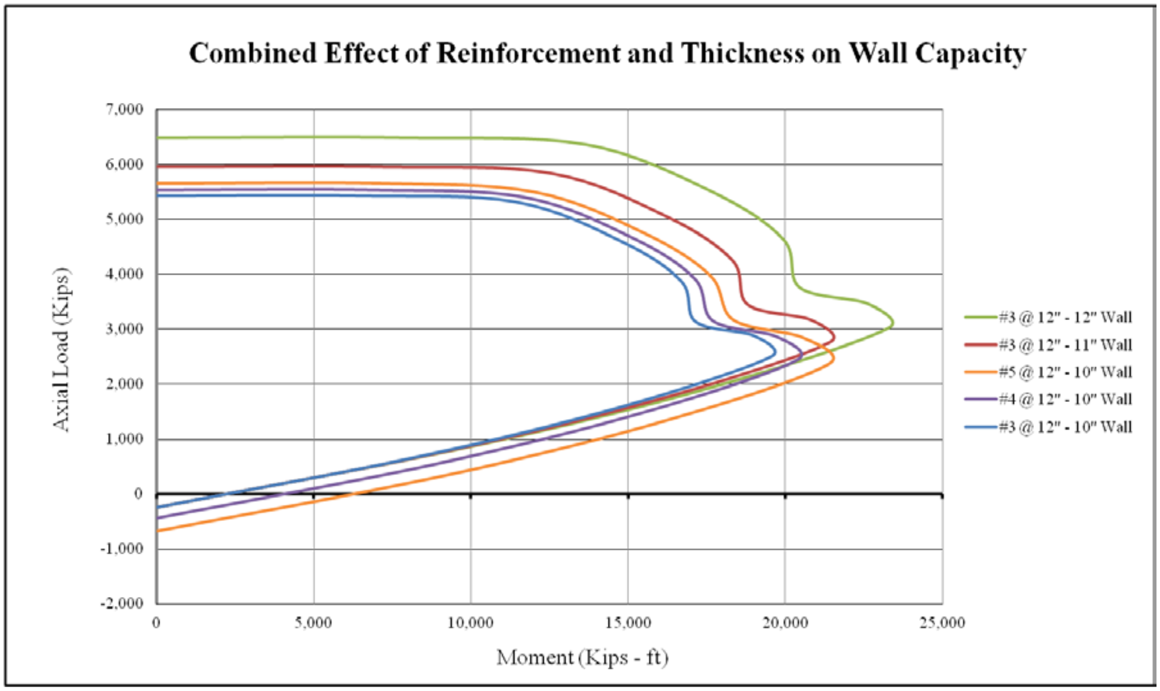


Figure 5.18: Combined comparison of vertical reinforcement and section thickness

Comparison of results and the figure above indicate that in order to be able to improve the capacity of a shear wall, it is very important to first know what behavior the wall will have under critical load combinations. If the section which is designed for the shear wall is controlled by tension under applied loads, then reinforcement can be helpful in improving the capacity. But if section acts as a compression controlled element due to applied loads (such as large axial loads) then extra thickness can contribute much more than reinforcement can.

It must also be considered that in situations where thickness can be increased, extra thickness can provide higher stiffness values which results into reduction of lateral displacements, story drifts and natural period. While increasing reinforcement does not contribute to stiffness of a structure due to the negligible influence it has on moment of inertia of a section. It is also important to include financial considerations in such decisions and try to find an optimal solution by choosing a cost effective approach.

Chapter 6

Conclusions and Recommendations

To summarize the findings, conclusions and recommendations, this chapter is divided into the following sections.

1. Shear Wall Configuration in Plan
2. Effects of Cracking on Stiffness of Shear Walls
3. Openings in Shear Walls
4. Skewed Walls
5. Flexural Capacity of Wall Sections

6.1 Shear Wall Configuration in Plan

Results from analyses of numerous models that were performed for both medium rise buildings and high-rise towers, were generally following the same pattern. In all of the models, those which had a central core showed a relatively higher stiffness, and as a result, less displacement and shorter natural period. A common practice among engineers is that if substantial torsional moments are generated for any reason, a wide distribution of walls around the perimeter of the plan can be the most efficient in resisting torsion. The reason is that torsion resisted by each wall is related to its lateral stiffness about the strong axis multiplied by the distance to center of mass, so the farther the shear wall is from center of mass the more it can resist torsion.

But based on results obtained from analyses in this study, it can be concluded that in cases where the structure is symmetric in plan and center of mass is not far away from center of rigidity (in other words, significant torsion is not generated in the structure), there is no need to insist on placing the shear walls at the perimeter of the plan. In fact, if limiting lateral displacements and higher stiffness is a priority by using the same number of shear walls, it can be a more efficient design if a central concrete core is considered for the structure. This behavior of central core and its efficiency in limiting lateral displacements is more useful in high-rise towers due to much larger lateral loads that these towers usually experience and the limits they have on lateral displacements.

Analyses and results in many cases also indicated that shear walls which act together as a uniform element, tend to show higher stiffness values. The higher moment of inertia cause by combination of different sections can help increase the lateral stiffness of a building. This way, walls that are perpendicular to direction of load can act as flanges for the walls that are parallel to loading and therefore help the structure gain higher stiffness. It also explains another reason why central core was more efficient in limiting lateral displacements relative to other models. In central core, all of the shear walls were acting together and as a result, displayed a much higher stiffness while in other models, walls were either acting individually or in smaller groups and thus could not provide the same stiffness as central core. However, it was observed that walls which act as flanges may have to resist higher loads because of the tension and compression that flanges usually take in a section. This extra load was shown in form of axial load caused by earthquake in walls that were perpendicular to direction of seismic forces.

Based on observations, it can be concluded that models with a higher stiffness may experience larger lateral loads which means, they have to be designed for higher forces. This can be considered as one of the downsides of using a very stiff lateral resisting system. Smaller natural period as a result of higher lateral stiffness, may cause the structure sustain larger forces.

Another conclusion from shear wall configuration in plan is the importance of frame contribution to shear wall-frame structures. Even though shear walls are usually stiffer at lower levels, frames tend to show higher stiffness values at top floors (depending on their relative stiffness). This behavior of frames can influence the overall stiffness of the structure and lateral displacements as a result. It is shown that in cases where frames do not have the same stiffness (which might occur in flat slab systems) stiffness of the structure can change depending on where and in which frame shear walls are located. If frames are stiffer, they can push back on the wall at higher floors and resist larger displacements while if the frames are not stiff enough, they might not be as efficient in resisting lateral displacements and helping the shear wall in top floors.

After all, positioning of shear walls in many cases might not be a choice for structural engineers and it can be dictated by architectural plans. However, in cases where different options are available, engineers must take the most advantage and based on priorities and conditions choose the best layout which satisfies the requirements properly while optimizing the design. It is important to minimize the separation between center of mass and center of rigidity by using a proper layout in order to reduce inherent torsion. It can be recommended that in situations where limiting deformations and story drifts is a priority, it can be helpful to use concrete cores to increase stiffness and reduce lateral displacement for structures. Using boundary elements or perpendicular walls as flanges can also be helpful. But it should also be considered that higher stiffness may cause the structure experience higher seismic forces and elements should be designed to resist larger loads. Therefore, it is recommended to achieve an optimized balance between the lateral stiffness which is required and loads that the structure will have to sustain. These considerations can be of different importance in different projects. If limiting lateral displacement is a priority, increasing stiffness can be helpful but if that is not a decisive factor, structures can be designed for lower seismic forces if ductility is increased.

6.2 Effects of Cracking on Stiffness of Shear Walls

Based on the results from analyses it can be concluded that cracked sections in a concrete element can have a significant influence on stiffness of a structure. Analysis showed that total amount of cracked elements in a shear wall might not be obtainable in only one round of analysis and more runs may be required for a more precise procedure. Due to redistribution of loads, cracked elements in each stage of analysis pass more loads to adjacent elements (which have not been cracked yet and have a higher moment of inertia than cracked elements). Extra load that is passed to uncracked elements might help it reach a stress higher than the rupture value and therefore an element which was not cracked in first round of analysis might crack in second run. Results show that this method of analysis will eventually stabilize and number of cracked elements will converge. After this convergence is achieved, extra rounds of analysis are not required and there will not be any new cracked elements.

It is very important to know that cracking is highly influenced by combination of loads applied to a member including axial, shear and flexural loads. Each model that represents the wall must include all of the loads that are applied to it. From the results it can be concluded that if significant cracking is observed under lateral loads, results for stiffness and displacement can be very different when cracking modifiers are applied and it cannot be ignored.

Analysis also shows that in this particular model after the first round of cracked analysis was performed, results did not change considerably in next rounds of cracked analysis. In other words, most of the elements which were cracked due to combination of loads were identified in the first round of cracked analysis and those which cracked in following steps did not contribute much to the results of analysis. However, this conclusion might not apply to all cases but for a case where precise analysis is not required, first cracked analysis might suffice.

The chapter about effects of cracking emphasizes the importance of cracking in structural analysis. It is up to the structural engineer to decide what property modifiers to use and where to apply them depending on loading criteria and precision that is required. On models where initial structural analysis does not show cracking, ACI recommends using a property modifier of 0.7Ig for all uncracked wall elements. However, cracked elements in shear walls must have a property modifier of 0.35 Ig according to ACI. If engineers decide to take a conservative approach, they can apply a 0.35Ig to all wall elements and assume the entire wall is cracked. But if more precision is required to find the cracked elements in a shear wall, a similar approach described in chapter 4 is recommended with some modifications to fit the needs of the project.

6.3 Openings in Shear Walls

Analysis results from several different models show that openings in shear walls reduce the stiffness of structure in general but this reduction can vary from negligible to significant depending on size, position and ratio of the openings. Based on results, it can be concluded that in this particular study small openings up to a ratio of 10% of the shear wall did not have a remarkable influence on lateral stiffness of the building. These openings which may be required for architectural reasons or MEP (mechanical, electrical or plumbing) purposes can be safely ignored in analysis of the structure. However, openings with higher ratios can severely reduce the stiffness of shear walls and entire building. In the case of a medium rise building with a central core, opening ratios higher than 15% displayed more undesirable effects and it may not be safe to ignore their influence. While these numbers and percentages may differ from case to case, the general concept and pattern remains the same for all other cases.

The main conclusion is that engineers may not be able to simply ignore the effects of all openings for simplicity of analysis. In some cases, based on engineering judgment, it can be assumed that openings do not affect the results of analysis because they are too small compared to the size of shear wall itself. But if openings increase in size (such as those required for large windows or elevators doors on a central core) the influence of openings must be considered in the analysis. These openings not only reduce the lateral stiffness of the entire structure, but they also have undesirable impacts on lateral displacements and story drifts as well. They can also significantly reduce the shear and flexural capacity of shear walls. It is also important to note that no matter how small an opening is, special considerations must be applied regarding the detailing around the corners and rebar formation in shear walls. Issues such as stress concentration and cracking near openings are important considerations that need to be properly addressed, otherwise they can lead to many problems.

Results in this research are representing a model with similar openings in all sides of a central core. While the concept and pattern usually stays the same, it is critical to know that other factors such as position of openings, shape of openings and un-symmetric formations can cause different results for different projects which should all be considered in a proper method.

Undesirable influence of openings on stiffness, displacements and story drifts in the structure combined with reduction in section capacity and complexity in detailing are all reasons that it is recommended for engineers to avoid openings in shear walls as much as possible. If it is possible to have a shear wall in different location of a building, it is recommended to choose the position which the least amount of openings are needed. Although other factors might be superior to this in shear wall configuration, it is still important to be aware of undesirable effects caused by openings.

6.4 Skewed Walls

Walls that are not parallel to direction of loading show much less stiffness than those that are parallel to applied loads. Conclusions in this part of the research are basically a verification of one of the most common assumptions amongst engineers. Results indicate that even a small difference between the direction of loading and strong axis of the wall can decrease the stiffness mostly due to presence of a much weaker axis. Walls are considered skewed if they are not placed in orthogonal directions and they would not provide similar stiffness of their strong axis. This is the main reasons why it is recommended to place shear walls in two orthogonal directions and avoid having skewed walls resist lateral loads. Even small angles can cause big problems when skewed walls are expected to resist lateral forces. It is also recommended to ignore the stiffness of shear walls that are perpendicular to direction of loading simply because their negligible stiffness about weak axis does not contribute to lateral stiffness of the structure. Ignoring this stiffness can also help computer models run the analysis relatively faster.

6.5 Flexural Capacity of Wall Sections

Thickness of a section and vertical reinforcement play a critical role in flexural capacity of a shear wall. Since shear wall design in buildings over three or four stories is usually governed by flexural forces, it is important to know which factors can help the section resist higher loads in flexure. However, one of the conclusions from this chapter indicates that these two parameters (thickness and vertical reinforcement) can affect the capacity of a shear wall in two different ways. It is recommended that prior to making decisions on how to increase the flexural capacity of a shear wall, it is verified whether the section design is being controlled by compression forces or tension forces caused by combination of loads. In other words, knowing that the section is tension controlled or compression controlled can be decisive.

It is concluded that an increase in thickness of the shear wall can be very helpful when compression is the controlling force due to combination of axial and flexural forces. Besides the contribution that increased thickness can have on the capacity of a section (in both flexure and shear) it can also help the stiffness of a structure by increasing the moment of inertia about strong axis of the wall. However, increasing the thickness of the wall may not be as effective when the section is controlled in tension. Increasing the reinforcement ratio on the other hand may help a section which is tension controlled more effectively.

In most cases, structural engineers might not always be free to decide between several options because of limitations mostly from architectural plans, but by knowing how to efficiently increase the capacity of a section, they can reach the desired strength while optimizing the design and addressing issues with a smarter approach.

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