COMPARISON OF LATERAL FORCE RESISTING SYSTEMS WITH & WITHOUT RETROFITTED PROVISIONS

FINAL YEAR PROJECT UG 2019

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This is to certify that the

Final Year Project Titled

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the undergraduate degree

in

CIVIL ENGINEERING

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ABSTRACT

Comparison of different Lateral Force Resisting System with or without braces was done using different tools which includes both physical and software testing of the model. Braces were provided at different orientation in a frame. Empirical results were obtained using Shake Table. Software results were obtained using Etabs and Mastan2. Etabs was used for linear elastic modelling and Mastan2 was used for non-linear elastic modelling. Both empirical and software results were compared and performance of the structure using a brace was checked. It was concluded that using a brace in a structure improves its performance a lot and the structure was less prone to damage by earthquake loading. Furthermore, it was checked that which type of brace perform best in case of an earthquake.

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CHAPTER NO. 1

INTRODUCTION

1.1 General

How an earthquake takes place? Earthquake take place by the movement along the fault line, releasing a large amount of energy having a certain time period and frequency. The building has their natural time period and frequency. When the time period of earthquake and that of building becomes same, the phenomena of resonance occur causing collapse to the building.

Earthquakes are cause of damage to a majority of building and loss to human life. Recently, during the 2005 Kashmir Earthquake, losses were enormous in terms of human lives and financial setback. More than 87,000 people lost their lives and 138,000 people were seriously injured (EERI, 2005). Approximately 3.5 million people became homeless, bereaving them of food and shelter (ERRA, 2007). Over 270,000 buildings were collapsed and over 180,000 buildings were severely damaged. In summary, economic loss of over five billion dollars was suffered due to this earthquake (Durrani et al., 2005). This is one of those natural calamities which cannot be predicted and hence, preparing for it is the best course of action.

RC frame structures are being widely used for constructing a building in a seismic zone. New building codes are being developed and implantation of these codes have led to the better performance of a structure in a seismic area. However, it was not like this in the 20th century. Implementation of these building codes were not followed especially in developing countries like Pakistan. So, most of the structures that were built during that era were not up to the standard. These structures are prone to collapse under earthquake. So, there is need to rehabilitate these structures and retrofit so that it can work better under seismic loading. There are different methods for improving the performance of these structures. Jacketing of column and beam joint using steel and fiber reinforced polymer, Bracing and shear wall are some of the procedure to rehabilitate these structures.

Moreover, to check the dynamic response of a building, shake table testing and pushover analysis are used. It is very important to find the dynamic response as it the most important cause to the damage to a building during an earthquake. Make a full building and testing it on shake table is very costly and nearly impossible, so there is a need to scale down the structure and then, check out its response against acceleration and displacement on a shake table.

1.2 Statement of Purpose

Every year earthquake is a primary cause of loss of thousands of lives as well as huge loss to property. So, there is need to take precautionary measures in high seismic areas along with retrofitting already built buildings so that there is reduction in the number of casualties.

The purpose of this project is creating a small-scale model and then testing it on a shake table as well as software. Moreover, to compare ordinary moment resisting frame with brace with special moment resisting frame using different parameters like displacement, story shear etc.

1.3 Objectives

- To create a small-scale model for testing on shake table
- To compare OMRF with brace and SMRF.
- To analyze a structure the performance of a structure using different types of bracing.

1.4 Scope

This study is limited to:

- Design of two-story frame, preparation of its small-scale model and then testing it on shake table as well as pushover analysis. Comparison of SMRF and OMRF with brace.
- Prediction of behavior of the structure using braces on software.
- The extent to which parameters of dynamic response as well as static response could be reduced using bracing.
- Comparison of different types of bracing.

CHAPTER NO. 2

LITERATURE REVIEW

2.1 Moment Frames

A fundamental structure in engineering – the frame – is a two-dimensional series of interconnected members joined together. The members are not necessarily straight and may be free jointed anywhere along their length. In real structures, moment frames in two orthogonal directions are often connected together to form a three-dimensional frame of columns and beams. Moment frames are designed to carry vertical and horizontal loads in the same plane but may also be drawn on to provide resistance to horizontal loads out of the plane of the frame. A moment frame is a special type of frame that uses rigid connections between each of its constituent members. This configuration is able to resist lateral and overturning forces because of the bending moment and shear strength that is inherent in its members and the connecting joints. Therefore, the stiffness and strength of the moment frame in seismic design depends on the stiffness and strength of its members. Because moment frames can be more flexible than other options, such as shear walls, they allow larger movements during an earthquake. Non-flexible elements attached to the frame, such as the cladding, must be designed to accommodate the additional movement to avoid damage.

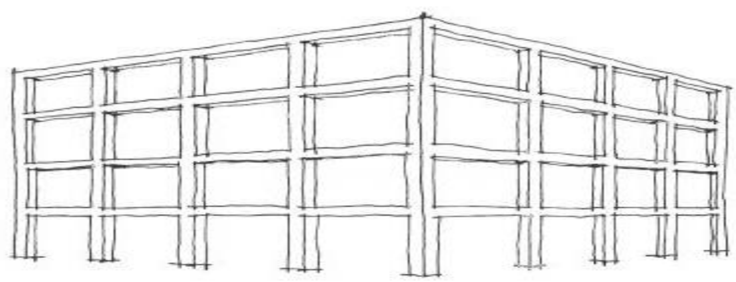


Figure 2.1: Moment resisting frame

Figure 2.1: Concrete Moment Frames Consisting series of Columns and Beams

2.1.1 Ordinary Moment Resisting Frames

OMRF is a moment resisting frame not meeting special detailing requirements for ductile behavior. OMRF’s are expected to withstand limited inelastic deformations. They are stiffer and less ductile. They require comparatively less reinforcement details. OMRF is generally used in zones with low seismic activity. OMRF is stiffer > attracts higher base shear (seismic force) > less capable to redistribute forces from member to joint and joint to member due to its limitations of detailing. SMRF are detailed aiming ductile behavior, they respond in better manner. Differing, SMRF is lesser stiff > attracts lesser base shear > more capable to redistribute forces from member to joint and joint to member due to its special ductile detailing. To calculate Base shear for OMRF you are advised to use Response reduction factor as 3 and 5 respectively, in formula which results in higher base shear for OMRF and lesser for SMRF. By this way you can imagine that how response reduction factors would have been derived. One should comprehend in such way that; you should select Response Reduction Factor for the Behavior of Structure that you want rather predicting Behavior of Structure by uncertainly picking any Response Reduction Factor.

2.1.2 Special Moment Resisting Frames

SMRF is designed with more rebar and stirrups detailing than OMRF. SMRF’s are capable of withstanding significant inelastic deformations. SMRF’s offer much more ductility. They have rigorous reinforcement detailing and proportioning. Generally used in zones with high seismic activity.

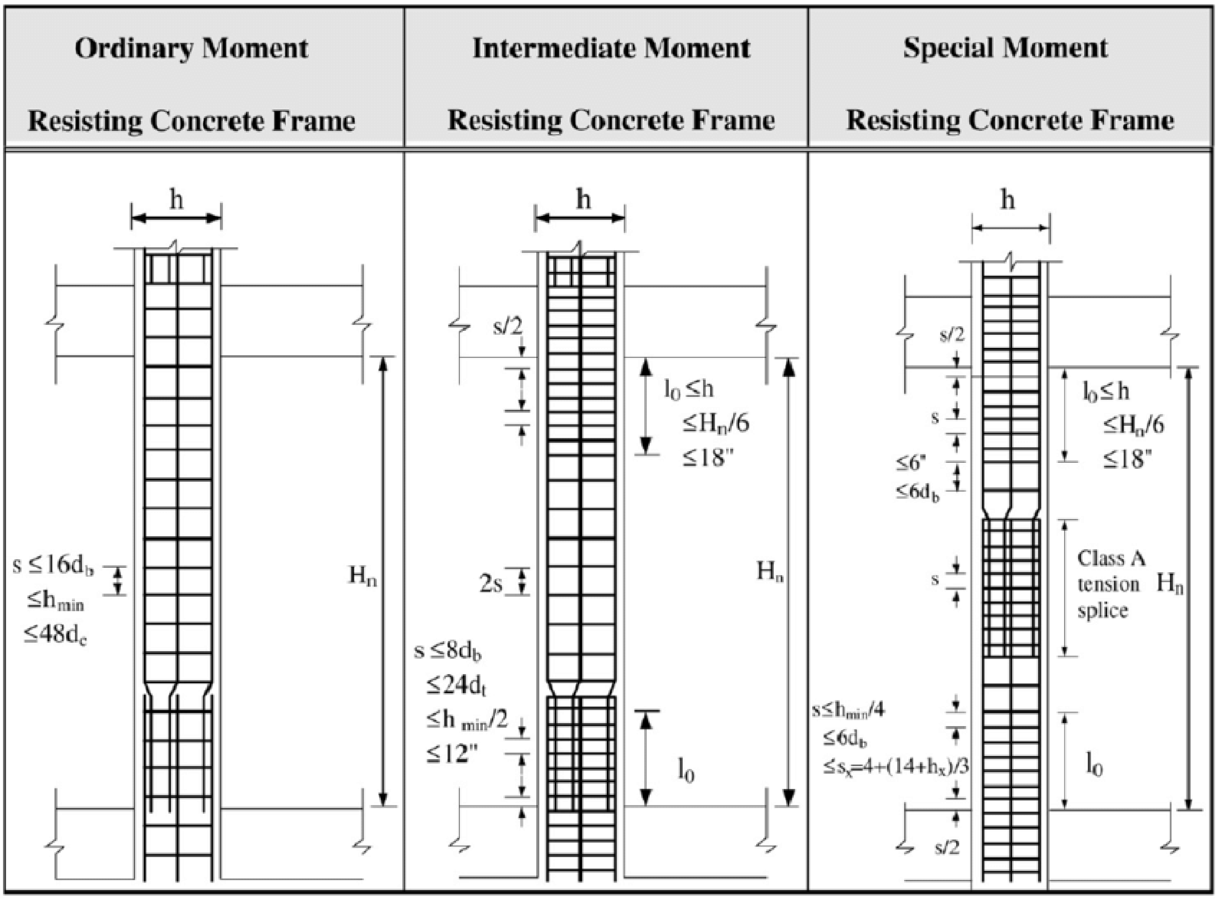


Figure 2.2: Column Detailing of Different Moment Frames

2.1.3 Plastic Hinge:

A plastic hinge, in structural engineering, refers to the deformation of a part of a beam wherever plastic bending happens. Hinge means that having no capability to resist moment. Therefore, a plastic hinge behaves like a standard hinge - permitting free rotation. The concept of plastic hinge is important in understanding structural failure.

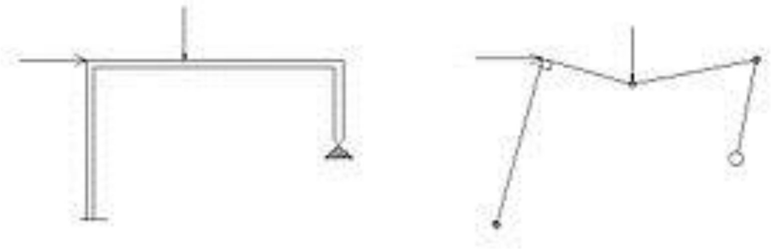


Figure 2.3: Diagram of Structure Featuring Plastic Hinge

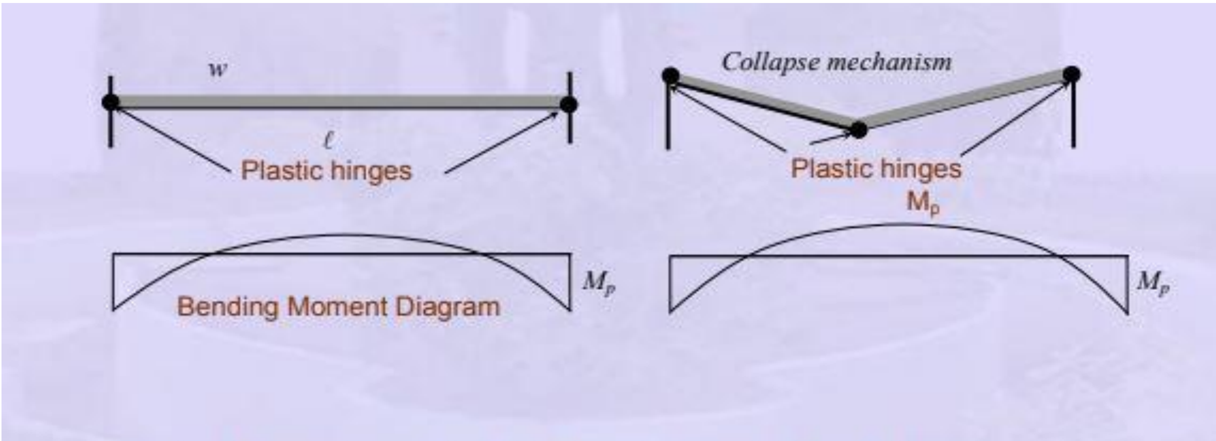


Figure 2.5: Plastic Hinge

2.2 Braced Frames:

Another fundamental concept in engineering – bracing – involves added additional elements to a frame in order to increase its ability to withstand lateral loads . There are two main varieties of braced frames – concentric and eccentric.

2.2.1 Concentric bracing

Concentric bracing consists of diagonal braces located in the plane of the frame. Both ends of the brace join at the end points of other framing members to form a truss, creating a stiff frame.

Concentric bracing may be arranged in several different configurations – such as X, K or one-directional diagonal bracing – and the bracing members may be designed to act in tension or compression or both. Balanced diagonal bracing is the most common for medium-rise structures because it provides the same strength in both directions. Efficient energy dissipation is difficult to achieve in concentrically braced frames.

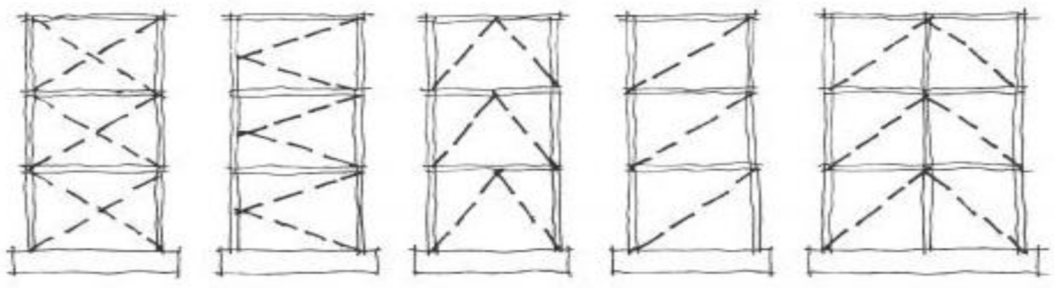


Figure 2.6: Common Types of Concentric Bracing

2.2.2 Eccentric bracing

Eccentric bracing consists of diagonal braces located in the plane of the frame where one or both ends of the brace do not join at the end points of other framing members. The system essentially combines the features of a moment frame and a concentrically braced frame, while minimizing the disadvantages of each system.

The eccentric connection to the frame means an eccentric brace transfers lateral forces via shear either to another brace or to a vertical column. When properly proportioned, eccentric braced frames may exhibit a more ductile characteristic and greater energy dissipation capabilities than a concentric braced frame in the same material.

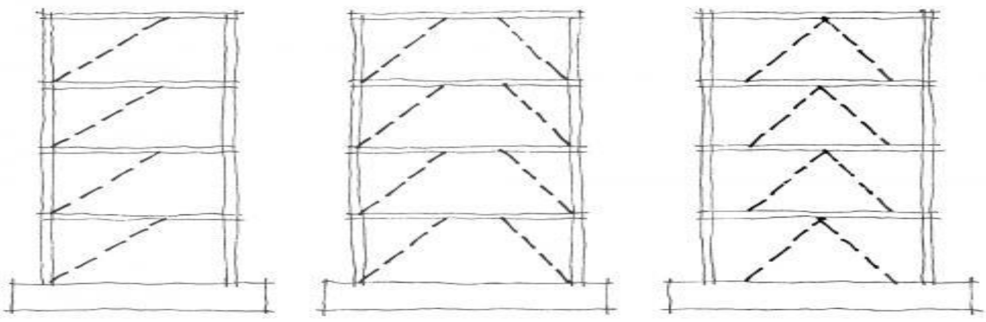


Figure 2.7: Common Types of Eccentric Bracing

To provide bracing to a structure we must know that how we are going to make a connection between a brace and a concrete structure. Different methods for a cross bracing to make connection are described below in the Figure:

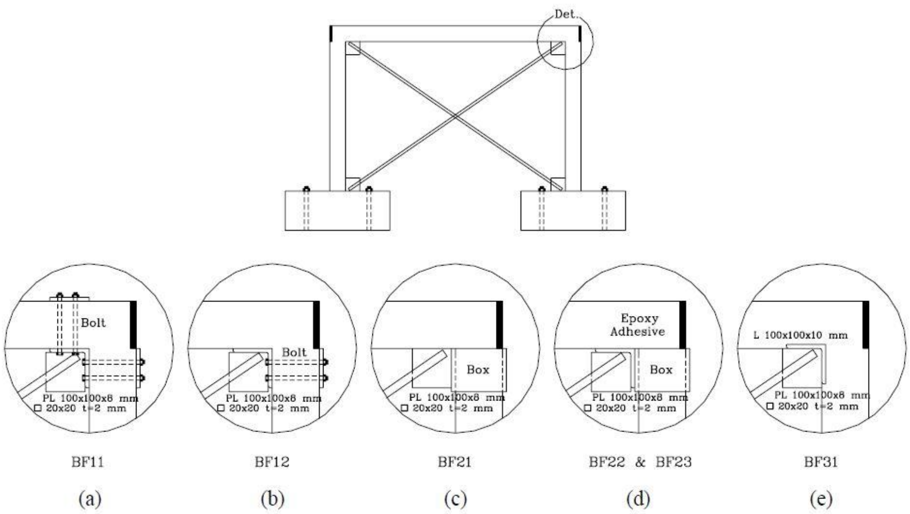


Figure 2.8: Connection details

2.3 Small Scale Modelling

The study of dynamic structural response of full-scale reinforced concrete structures subjected to earthquake loadings requires testing facilities with extremely high load capacities, and is possible at only a few highly-specialized laboratories. The cost of using these facilities, and of building and disposing of the test specimens, is very high. For this reason, small-scale structural models (at geometric scale factors in the range of 1/6 to 1/10) offer an attractive means to perform dynamic loading experiments without incurring the high costs of full-scale testing. When a prototype reinforced concrete system is modeled for strength, it is necessary to reproduce all significal1c physical characteristics on a one-one basis. Any distortion of similitude must be understood and its effects must be predictable. These distortions, which result in the so-called "scale effects," must be minimized through application of the very best modeling techniques and practices. The model concrete mix should be proportioned to match the compressive stress-strain characteristics of the prototype concrete while minimizing the overly high tensile strengths so often found in model concretes. Model reinforcement should have a stress-strain curve identical to that of the prototype reinforcement, including the strain-hardening region. Furthermore, bond behavior, which is the single most important measure of the composite action between the concrete and reinforcement, should be similar (if not identical) in prototype and model. To scale down a RC structure we have to scale down both concrete and reinforcement detailing.

2.3.1 Model Concrete

One of the most difficult steps in small scale modeling is the selection of model concrete. Accurate duplication of the prototype concrete properties is required if the model is to simulate the whole range of behavior of the structural system as it is loaded to failure. It is generally required that a model concrete have specific values of four properties under short-term load:

- Ultimate compressive strength, f c
- Tangent or secant modulus of elasticity, E
- Ultimate compressive strain eu
- Ultimate tensile strength, f t

Various studies using micro concrete (defined here as concrete made from Portland cement, water, and sand without coarse aggregate) have shown that reasonably adequate results can be obtained if the material is controlled properly. Thus, micro concrete is the logical choice as a concrete substitute in small scale models. Other cementitious materials such as gypsum have also been used in model concretes with reasonable success.

The micro-concrete used only sand and cement without any gravel. In order to get variously graded sands for the micro-concrete, the sand was divided into two parts; one had particles larger than #8 sieve size and smaller than #4 sieve size (called model gravel) and denoted by Gm in this study. The other fraction had particles smaller than #8 sieve size (called model sand) and denoted by Sm. Sands having different gradation curves were made by recombining the model sand and the model gravel with different mix ratio. For Micro-concrete I and II, the original sand was used with a sand to cement ratio of 3 and a sand to gravel ratio of 4. In Micro-concrete II the coarse particles corresponding to the model gravel size were coated with polystylene. This was done to reduce model concrete tensile strength by reducing the bond strength between the cement paste and the coarse aggregate. One-eighth diameter, high polymer polystylene pellets were added to commercial grade toluene to give a 10% solution by weight. The solution was kept in a sealed container to prevent evaporation of the toluene. The model gravel to be coated was thoroughly washed, then oven dried at 110°C for one day to remove the hydroscopic moisture, and then allowed to cool. The model gravel was completely submerged in the polystylene solution two times, being allowed to drain and dry completely between each application. Then the coated model gravel was mixed with the model sand to make Micro-concrete II. The aggregate used for Micro-concrete III consisted of the model sand and gravel in a mix ratio of 3 to 3 in order to increase the portion of large particles. To further increase the portion of large particles, the model sand and gravel were mixed in a ratio of 2 to 4, in Micro-concrete IV.

In different concrete cylinder test, it was seen that Micro-concrete III gives results close to Prototype concrete so we should use Micro-concrete III for our small-scale modelling. Results for all types concrete cylinders are given below in the figure:

2.3.2 Model Reinforcement

One of the main objectives of the present work was to reproduce the prototype structure response at various stages of loading up to failure at model scale. The considered range of loading covers the elastic, inelastic, and the ultimate stages of behavior. Since most reinforced concrete elements are usually under- reinforced to provide sufficient ductility and to achieve an economical use of steel reinforcement, the post-yield stress-strain characteristics of both the prototype and model reinforcement are critical in determining the structural behavior in the inelastic cracked range. Another important aspect of the selection of model reinforcement is the proper representation of bond. Various techniques have been proposed by model investigators to improve the bond characteristics of model reinforcement for best cracking similitude. Plain wires with rusted surfaces, cold-rolled threaded rods, deformed wires, etc. have been examined by many researchers. However, a definitive solution of the model reinforcement problem, including bond, is not yet available. The choice of bar diameter was based on a l/6 scale replica of the prototype reinforcement. The exact required diameters were almost impossible to find in the market, but every attempt was made to obtain model bars with diameters as close as possible to the required sizes. In some cases, such as when threaded bars were used, it was necessary to use a combination of small and large diameter wires. This was done on the expense of slightly distorting the exact reproduction of the total surface area of the prototype bars. The cross-sectional area of the knurled wires was obtained as follows:

Area = Weight of wire / (Density x Length)

Several forms of surface deformations are examined in the present investigation to obtain the best correlation between model and prototype cracking patterns. Four types of wires were used: (1) plain wires with no surface deformations, (2) threaded rods, (3) commercially deformed wires, and (4) standard deformed wires.

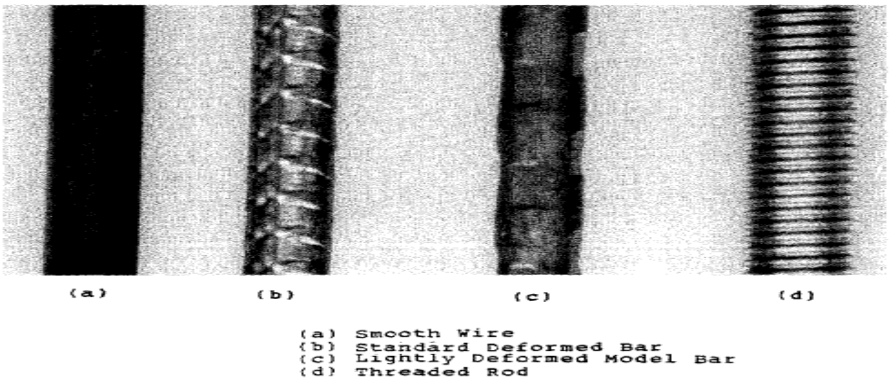


Figure 2.9: Different types of model reinforcements considered

2.3.3 Heat Treatment of Reinforcement

Heat treatment seems to be an essential process for proper simulation of reinforcing steel. Model bars will rarely have either the desired yield strength or sufficient ductility (yield plateau). Also, when smooth bars are cold-formed to produce the required surface deformation, their yield strength increases while their ductility decreases. This can be attributed to the state of high internal strain produced by cold-forming. Heat treatment or annealing of model bars is used to control the yield strength, and to improve the yield and post-yield characteristics, such as developing a clear sharp yielding point and increasing the ductility.

CHAPTER NO. 3

METHODOLOGY

3.1 General

The methodology used for this project consists of two parts, Empirical Testing and Software Analysis. Data extracted from empirical testing was used in the software analysis. Empirical testing involved testing compressive strength test for concrete and tensile strength for steel bars. For concrete mix cement and coarse sand was used with additive of plasticizer. Rebar used was wires of diameter 2mm. Details of these test is as follows

3.2 Empirical Testing

3.2.1 Concrete mix Design

Cement sand mortar was used for casting the cubes. 1:1.5 ratio was used for cement sand with a water cement ratio of 0.33. Plasticizer was used to help the self-setting of the paste, to eliminate the need of tamping. Plasticizer was added at the rate of 1% of the total mass of the dry mix. Hobart mixer was used for the mixing of the mortar.

3.2.2 Compressive Strength for Concrete

For finding the compressive strength of concrete, there cubes of size 50mm X 50mm were casted. Two cubes were cured for 14 days and one was cured for 28 days. After curing cubes were air dried and then tested using Compression testing machine using ASTM C39. The results of these test are provided in the chapter of results.

Figure 3.1: Concrete cube after compressive strength test

3.2.3 Tensile Strength Test for Rebar

Steel wires of 2mm diameter were used as rebar in our project. Preferred rebars for our project were supposed to be corrugated, because of the unavailability of the corrugated wires smooth wires were used. Ultimate tensile strength of these wires was found using Universal Testing Machine. UTM is not able to hold the wires of size less 0.375 inches (9.525mm). Due to this fact we were unable to test the wires in normal way using UTM. For adjusting the wires of the 2mm in UTM's gauge a bundle of wires was prepared combining 6 to 8 bars of 2mm dia. Wires were welded together at the ends to hold them in place for placing them in UTM.

Figure 3.2: Model reinforcement undergoing tensile strength test

3.3 Software Analysis

A representative frame was taken from a research paper, Seismic fragility functions for code compliant and non-compliant RC SMRF structures in Pakistan. The frame was modeled in ETABs. Response spectrum analysis and non-linear static pushover analysis was performed on this frame twice, with and without steel brace. Second order in elastic analysis was performed using MASTAN2. For second order inelastic analysis steel frames were used in MASTAN2 because this software cannot model concrete frames.

3.3.1 ETABs Analysis

ETABs 15 was used for the modeling and analysis of the concrete frame. Response spectrum analysis and static pushover analysis were performed on the frame, provided with and without retrofitted provisions. Results were compared by plotting different strength perimeters. Three different arrangements of the steel braces were used for comparison, namely X-Brace or Diagonal Brace, Inverted V Brace and Eccentric Brace.

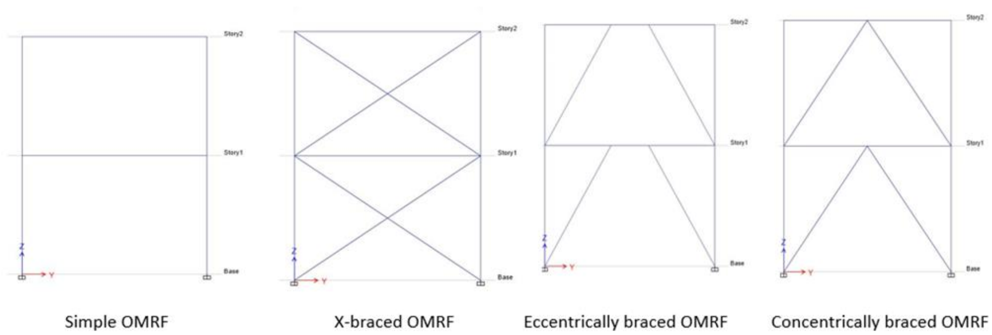


Figure 3.3: ETABs models with different types of bracing

3.3.1.1 Representative Frame

Following beam column assembly was used in ETABs. Columns were 12 feet tall supporting a beam 18 feet length. Square columns consisted of 8#8 bars with stirrups of #3 bars at 6 inches center to center spacing. Beam section was 1 foot wide and 1.5 feet deep with 5#6 bars that run throughout the length of the beam and one #6 cutoff bar, with ties placed at 6 inches center to center spacing at mid-section of the beam. Ties spacing was reduced to 3 inches center to center close to the supports for a length of L/6 of the beam.

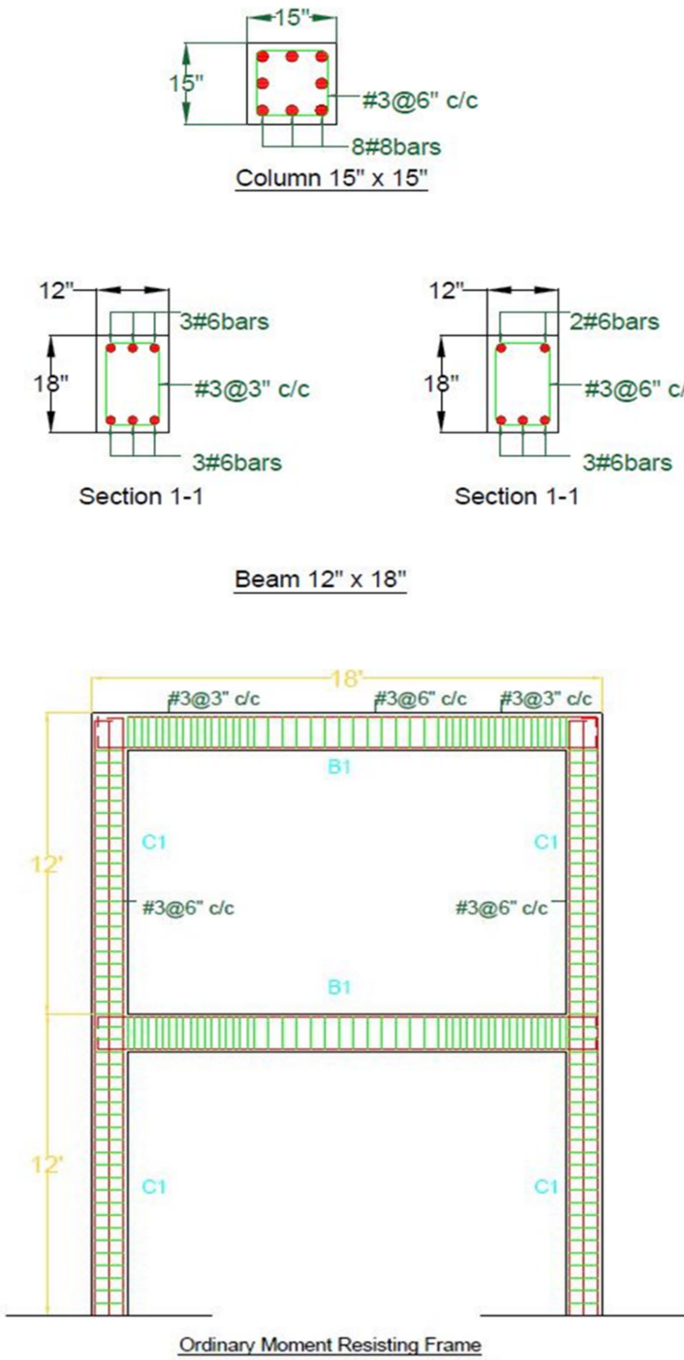


Figure 3.4: Model details

3.3.1.2 Response Spectrum Analysis

Response-spectrum analysis (RSA) is a linear-dynamic statistical analysis method which measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure. Response-spectrum analysis provides insight into dynamic behavior by measuring pseudo-spectral acceleration, velocity, or displacement as a function of structural period for a given time history and level of damping. Response-spectrum analysis is useful for design decision-making because it relates structural type-selection to dynamic performance. Structures of shorter period experience greater acceleration, whereas those of longer period experience greater displacement. Structural performance objectives should be taken into account during preliminary design and response-spectrum analysis. We used response spectrum analysis for acquiring the plots of Storey Shear, Max Storey Displacement, Max Storey Drifts and Over turning Moments. Results were obtained by comparing these plots for different arrangements of braces.

3.3.1.3 Static Pushover Analysis

Pushover is a static-nonlinear analysis method where a structure is subjected to gravity loading and a monotonic displacement-controlled lateral load pattern which continuously increases through elastic and inelastic behavior until an ultimate condition is reached. Lateral load may represent the range of base shear induced by earthquake loading, and its configuration may be proportional to the distribution of mass along building height, mode shapes, or another practical means.

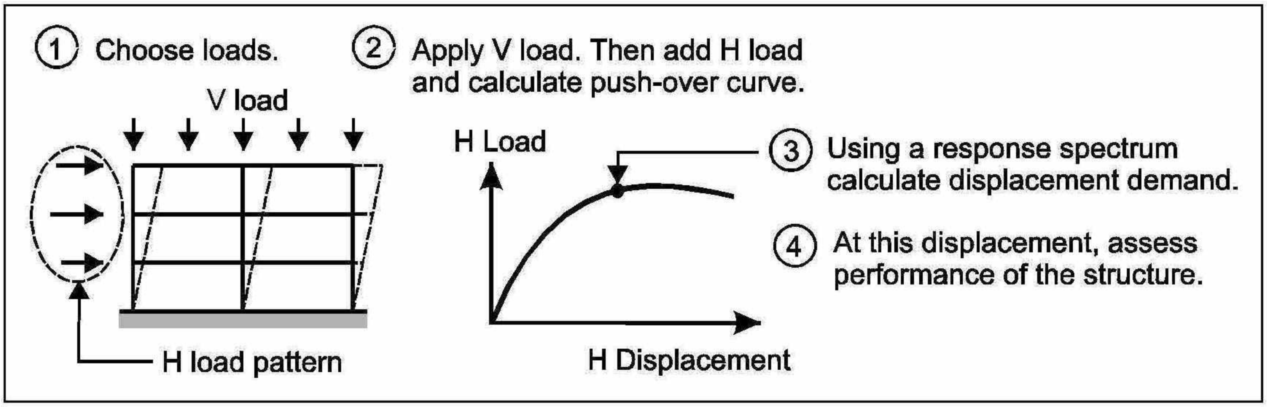


Figure 3.5: How RSA works

"Output generates a static-pushover curve which plots a strength-based parameter against deflection. For example, performance may relate the strength level achieved in certain members to the lateral displacement at the top of the structure, or bending moment may be plotted against plastic rotation. Results provide insight into the ductile capacity of the structural system, and indicate the mechanism, load level, and deflection at which failure occurs.

When analyzing frame objects, material nonlinearity is assigned to discrete hinge locations where plastic rotation occurs according to FEMA-356 or another set of code-based or user-defined criteria. Strength drop, displacement control, and all other nonlinear software features, including link assignment, P-Delta effect, and staged construction, are available during static-pushover analysis.

3.3.2 Mastan2 Analysis

Mastan2 is a software used Finite Element Modeling of the structures. In our case we use this software to perform second order inelastic analysis for steel structure. Because mastan2 does not have the option of modeling concrete structures we have to switch to steel. Mastan2 does provides better results of second order in elastic analysis as compared to the ETABs, because in mastan2 you can provide for material as well as geometric nonlinearity of the structure. The steel frame we used in our project have same configuration of 12 feet tall column and 18 feet long beam. The arrangement of the braces is also similar as it was in the case of RC frame. The sections that we used are, Beams-W12x14, Columns-W21x93 and Braces- HSS10X0.188.

3.3.2.1 Second Order Inelastic Analysis

This analysis method computes the load considering the stability of the structural frame. Second order analysis is used in the analysis where the lateral sway caused by secondary moments are important. Second order analysis will be called as P-delta analysis.

Second order effects are of two types as follows,

1) P- Δ effect is used when the displacement of the joint in the structure occurs.

2) P- δ effect is used when the deformation in the member of the structure occurs.

In the elastic analysis, the structural elements are considered as linear elastic and the stress corresponds to the strain of 0.002 will be a limiting value. Second order effects or P delta effect is used in those structures in which the structural elements are subjected under the effect of external compressive loads. The figure given below shows the deformation in frames due to sway.

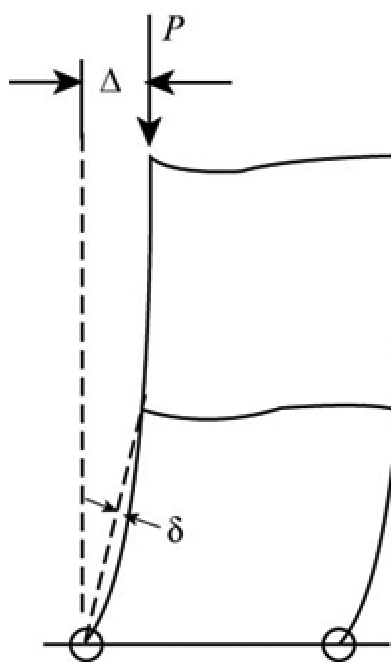


Figure 3.6: P-Δ effect

Buckling is the effect which occurs in the structural members, when they are subjected to the external effects with loading parallel to longitudinal or axial direction. The sideway deflection is due to column buckling. The structural failure occurs in critical condition, when the applied load exceeds the critical load. The secondary effects result due to the eccentricity in loading that exists in steel columns.

CHAPTER NO. 4

ANALYTICAL RESULTS

4.1 Introduction

Two software programs were used for the analysis, ETABs and MASTAN2. ETABs was used for the analysis of RC moment resisting frame where as MASTAN2 was used for the analysis of steel moment resisting frame. The analytical results have been discussed in this chapter.

4.2 Analysis on ETABs using Response Spectrum Analysis

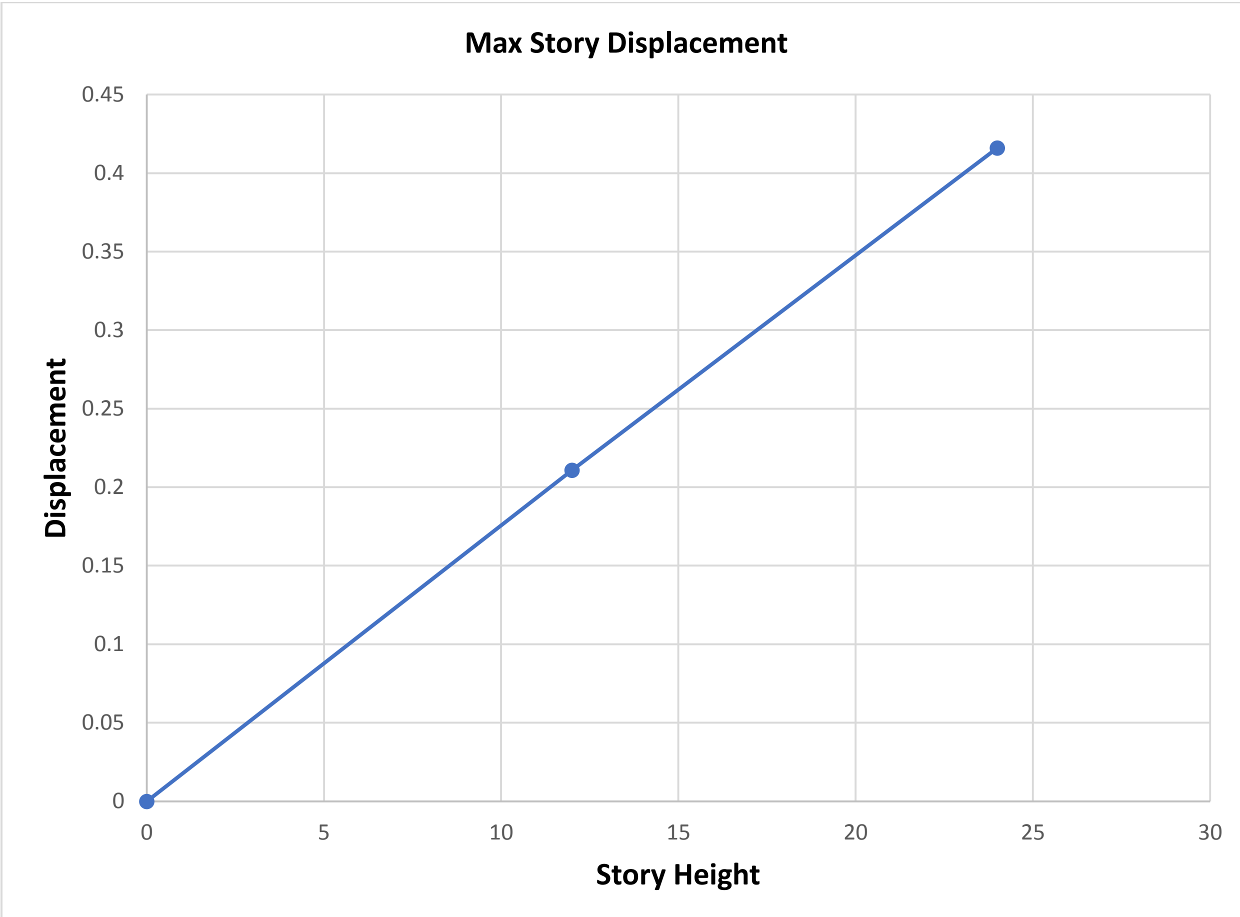
4.2.1 Ordinary Moment Resisting Frame

For response spectrum analysis of the OMRF, following results were obtained.

4.2.1.1 Max Story Displacement

	Story	Elevation	Location	X-Dir	Y-Dir
Simple OMRF					
		ft		**in**	**in**
	Story 2	24	Top	0.416117	0
	Story 1	12	Top	0.210892	0
	Base	0	Top	0	0

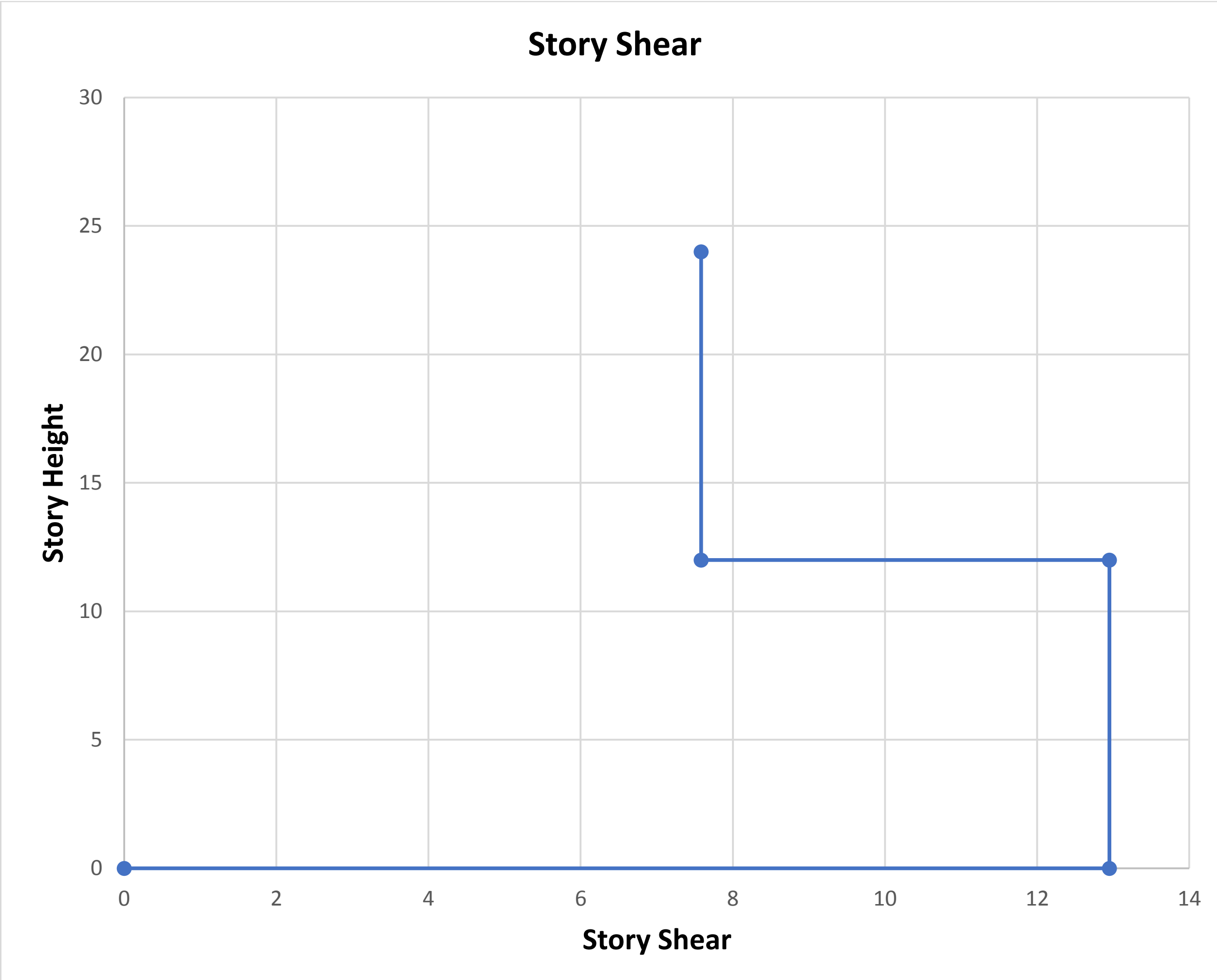
Table 4.1: Max Story Displacements for OMRF



4.2.1.2 Story Shear

	Storey	Elevation	Location	X-Dir	Y-Dir
Simple frame					
		ft		**kip**	**kip**
	Storey2	24	Top	7.585	0
		12	Bottom	7.585	0
	Storey1	12	Top	12.95	0
		0	Bottom	12.95	0
	Base	0	Top	0	0
			Bottom	0	0

Table 4.2: Story Shear for OMRF



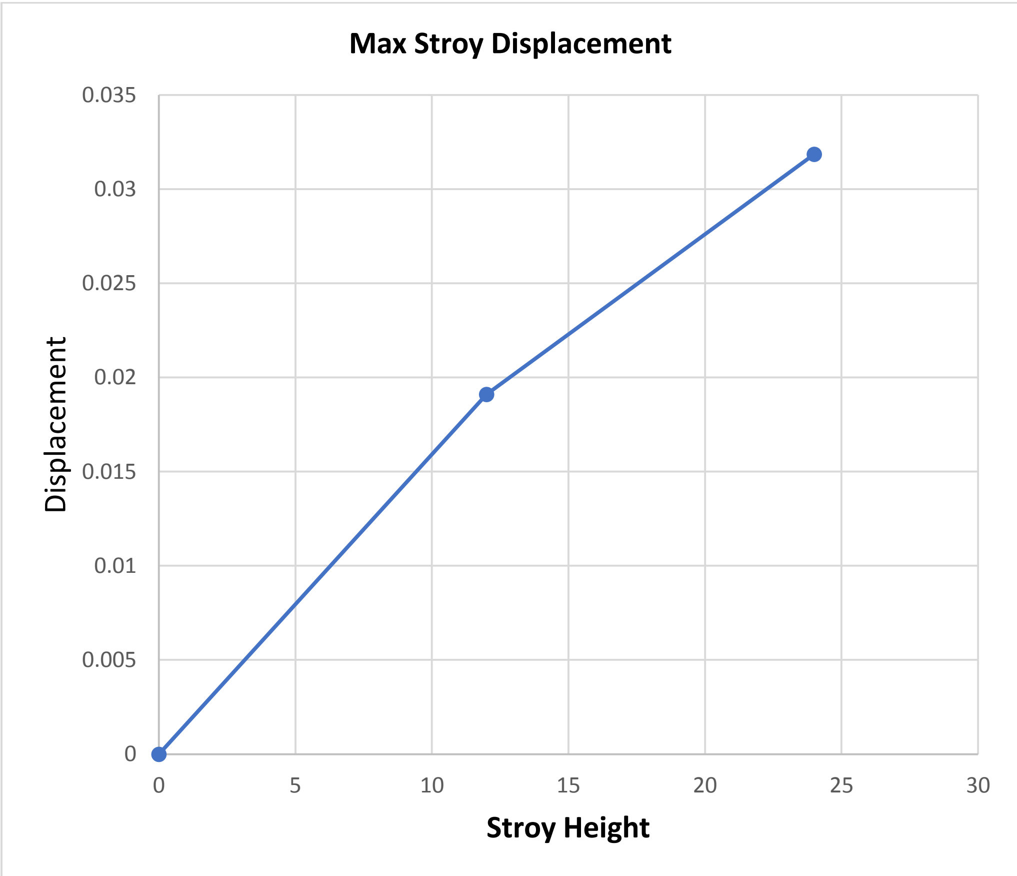
4.2.2 X-braced OMRF

For response spectrum analysis of the X-braced OMRF, following results were obtained.

4.2.2.1 Max Story Displacement

	Story	Elevation	Location	X-Dir	Y-Dir
X-Brace					
		ft		**in**	**in**
	Story 2	24	Top	0.03186	0
	Story 1	12	Top	0.019104	0
	Base	0	Top	0	0

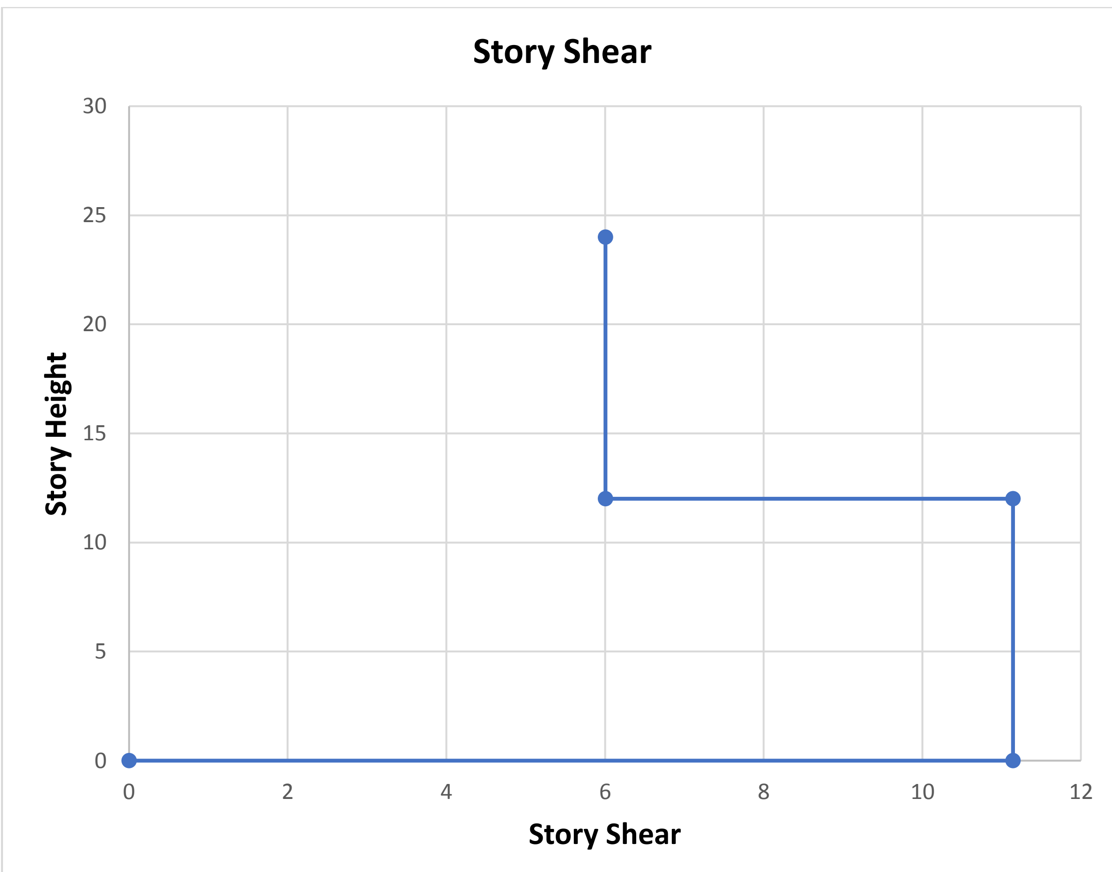
Table 4.3: Max Story Displacements for X-braced OMRF



4.2.2.2 Story Shear

	Storey	Elevation	Location	X-Dir	Y-Dir
X-Brace					
		ft		kip	kip
	Storey2	24	Top	6.007	0
		12	Bottom	6.007	0
	Storey1	12	Top	11.143	0
		0	Bottom	11.143	0
	Base	0	Top	0	0
			Bottom	0	0

Table 4.4: Story Shear for X-braced OMRF



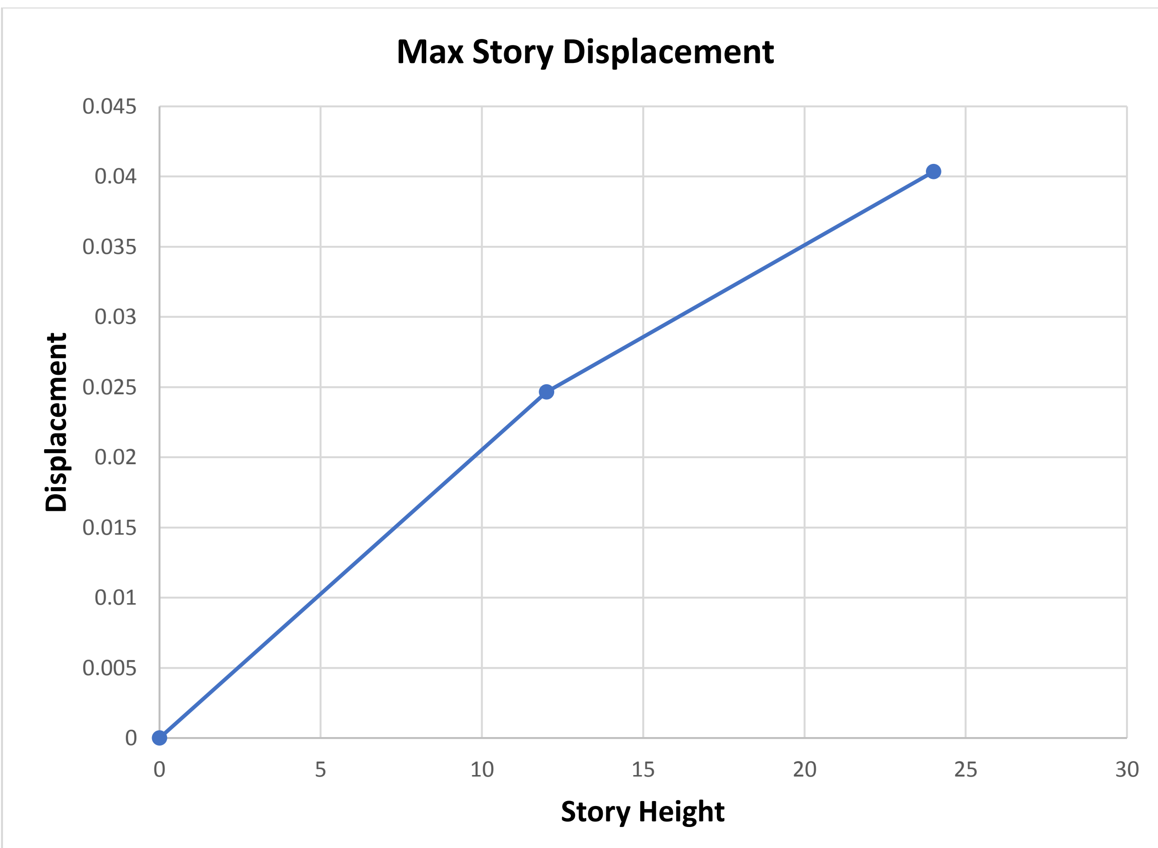
4.2.3 Inverted V- braced OMRF

For response spectrum analysis of the inv v-braced OMRF, following results were obtained.

4.2.3.1 Max Story Displacement

	Storey	Elevation	Location	X-Dir	Y-Dir
Inverted V					
		ft		**in**	**in**
	Storey 2	24	Top	0.040346	0
	Storey 1	12	Top	0.024653	0
	Base	0	Top	0	0

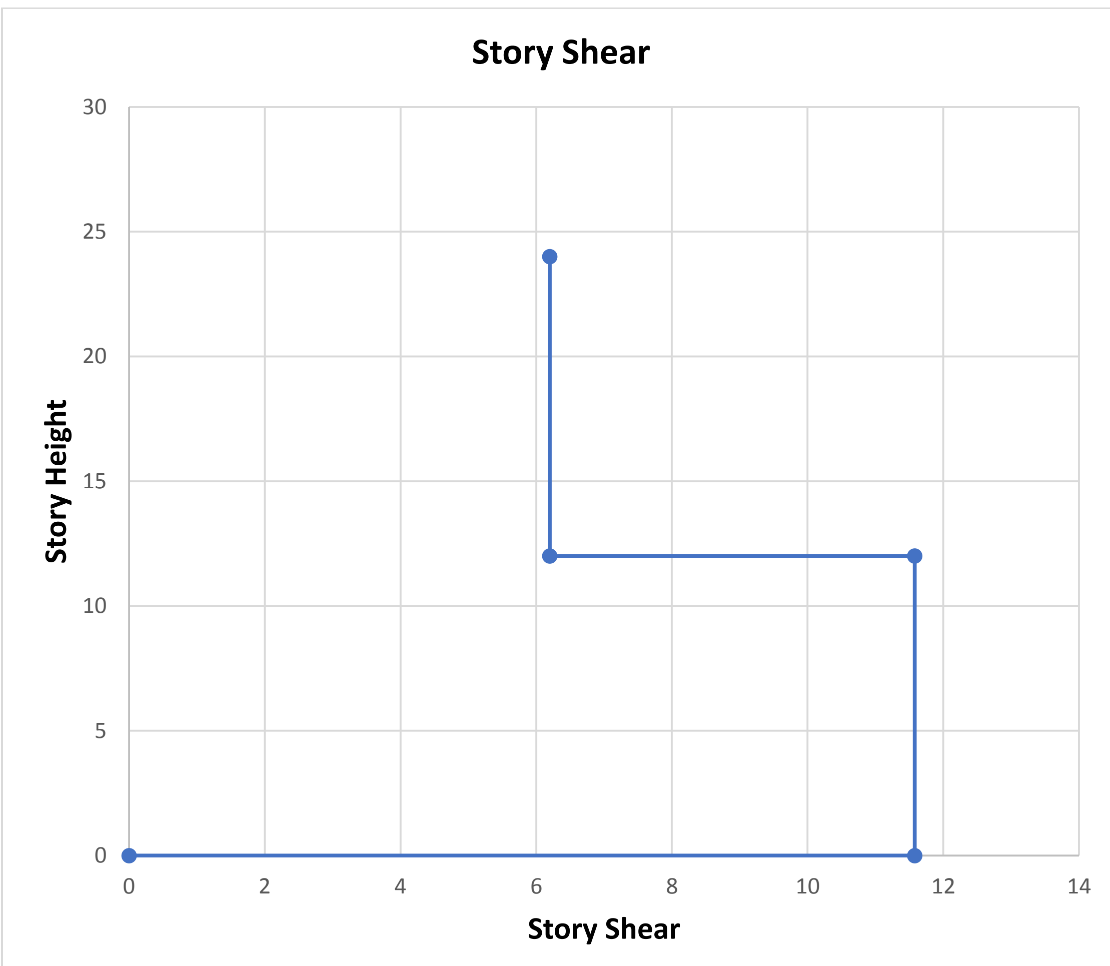
Table 4.5: Max Story Displacements for Inverted V-braced OMRF



4.2.3.2 Story Shear

	Storey	Elevation	Location	X-Dir	Y-Dir
Inverted V Brace					
		ft		kip	kip
	Storey2	24	Top	6.199	0
		12	Bottom	6.199	0
	Storey1	12	Top	11.578	0
		0	Bottom	11.578	0
	Base	0	Top	0	0
			Bottom	0	0

Table 4.6: Story Shear for Inverted V-braced OMRF



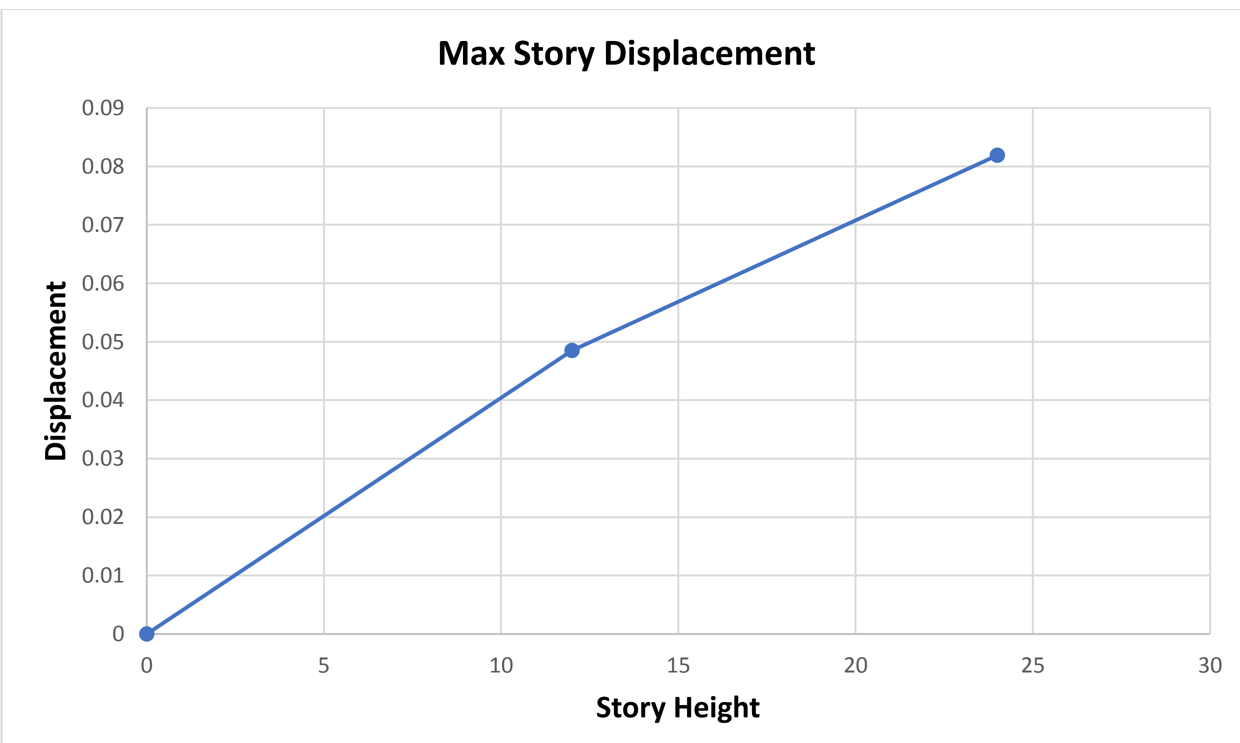
4.2.4 Eccentrically braced OMRF

For response spectrum analysis of the e-braced OMRF, following results were obtained.

4.2.4.1 Max Story Displacement

	Storey	Elevation	Location	X-Dir	Y-Dir
Ecentric Brace					
		ft		in	in
	Storey 2	24	Top	0.081898	0
	Storey 1	12	Top	0.048507	0
	Base	0	Top	0	0

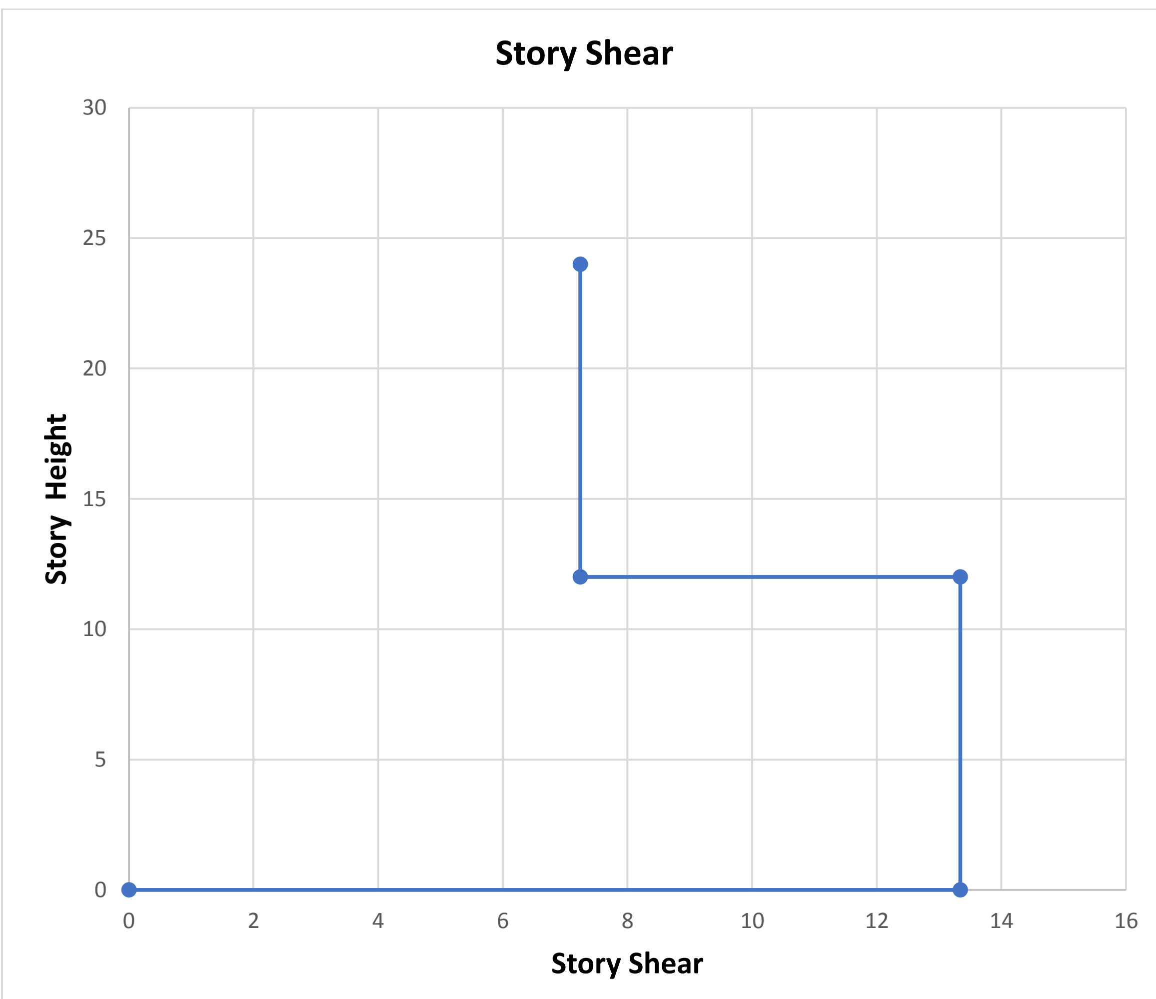
Table 4.7: Max Story Displacements for Eccentrically braced OMRF



4.2.4.2 Story Shear

	Storey	Elevation	Location	X-Dir	Y-Dir
Ecentric Brace					
		ft		**kip**	**kip**
	Storey2	24	Top	7.242	0
		12	Bottom	7.242	0
	Storey1	12	Top	13.339	0
		0	Bottom	13.339	0
	Base	0	Top	0	0
			Bottom	0	0

Table 4.8: Story Shear for Eccentrically braced OMRF



4.3 Analysis on ETABs using Non-linear Static Pushover Analysis

4.3.1 Ordinary Moment Resisting Frame

For pushover analysis of the OMRF, following results were obtained.

	Step	Monitored Displacement (in)	Base Force (kip)	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
Simple Frame	0	0.000106	0	12	0	0	0	0	12	0	0	0	12
	1	0.400106	20.948	12	0	0	0	0	12	0	0	0	12
	2	0.482456	25.261	12	0	0	0	0	12	0	0	0	12
	3	0.643442	31.351	10	2	0	0	0	12	0	0	0	12
	4	0.951279	37.783	8	4	0	0	0	12	0	0	0	12
	5	1.551279	42.658	8	4	0	0	0	11	1	0	0	12
	6	1.951279	45.908	8	4	0	0	0	8	4	0	0	12
	7	2.351279	49.158	8	4	0	0	0	8	4	0	0	12
	8	2.751279	52.408	8	4	0	0	0	8	4	0	0	12
	9	3.151279	55.658	8	4	0	0	0	8	4	0	0	12
	10	3.551279	58.908	8	4	0	0	0	8	4	0	0	12
	11	3.771411	60.562	6	6	0	0	0	8	4	0	0	12
	12	4.000106	60.85	6	6	0	0	0	8	4	0	0	12

Table 4.9: Pushover analysis results for OMRF

4.3.2 X-braced Ordinary Moment Resisting Frame

For pushover analysis of the x-braced steel OMRF, following results were obtained.

	Step	Monitored Displacement (in)	Base Force (kip)	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
X-Brace	0	0.000075	0	12	0	0	0	0	12	0	0	0	12
	1	0.368923	215.693	11	1	0	0	0	12	0	0	0	12
	2	1.120752	637.084	10	2	0	0	0	11	1	0	0	12
	3	1.52376	858.23	9	3	0	0	0	11	1	0	0	12
	4	1.929496	1075.942	8	3	1	0	0	9	2	0	1	12
	5	2.085279	1158.421	8	3	1	0	0	8	3	0	1	12

Table 4.10: Pushover analysis results for X-braced OMRF

4.3.3 Inverted V-braced Ordinary Moment Resisting Frame

For pushover analysis of the inv v-braced OMRF, following results were obtained.

	Step	Monitored Displacement (in)	Base Force (kip)	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
Inverted V Brace	0	0.000022	0	12	0	0	0	0	12	0	0	0	12
	1	0.400022	188.709	12	0	0	0	0	12	0	0	0	12
	2	0.594802	280.602	11	1	0	0	0	12	0	0	0	12
	3	1.264025	577.628	8	4	0	0	0	12	0	0	0	12
	4	1.664025	752.623	8	4	0	0	0	10	2	0	0	12
	5	2.375006	1056.715	7	5	0	0	0	8	3	1	0	12
	6	2.788564	1229.495	6	5	1	0	0	7	4	0	1	12
	7	3.116651	1365.221	5	6	1	0	0	7	4	0	1	12

Table 4.11: Pushover analysis results for Inverted V-braced OMRF

4.3.4 Eccentrically braced Ordinary Moment Resisting Frame

For pushover analysis of the e-braced OMRF, following results were obtained.

	Step	Monitored Displacement (in)	Base Force (kip)	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
Ecentric Brace	0	0.00004	0	12	0	0	0	0	12	0	0	0	12
	1	0.40004	108.349	12	0	0	0	0	12	0	0	0	12
	2	0.642134	173.925	11	1	0	0	0	12	0	0	0	12
	3	1.182257	305.326	8	4	0	0	0	12	0	0	0	12
	4	1.582257	399.902	8	4	0	0	0	12	0	0	0	12
	5	1.982257	494.477	8	4	0	0	0	9	3	0	0	12
	6	2.382257	589.052	8	4	0	0	0	8	4	0	0	12
	7	2.998447	732.506	7	5	0	0	0	8	4	0	0	12
	8	3.645015	878.68	6	6	0	0	0	7	4	1	0	12
	9	4.00004	957.466	6	5	1	0	0	7	4	0	1	12

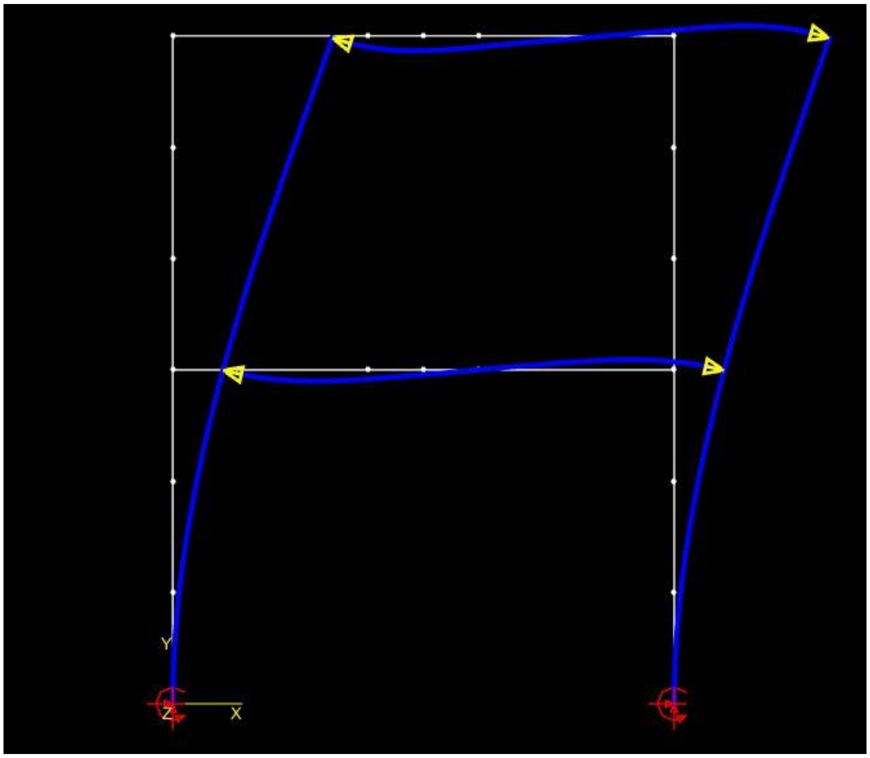
Table 4.12: Pushover analysis results for Eccentrically braced OMRF

4.4 Analysis on MASTAN2 using Second Order Inelastic Analysis

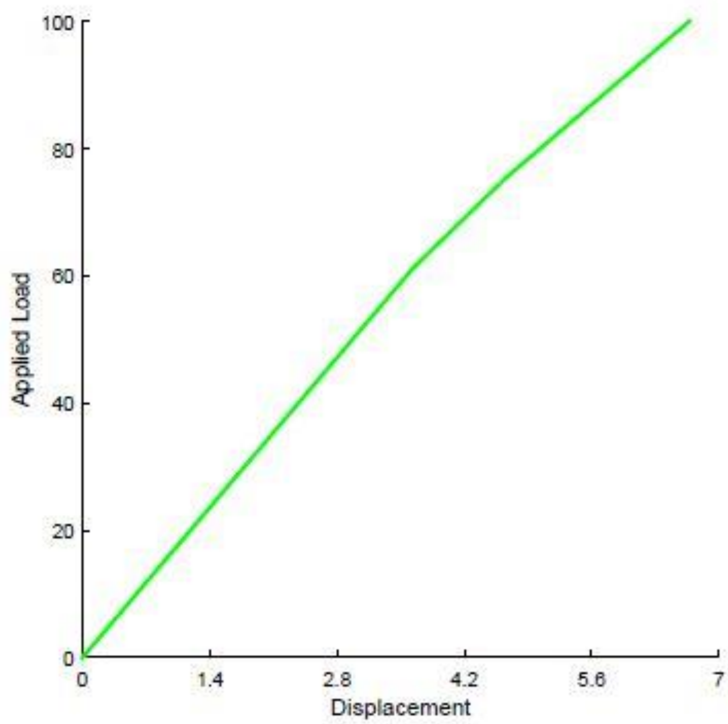
Following steel sections of grade 60 were used for different members of the frame;

- Beams-W12x14
- Columns-W21x93
- Braces- HSS10X0.188

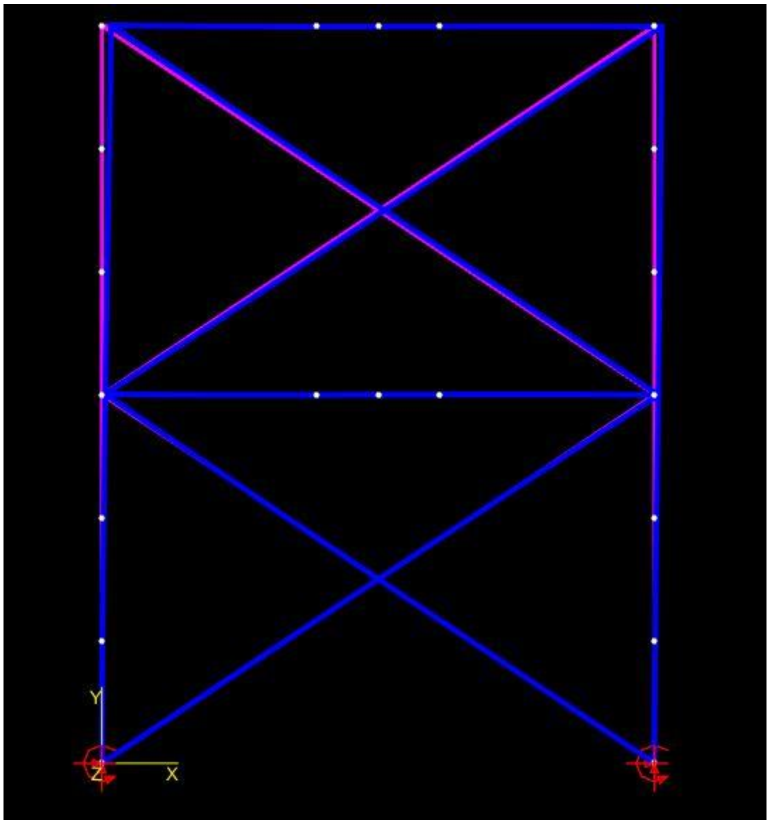
4.4.1 Ordinary Moment Resisting Frame



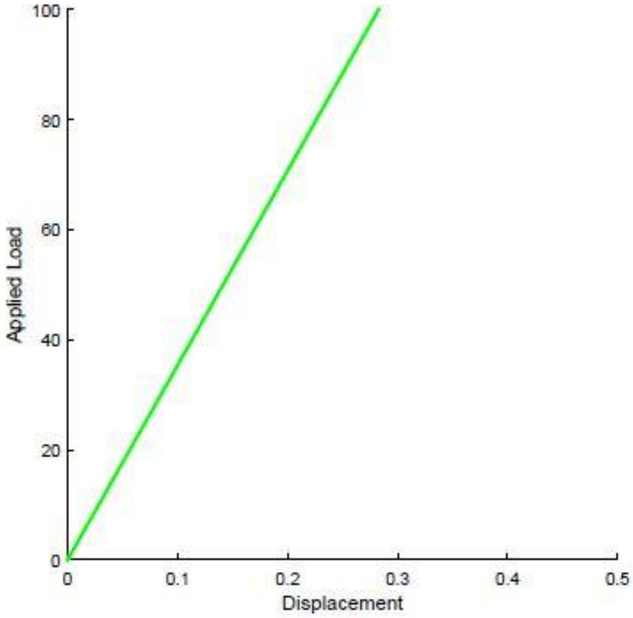
4.4.1.1 Load vs. Displacement



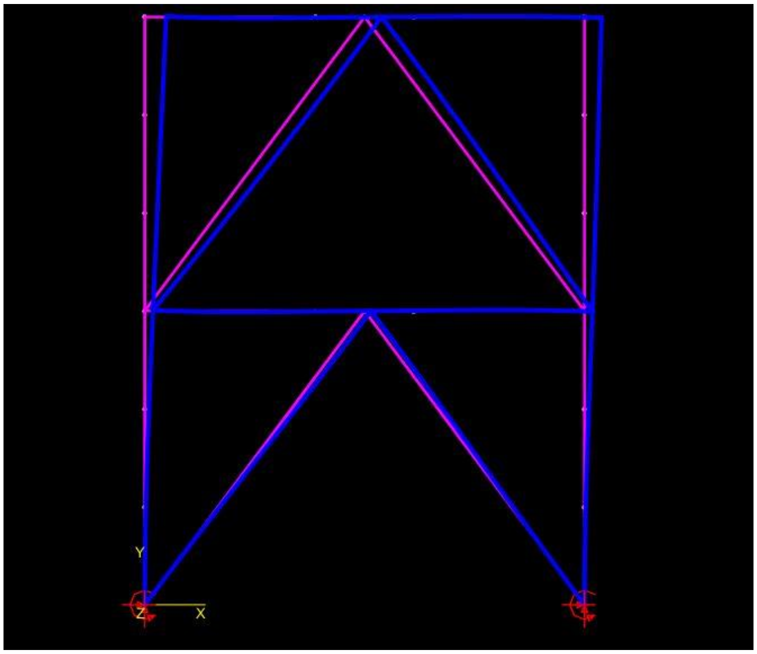
4.4.2 X-braced OMRF



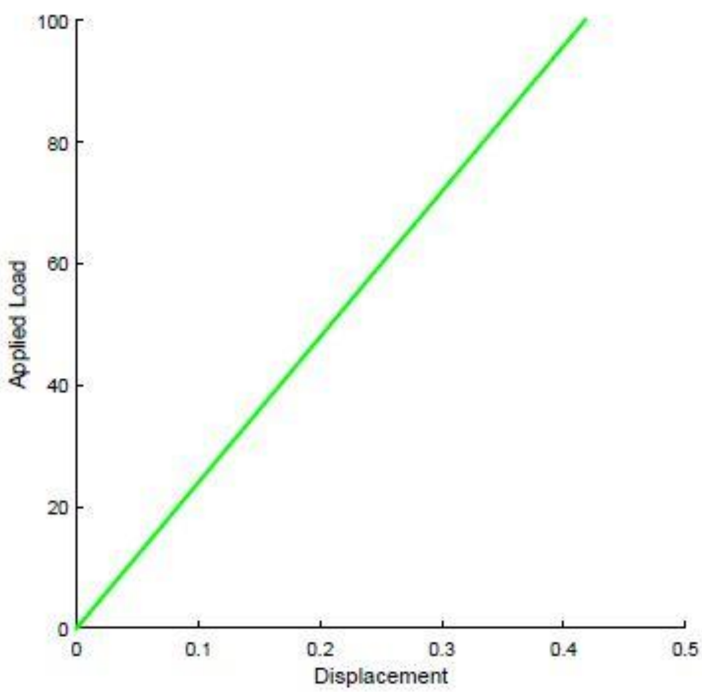
4.4.2.1 Load vs. Displacement



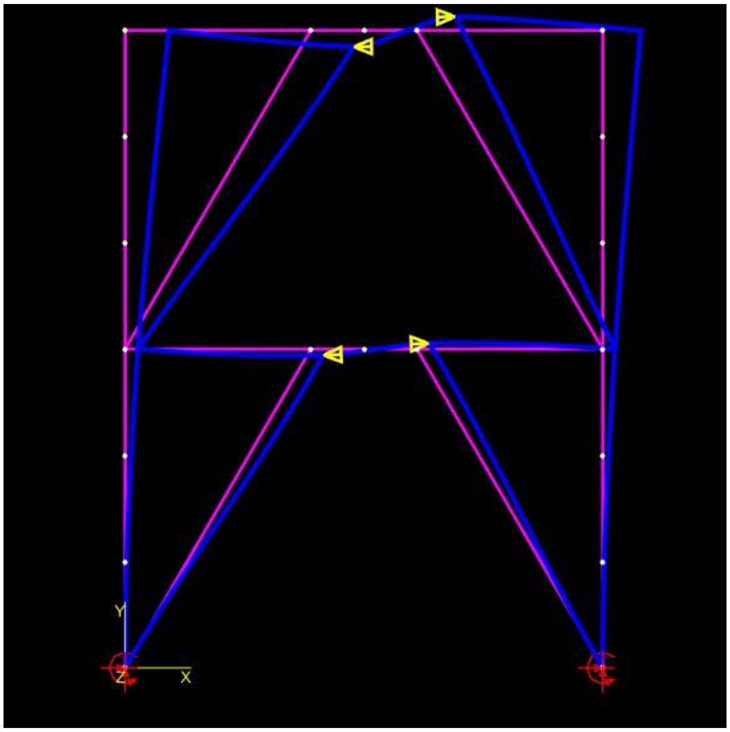
4.4.3 Inverted V-braced OMRF



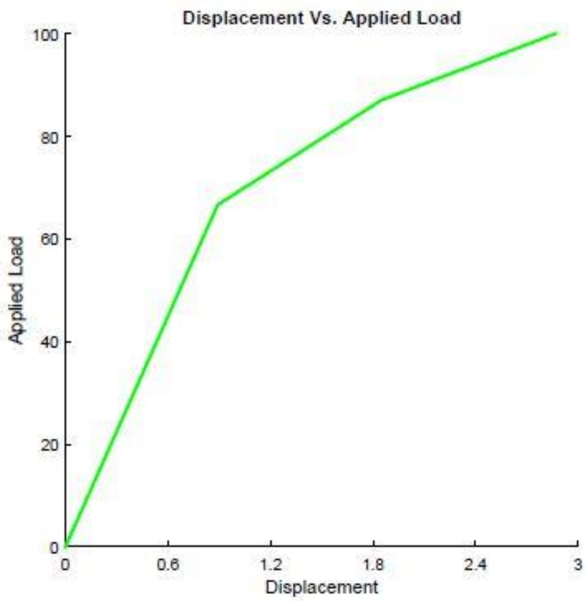
4.4.3.1 Load vs. Displacement



4.4.4 Eccentrically braced OMRF



4.4.4.1 Load vs. Displacement



CHAPTER NO. 5

CONCLUSION

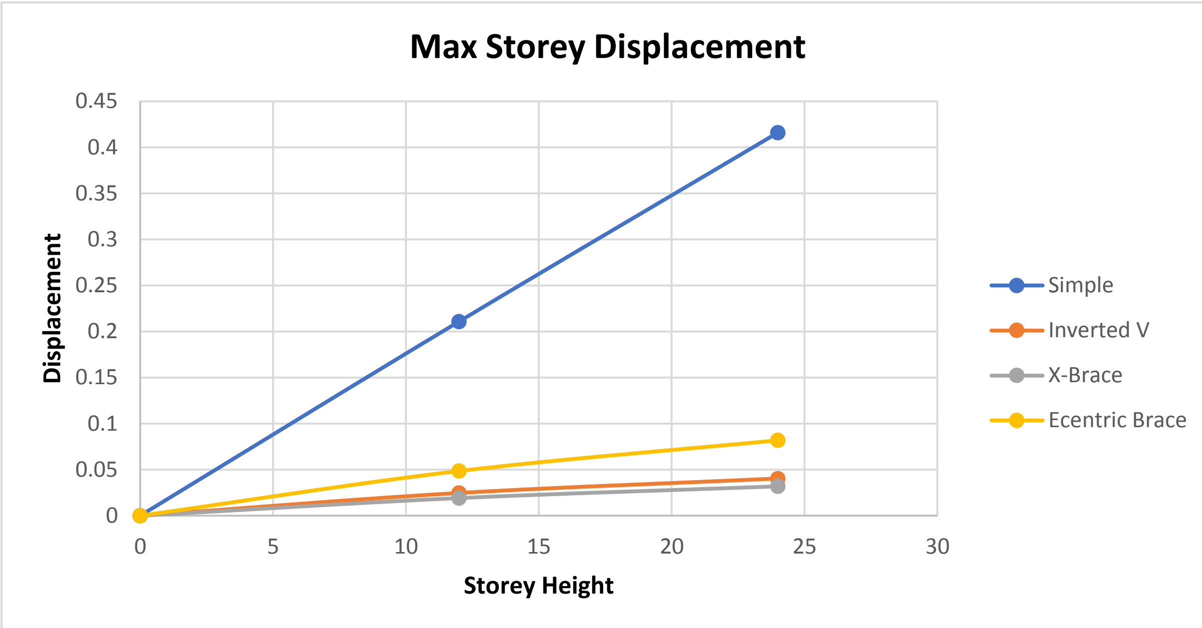
5.1 Introduction

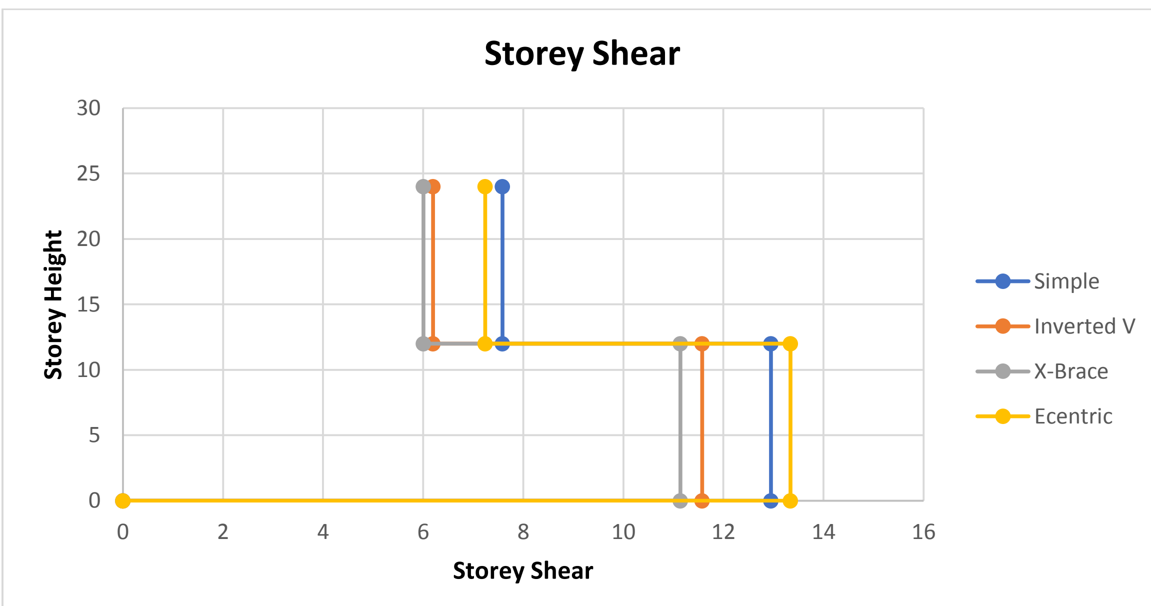
Analytical results for different configuration of the braces were compared using Response Spectrum Analysis (ETABs), Non-linear Static Pushover Analysis (ETABs) and Second Order Inelastic Analysis (MASTAN2).

5.2 Comparison of ETABs Results

5.2.1 Comparison of RSA

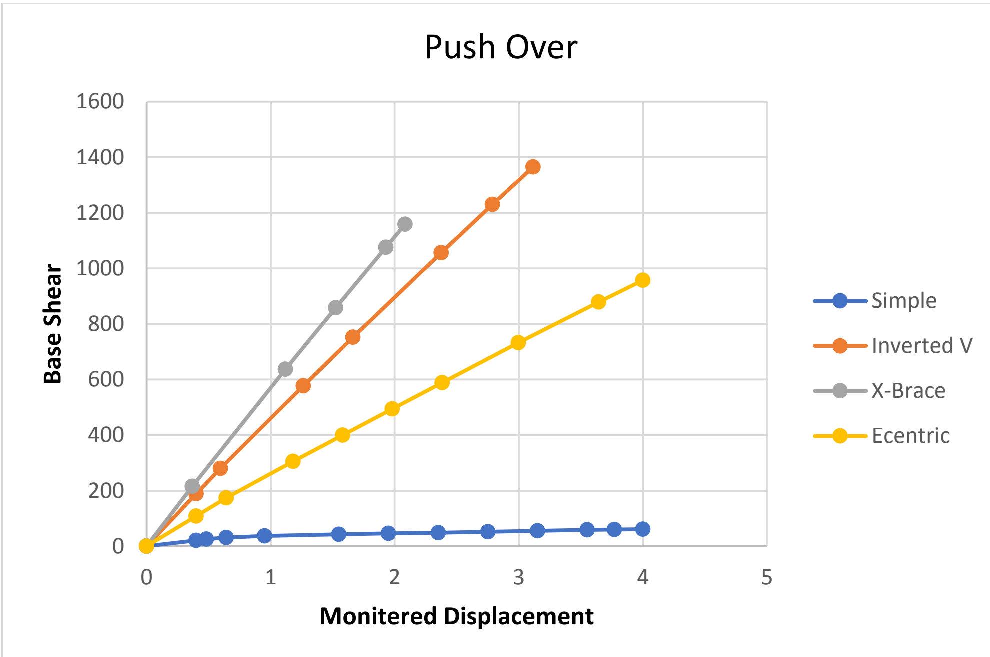
Upon comparing the results of RSA for different configuration of braced, it can be seen that the x-braced omrf offered the least displacement. However, eccentrically braced omrf offered the least story shear.


Max Storey Displacement
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
Displacement
0
5
10
15
20
25
30
Storey Height
Simple
Inverted V
X-Brace
Ecentric Brace


Storey Shear
30
25
20
15
10
5
0
Storey Height
0
2
4
6
8
10
12
14
16
Storey Shear
Simple
Inverted V
X-Brace
Ecentric

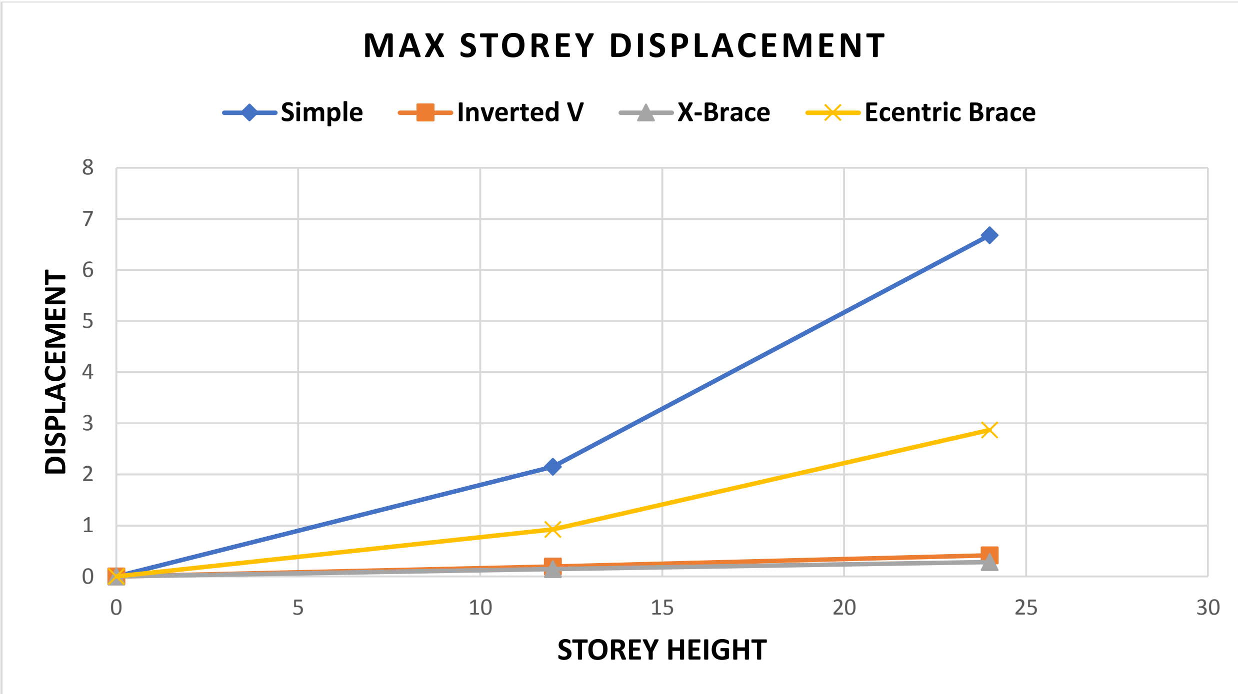
5.2.2 Comparison of Pushover Analysis

It is evident that the eccentrically braced omrf was the only one, apart from the simple omrf, which reached the target monitored displacement of 4 in.



5.3 Comparison of MASTAN2 results

Analysis on MASTAN2 also confirms the results from RSA and Pushover Analysis that the x-braced omrf offers minimum displacements.



5.4 Conclusion

After careful analysis of different types of bracing configuration, it is safe to say that x-bracing offered the minimum displacements. This means that the stiffness of the structure increases the most when retrofitting with the x-braced.

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