UNIVERSITY OF GAZIANTEP
GRADUATE SCHOOL OF
NATURAL AND APPLIED SCIENCES

EARTHQUAKE RESPONSE OF REINFORCED CONCRETE BUILDINGS RETROFITTED WITH INVERTED-V AND ZIPPER BRACES

M. Sc. THESIS
IN
CIVIL ENGINEERING

BY
AYŞEGÜL GÜLTEKİN
JANUARY 2014

Earthquake Response of Reinforced Concrete Buildings Retrofitted With Inverted-V and Zipper Braces

M.Sc. Thesis
in
Civil Engineering
University of Gaziantep

Supervisor
Assoc. Prof. Dr. Esra METE GÜNEYİSİ

By
Ayşegül GÜLTEKİN
January 2014

REPUBLIC OF TURKEY
UNIVERSITY OF GAZIANTEP
GRADUATE SCHOOL OF NATURAL & APPLIED SCIENCES
CIVIL ENGINEERING DEPARTMENT

Name of the Thesis: Earthquake Response of Reinforced Concrete Buildings Retrofitted with Inverted-V and Zipper Braces

Name of the Student: Ayşegül GÜLTEKİN

Exam Date: 09.01.2014

Approval of the Graduate School of Natural and Applied Sciences.

Assoc. Prof. Dr. Metin BEDİR
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Prof. Dr. Mustafa GÜNAL
Head of Department

This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope of quality as a thesis for the degree of Master of Science.

Assoc. Prof. Dr. Esra METE GÜNEYİSİ
Supervisor

Examining Committee Members	Signature
Assoc. Prof. Dr. Esra METE GÜNEYİSİ	....
Assoc. Prof. Dr. Mehmet GESOĞLU	....
Assist. Prof. Dr. Faruk GEYİK	....

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by this rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Ayşegül GÜLTEKİN

ABSTRACT

EARTHQUAKE RESPONSE OF REINFORCED CONCRETE BUILDINGS RETROFITTED WITH INVERTED-V AND ZIPPER BRACES

GÜLTEKİN, Ayşegül
M.Sc. in Civil Engineering
Supervisor: Assoc. Prof. Dr. Esra METE GÜNEYİSİ

January 2014, 73 pages

Steel bracing system is one of the structural systems used to resist the earthquake. Many existing non-ductile reinforced concrete (RC) structures need to be retrofitted to improve the lateral load carrying capacity. The use of steel bracing for retrofitting is one of the convenient solutions for increasing the earthquake resistance of the structure. In this thesis, 4 and 8 storey existing RC buildings having the same plan and three bays on each direction were considered as a case study. For the rehabilitation of the structures, inverted-V and zipper braces were used. The seismic performance of the retrofitted structures was investigated and compared with the existing ones. In this regard, the effectiveness of using inverted-V and zipper bracing in strengthening the building was examined under different earthquake accelerations. The structures were modeled using a finite element method and evaluated by both nonlinear static and time history analyses. Capacity curves, interstorey index, variation of storey displacement, and roof drift time history were computed for each structural system. The results indicated a considerable enhancement in the earthquake performance of the retrofitted structures and it was observed that the zipper braced systems had more lateral load carrying and energy absorption capacity than the inverted-V braced systems.

Keywords: Inverted-V brace; Nonlinear analysis; Reinforced concrete building; Seismic performance; Zipper brace.

ÖZET

TERS V VE FERMUAR ÇAPRAZLARLA GÜÇLENDİRİLMİŞ BETONARME BİNALARIN DEPREM DAVRANIŞLARININ İNCELENMESİ

GÜLTEKİN, Ayşegül
İnşaat Mühendisliği, Yüksek Lisans Tezi
Danışman: Doç. Dr. Esra METE GÜNEYİSİ
Ocak 2014, 73 sayfa

Çelik çapraz sistemler depreme karşı performansı arttırmak amacıyla kullanılan yapısal sistemlerden biridir. Birçok mevcut sünek olmayan betonarme binanın yanal yük taşıma kapasitesini iyileştirmek için güçlendirmeye ihtiyaç duyulmaktadır. Güçlendirme amaçlı çelik çaprazların kullanılması yapıların depreme karşı dayanıklılığını arttırmak için kullanılan yöntemlerden biridir. Bu tez çalışmasında, her iki yönde üç açıklıktan oluşan ve aynı kat planına sahip 4 ve 8 katlı mevcut betonarme binalar örnek olarak alınmıştır. Binaların yapısal sistemlerinin güçlendirilmesinde ters-V ve fermuar tipi çelik çaprazlar kullanılmıştır. Güçlendirilmiş binaların sismik performansları mevcut binalarla kıyaslanarak incelenmiştir. Bu bağlamda, farklı deprem kayıtları altında, ters-V ve fermuar tipi çelik çaprazların güçlendirmedeki etkinliği araştırılmıştır. Yapılar sonlu elemanlar yöntemi kullanılarak modellenmiş ve doğrusal olmayan statik ve dinamik analizler kullanılarak değerlendirilmiştir. Kapasite eğrisi, göreli kat ötelenmesi indisi, kat yerdeğiştirmesi ve çatı ötelenmesinin zamana bağlı değişimi her bir yapısal sistem için hesaplanmıştır. Sonuçlar güçlendirilmiş yapıların deprem performanslarının önemli ölçüde iyileştiğini ve fermuar tipi çelik çaprazlı sistemlerin ters-V çaprazlı sistemlere göre daha fazla yanal yük taşıma ve enerji yutma kapasitesine sahip olduğunu göstermiştir.

Anahtar Kelimeler: Ters-V çapraz; Lineer olmayan analiz; Betonarme bina; Sismik performans; Fermuar çapraz.

To My Mother, Father and Sisters...

ACKNOWLEDGEMENTS

I would like to express my sincere gratitude to my supervisor Assoc. Prof. Dr. Esra METE GÜNEYİSİ for her invaluable guidance, advices, and supervision.

Moreover, my sincere appreciation extends to all my research assistant friends who were directly or indirectly involved in the process of producing this research report, for their generous assistance. Without their support and contribution, this thesis would not have been completed.

I would also like to thank to the committee members for their support and contribution.

Finally, my special thanks are reserved for my parents, all my family. They have given me an endless enthusiasm and encouragement.

TABLE OF CONTENTS

Page

LIST OF FIGURES

Page

LIST OF TABLES

Page

LIST OF SYMBOLS / ABBREVIATIONS

AISC	American institute of steel construction
BRBF	Buckling restrain braced frame
CBF	Concentrically braced frame
FEMA	Federal emergency management agency
IVBF	Inverted-V braced frame
M3	Flexural moment hinges
M_w	The magnitude of ground motion
NDP	Nonlinear dynamic procedure
NEHRP	National earthquake hazards reduction program
NSP	Nonlinear static procedure
OCBF	Ordinary concentric braced frame
PGA	Peak ground acceleration
PGD	Peak ground displacement
PGV	Peak ground velocity
PMM	Axial force-biaxial moment hinges
RC	Reinforced concrete
SCBF	Special concentric braced frame
ZBF	Zipper braced frame

CHAPTER 1

INTRODUCTION

1.1 General

In our country, destructive earthquakes resulted in several levels of failure upon a large number of reinforced concrete (RC) structures due to the fact that existing RC buildings that were only gravity load designed or designed to earlier codes might have insufficient lateral load carrying capacity and limited ductility. In this respect, so many existing reinforced concrete buildings need to be retrofitted to increase the lateral load capacity.

In the literature, Ghobarah et al. (2001) and Maheri et al. (2008) increased the lateral load resistance of steel structures by using steel bracing system. In recent years, the steel bracing has also been used for the retrofitting of reinforced concrete frames. The principal benefit of steel bracing in comparison with RC shear walls can be considered as increase in architectural flexibility, slight decrease in mass, economical application especially due to rapid installation, relatively low cost, and the ability to choose more ductile systems.

Two approaches in seismic retrofitting of original RC structures are commonly used, namely, external bracing and internal bracing. The steel trusses or frames for the external bracing are assigned either as a global external support to the building exterior or, more locally, to the face of the individual building frames. The steel bracing elements are placed in the empty space enclosed by columns and beams of RC frames. Therefore, each unit frame is individually braced from within (Maheri et al., 2008). In the literature, there are several studies in which the different types of concentric braces (single diagonal, X-brace, chevron, and two story X’s, etc.) and eccentric braces (V-bracing, K-bracing, and Y-bracing, etc.) have been used successfully for strengthening of low ductile RC buildings (Tagawa, 1992; Maheri et al., 1997; Symth et al., 2004; Özel and Güneyisi, 2011; Güneyisi and Gültekin, 2012; Kalkan et al., 2013).

Chevron braced frames are one of the popular concentrically braced frame configurations. However, chevron frames have seismic disadvantages such as inter-story drift concentrates in first story and unbalanced vertical force occurs. So Khatib et al. (1988) proposed zipper columns for decreasing the adverse effect of unbalanced force and called full-height zipper mechanism. This system has a good distribution of forces dissipation over all the height of the structures. At the same time, buckling of the compression braces has caused more uniform distribution of damage that is the required objective. However, full-height zipper systems have seismic drawbacks, for instances, compression member buckles directly and collapse can occur immediately, this system has adverse force redistribution capability. So full-height zipper mechanism has limited the applicability due to these disadvantages.

After all, Yang et al. (2006a) and Yang et al. (2006b) revealed that the idea overcoming the disadvantages of full-height zipper mechanism and proposed a design procedure for zipper braced frames targeted at obtaining ductile behavior, labelled suspension system "Suspended Zipper Frame". In this system, brace of the top story remains elastic when the compression braces have buckled and the zipper struts have yielded. They investigated the behavior and seismic performance of the steel frames with the zipper-braced systems by using nonlinear static analyses and nonlinear dynamic analyses. The analyses revealed that the design procedure produced safe zipper-braced systems.

The Federal Emergency Management Agency (FEMA) promulgated seismic evaluation and rehabilitation guidelines in the 1990s. The FEMA unified a standard of seismic retrofitting and enhanced various retrofit strategies. In addition, it supplied beneficial information on seismic retrofitting procedures. The results of this active research have led to significant developments in seismic retrofitting and rehabilitation procedures (FEMA, 2000).

Subjecting a mathematical model to monotonically increasing lateral loads, which represent inertia forces in an earthquake until a target displacement is exceeded, shall be considered in the case of selecting the nonlinear static procedure (NSP) for the seismic analysis of the structure. This mathematical model is directly incorporated the nonlinear load-deformation behaviors of individual components and elements of the structure. The target displacement is purposed to represent the maximum displacement which is likely to be experienced during the design earthquake. Due to accounting directly for effects of material inelastic response of the mathematical model, the calculated internal forces would be reasonable approximations of those expectations during the design earthquake (FEMA 356, 2000).

Furthermore, subjecting a mathematical model to earthquake shaking, which represents by ground motion time histories to achieve forces and displacements, shall be considered in the case of selecting the nonlinear dynamic procedure (NDP) for the seismic analysis of the structure. This mathematical model is directly incorporated the nonlinear load-deformation behaviors of individual components and elements of the structure. The basis, approach, acceptance, and criteria of the model for NDP and NSP are similar. The basic exception is that the response calculations are carried out using time history analysis. The design displacements are not specified using a target displacement; however behalf are determined directly through dynamic analysis using ground motion time histories for NDP. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, the analysis needs to be carried out with more than one ground motion record. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces would be reasonable approximations of those expectations during the design earthquake (FEMA 356, 2000).

1.2 Objective and scope

The principal purpose of this study is to compare the seismic performance of the existing reinforced concrete (RC) buildings retrofitted with inverted-V and zipper braces. As a case study, 4 and 8 storey RC buildings were considered. The buildings have the same plan and three bays on each direction. The effect of using inverted-V and zipper bracing in strengthening the building was examined. The structures were modeled using a finite element method and evaluated by both nonlinear static and time history analyses. The seismic response of the original frames and those with inverted-V and zipper braces were analyzed using different earthquake ground motions. In the analysis, 1984 Morgan Hill, 1992 Erzincan, and 1999 Hector Mine earthquake records were utilized. The performance of the original and retrofitted RC structures is evaluated in term of capacity curves, storey drift, time history of roof displacement, etc.

Chapter 1-Introduction: The aim and objective of the thesis are summarized.

Chapter 2-Literature Review: This chapter is briefly given the background on the previous studies about steel bracing systems, especially inverted-V and zipper braced frames.

Chapter 3-Methodology: In the methodology, the analytical model of the existing structures and those with inverted-V and zipper braces are explained. The description of the analysis methods and the characteristics of the ground motion records are also provided.

Chapter 4-Results and Discussion: This chapter presents and compares the results obtained from nonlinear static and dynamic analyses of each structural system.

Chapter 5-Conclusions: The conclusions are given in the light of findings from the overall results of the analysis.

Appendix A: Deflected shapes: The deformed shape of each structure is demonstrated.

CHAPTER 2

LITERATURE REVIEW

2.1 Concentrically Braced Frames

Concentrically braced frames (CBFs) are a class of structures resisting lateral loads through a vertical concentric truss system, the axes of the members aligning concentrically at the joints. The CBFs tend to be efficient in resisting lateral forces because they can provide high strength and stiffness. These characteristics can also result in less favourable seismic response, such as low drift capacity and higher accelerations. The CBFs are a common structural steel or composite system in areas of any seismicity. The CBFs are arranged considering beams, columns, and braces to form a vertical truss. This system resists to the lateral earthquake forces by truss action and also develops ductility through inelastic action in braces. This system is considered as being the most stiffness efficient when braces behave in elastic range. Once the inelastic response is initiated, the lateral stiffness starts degrading and an asymmetrical response is developed. The popularity of this system is attributed to the reduced cost, supervised fabrication process, and speed of erection. Despite the advantages mentioned above, the CBFs exhibit relatively poor energy absorption and dissipation through inelastic response. Progressive slackening of braces, degradation of compressive strength and premature fracture render this system inefficient and unreliable for seismic applications. The concentrically braced frames classify as either ordinary or special. Ordinary concentric braced frames do not have extensive qualification regarding elements or connections. They are generally utilized in low seismic risk areas. On the other hand, special concentrically braced frames are generally employed in the areas of high seismic risk. The goal of the concentrically-braced frame design is ensured satisfactory ductility (i.e., to stretch without breaking suddenly) (Sabelli et al., 2013).

Widespread CBF configurations are presented in Figure 2.1. The V-braced frames of Figures 2.1d and 2.1e are also known as chevron-braced frames (Sabelli et al., 2013).

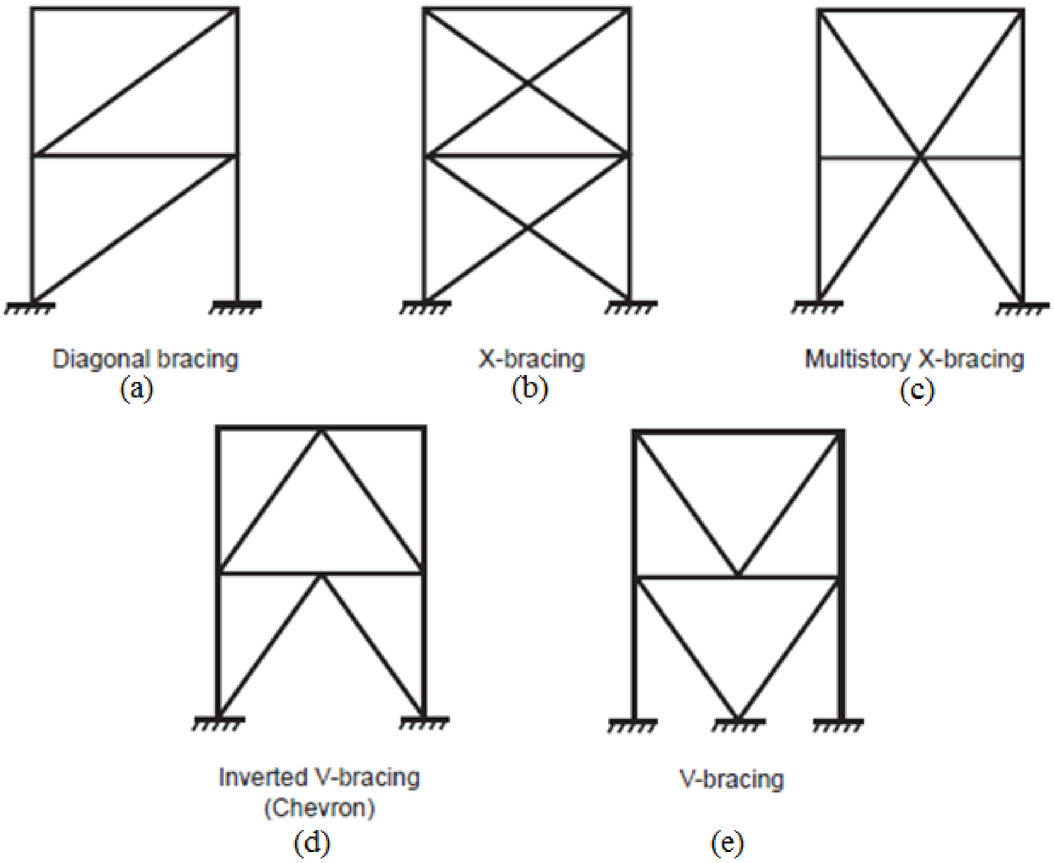


Figure 2.1 Different configurations of CBFs (Sabelli et al., 2013)

Ikeda and Mahin (1986) produced a pin-ended brace model with a plastic hinge located at its mid-point (Figure 2.2). Braced frame models were conducted to model the inelastic behaviour of steel braces. Therefore, frame systems could categorize as finite element, phenomenological and physical theory models. Phenomenological models extremely utilized for nonlinear seismic analyses in spite of the difficulty of determining input data. In the model, an analytical axial force-plastic hinge formulation was derived and the variation of tangent modulus of elasticity during cycles was studied. Cyclic analysis results of the braces and dynamic analysis results of braced frame structures were compared with the test data. Due to simplicity of the model center-plastic hinge approach is suitable for large scale structural system analyses. Model was capable of simulating the inelastic behaviour of braces in cyclic and dynamic analysis. Modifications on the formulation by considering variation of the tangent modulus of elasticity and local buckling could lead a better estimation of cyclic inelastic behaviour.

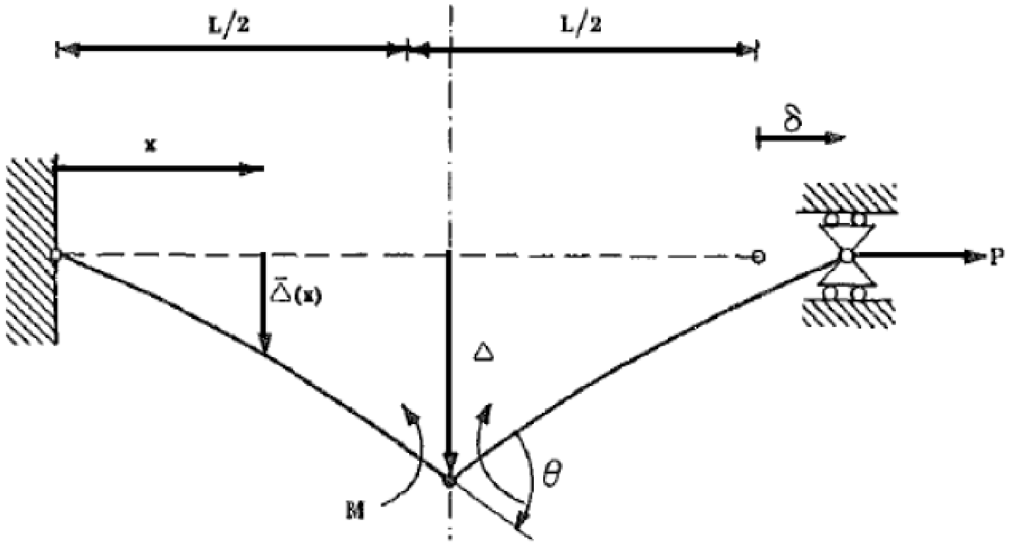


Figure 2.2 Brace model (Ikeda and Mahin, 1986)

Badoux and Jirsa (1990) investigated the behavior of braced frames both analytically and experimentally. Retrofitted frame was prepared to have deep beams and short columns and tested under lateral cyclic loading. An analytical study was conducted by simulating an interior column in a braced frame loaded laterally (Figure 2.3). In addition to that, a parametric study was conducted to understand the effect of slenderness ratio of braces on the response of retrofitted frame. Studies showed that RC frame and the bracing system could be taken as independent systems and designer could adjust desired strength and stiffness by changing brace sections. To have acceptable seismic behavior, brace sections might be designed to remain elastic because of the unpredictable nature of exposed seismic loading that could trigger buckling. Reducing slenderness ratio would help to prevent inelastic buckling effects on the brace sections. To prevent failure under gravity loads, failure mechanism of RC frame might be taken into account before designing the braces.

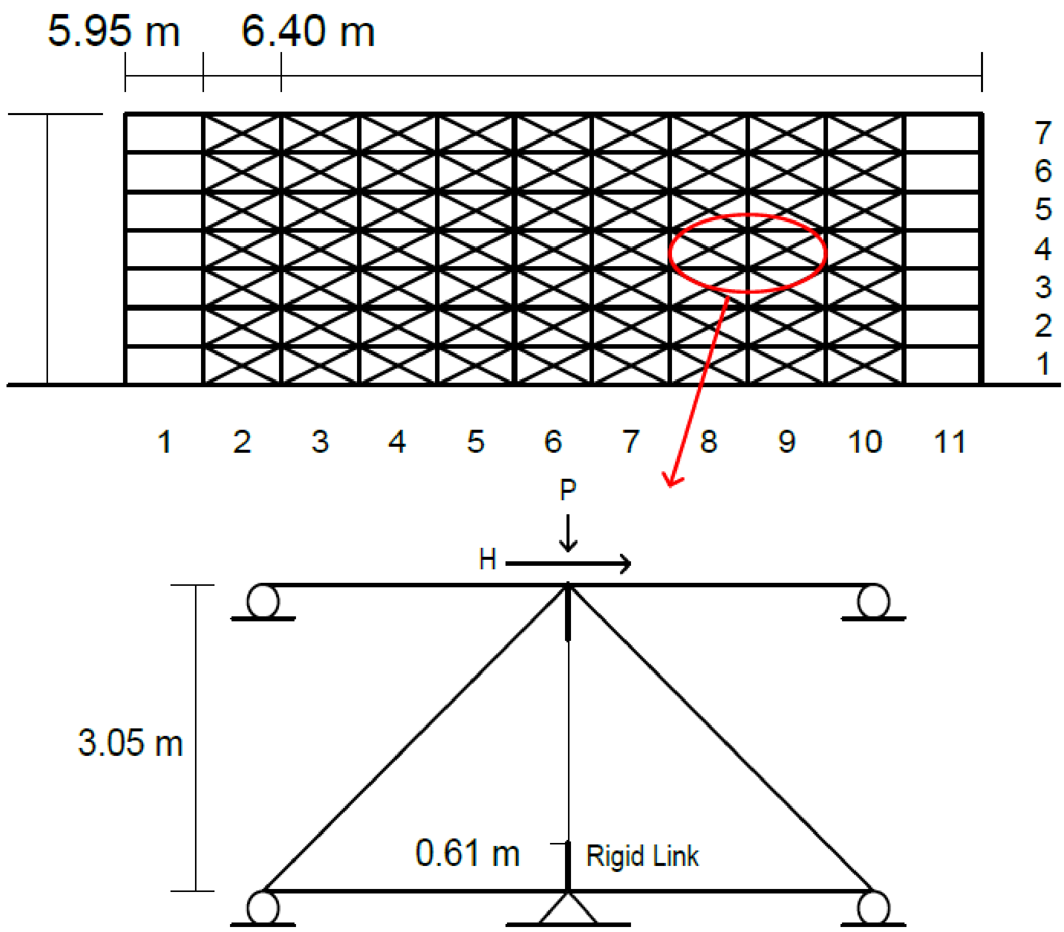


Figure 2.3 Retrofitted frame model (Badoux and Jirsa, 1990)

Pincheira and Jirsa (1995) examined the performance of three, seven and twelve story existing reinforced concrete (RC) buildings for five earthquake ground motion records measured on firm and soft soils. Structures were rehabilitated with three different rehabilitation schemes; post tensioned bracing, X-bracing, and addition of structural infill walls (Figure 2.4). All methods were compared with the response of original structures in terms of stiffness and strength enhancement. According to analysis results, it was observed that it was possible to choose from several different brace arrangements whilst the solution being non-unique in order provides acceptable seismic performance. For three-storey building, all the retrofitting methods exhibited satisfactory performance for all ground motions. For seven- and twelve-storey buildings, structural wall provided satisfactory performance, however, bracing systems did not provide the expected performance for all ground motion records. Brace systems could affect axial load levels on RC members adversely, so for such cases, it could be important to improve axial load capacities of RC members. For the post tensioned brace case, the distribution of internal forces on RC members are closely related with bracing pattern, brace size and initial level of brace pre-stress.

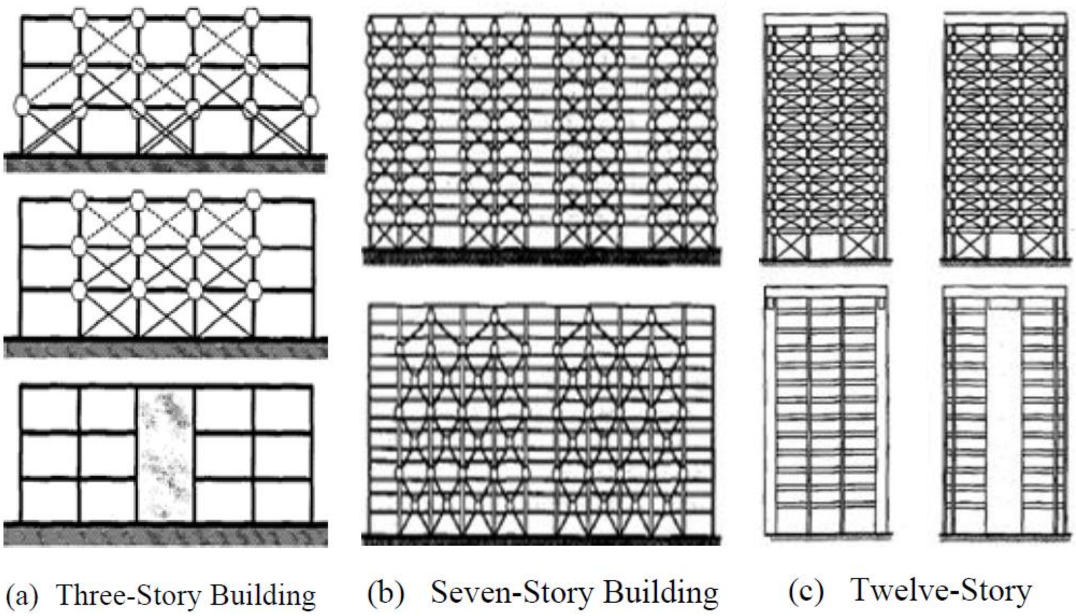


Figure 2.4 Retrofitted frame model of examined buildings (Pincheira and Jirsa, 1995)

Tremblay (2001) conducted several experimental studies on inelastic response of steel braces under cyclic loading (Figure 2.5). A wide range of brace parameters were investigated such as section type, dimensions, boundary conditions, slenderness, compactness, and material properties. Effect of displacement histories and the buckling modes (in-plane, out-of-plane) of braces were also investigated. Equations were proposed for post-buckling, displacement, and force relations. Recommendations of this study were reported as follows: Actual yield strength of all the specimens exceeded the nominal properties and this effect should be included in design. The compressive strength of the braces at first buckling (C_u) generally exceeded the value founded from the column design curves. For less slender braces ($\lambda = \sqrt{\frac{F_y}{\pi^2 E}} < 1$), compression and tension braces could develop simultaneously a compression force equal to C_u and a tension force equal to (A_g*F_y). For slender braces, compression force could be taken as 0.8*C_u when tension braces had yielded. Applied loading history affected the maximum tension force that would develop in a brace section and highest loads were observed under large tension excursions applied early in the test. Proposed equations for investigated parameters agreed well with the test data and values specified in several codes could be modified in order to have better estimations. Fracture of bracing members was highly dependent on slenderness ratio and slender braces could sustain higher ductility levels prior to fracture.

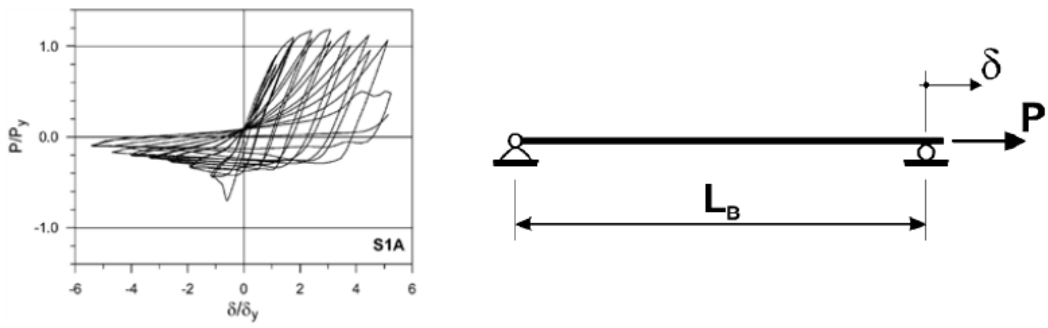


Figure 2.5 Typical brace section and its hysteretic response (Tremblay, 2001)

Ghobarah and Elfath (2001) investigated the seismic performance of a low-rise non-ductile RC structure subjected to various ground motion records. An existing three-storey office building was rehabilitated using; (a) concentric inverted-V-bracing, (b) inverted-V bracing with vertical steel links (eccentric), (c) different orientation of second case (Figure 2.6). According to the analysis results: Concentric bracing case V1 provided the highest increase in stiffness. However, inverted-V bracing with vertical steel links (case E2) resulted in higher lateral loading capacity than V1 and E1 cases. E1 and E2 cases suffered less damage and deformation under the load demanded by earthquake ground motions. Link deformation angle is an important factor for bracing systems and it could be kept under an allowable shear deformation limit. Plastic mechanism of the structure under seismic loads was significantly related with the distribution of braces over the structure.

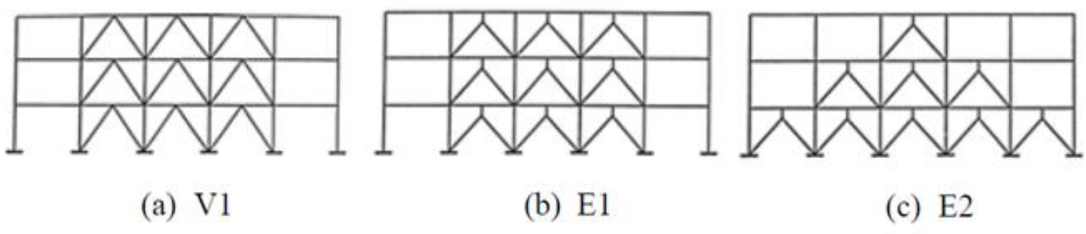


Figure 2.6 Applied rehabilitation system (Ghobarah and Elfath, 2001)

Broderick et al. (2008) examined three concentrically-braced sub-frames using the shake table tests (Figure 2.7). And these frames were compared with a series of correlative inelastic analyses. The brace cross-section was changed between tests to investigate the influence of brace slenderness on the stiffness, resistance and ductility displayed by the frame under strong earthquake loading. Time-history and nonlinear static analyses were considered on three concentrically-braced sub-frames. Response simulations using a two-dimensional analytical model of the test frame were compared with the experimental results. In their study, for three single-storey braced frames it was conducted experimental study using shake table and analytical study. Also it was calculated the base shear–frame drift relationship by nonlinear static analysis and the acceleration response of the frame using time-history analysis. The results showed that the axial force in brace for experimental and analytical study had similar level. The pushover analysis result provided an accurate envelope of the observed hysteresis curve, with slightly better agreement being achieved when both the compression and tension brace were included in the structural model.

Figure 2.7 View of test frame on shake table prior to testing (Broderick et al., 2008)

Mahmoudi and Zaree (2010) used response modification factor to determine the nonlinear performance of building structures during strong earthquake. And they evaluated the response modification factors of conventional concentric braced frames

(CBFs) as well as buckling restrained braced frames (BRBFs). It was used different brace configurations (chevron V, inverted-V and X bracing) as seen in Figure 2.8. These models included single and double bracing bays, multi-floors and was evaluated the response modification factor depends on ductility and overstrength according to the static nonlinear analysis. It was observed that the response modification factors for BRBFs were higher than the CBFs one. It was found that the number of bracing bays and height of buildings have had greater effect on the response modification factors. The results illustrated that the overstrength and response modification factors of CBFs and BRBFs decreased with an increase in the height of buildings. However, the reduction factors due to ductility for CBFs and BRBFs were different. For CBFs, it was constant value but for BRBFs, it was varied quantity for different numbers of storeys.

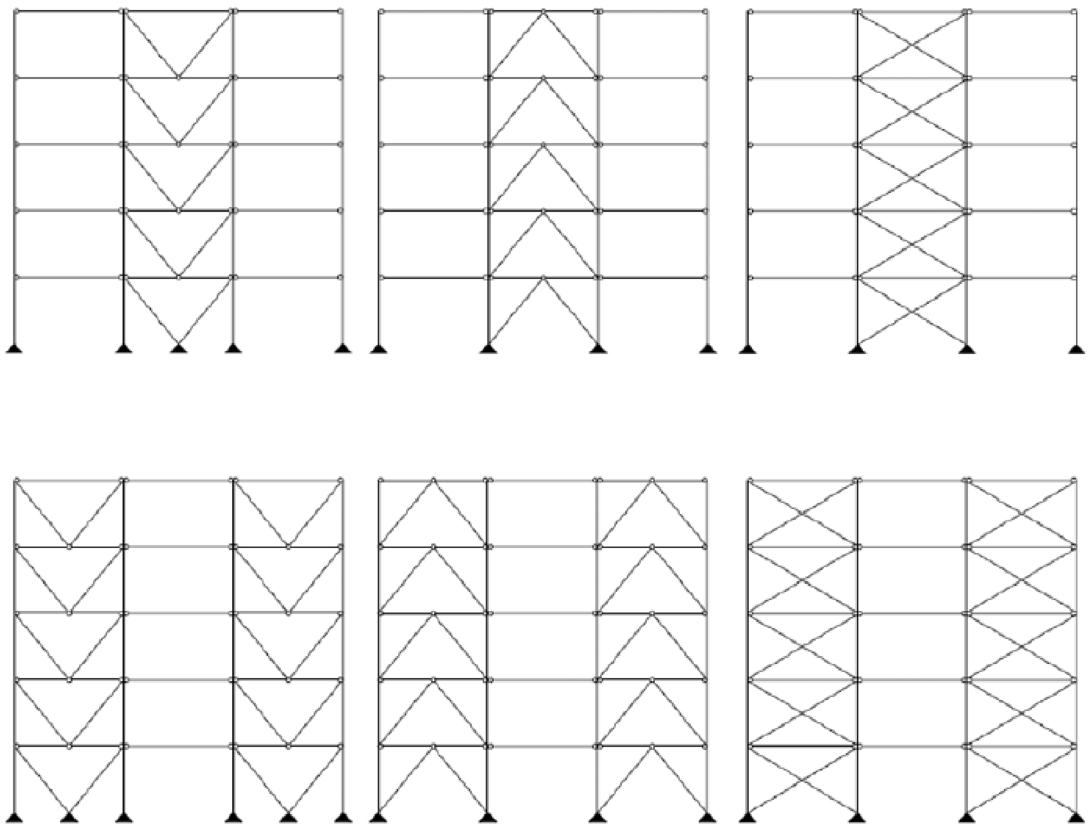


Figure 2.8 Brace configuration (Mahmoudi and Zaree, 2010)

Hajirasouliha and Doostan (2010) proposed a simplified analytical model for seismic response prediction of concentrically braced frames (Figure 2.9). It was carried out a multistorey frame model by performing a static pushover analysis. It was shown that the proposed improved shear-building models provide a better estimate of the non-linear dynamic response of the original framed structures, as compared to the conventional models. It was shown that the modified shear-building model was not sensitive to the ground motion intensity and maximum story ductility; and therefore, could be utilized to estimate the seismic response of concentrically braced frames from elastic to highly inelastic range of behaviour. The results indicated that the proposed model was also capable to estimate the global damage experienced by the concentrically braced frames from low (less than 20%) to high (more than 70%) level of damage intensity.

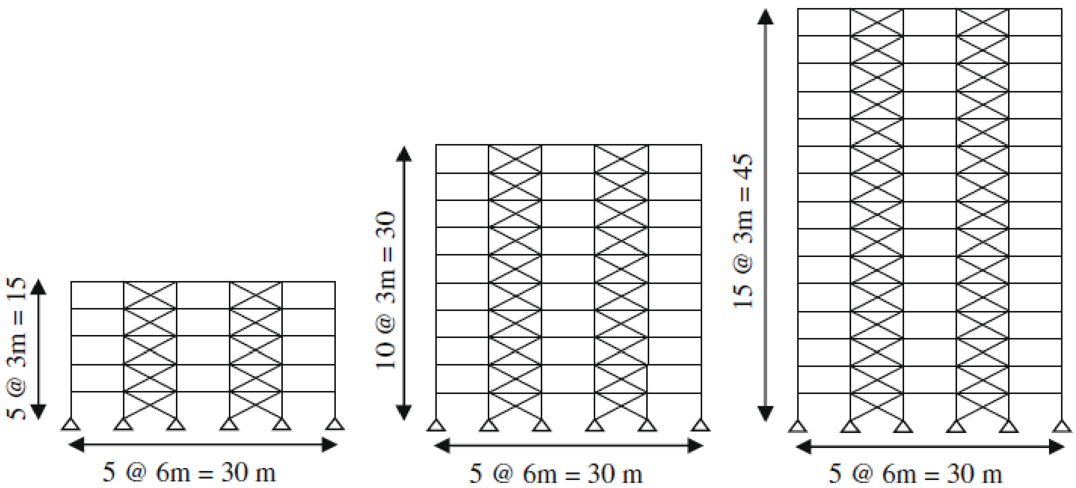


Figure 2.9 Type of brace configuration (Hajirasouliha and Doostan, 2010)

Lumpkin et al. (2012) presented in two three-storey special concentric braced frames (SCBFs) that were tested at the laboratory. Figure 2.10 shows typical configurations of prior test specimens. The specimens evaluated a new design approach for midspan gusset plate connections. The two specimens had hollow section or wide-flange braces in combination with framing members and connections typical of those used in a three-storey building in regions of high seismicity. Composite, concrete slabs were placed on each storey. The tests were designed using a recently proposed design method to balance the desired yield mechanisms and form yield hierarchy. The results demonstrated that multi-storey SCBFs exhibited good inelastic seismic performance with proper design detailing. Together with prior test results, the test specimens advanced design recommendations for SCBFs, which resulted in thinner, more compact corner gusset plate connections, a rational method of dimensioning mid-span gusset plates, and a balanced-design procedure for enhanced ductility.

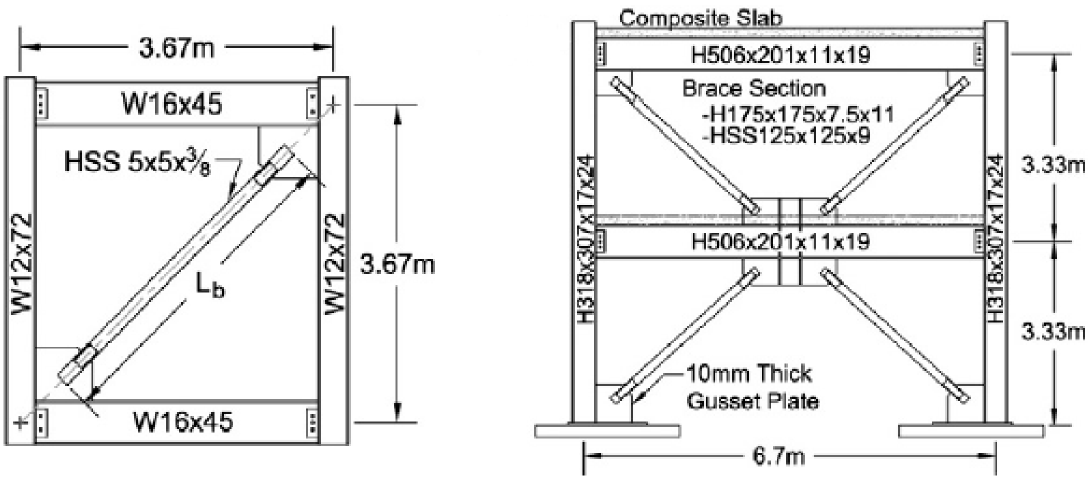


Figure 2.10 Typical configurations of prior test specimens (Lumpkin et al., 2012)

2.2 Description of Chevron and Zipper Braced Frames

The chevron or inverted-V brace frames are the most popular configuration, however, some important design problems are appeared. When subjected to lateral force, the braces resist in tension and compression. For steel members, the capacity in compression is, in general, smaller than the capacity in tension and its value depends both on the properties of the cross section and the boundary conditions. When the compression capacity is attained, the brace buckles and a plastic hinge develops at mid-height. At this stage, the midpoint of the brace undergoes large displacements, generating even larger moments. Since the section is fully plastic, the axial capacity of the member reduces to accommodate a larger moment capacity. On the other hand, the brace in tension attracts even more load to compensate for the loss in capacity of the compression brace. This generates an unbalanced vertical force that is transmitted to the beam at its midpoint. Thus, the capacity design of the beam becomes very costly due to the big section size required to resist such a force. To improve the performance of the frame, the “suspended zipper frame” concept has been proposed in the literature (Leon and Yang, 2003; Yang, 2006a; Yang, 2006b; Yang, 2008a; Yang, 2008b).

Bruneau et al. (2005) developed a solution about the seismic design of the retrofitted steel buildings by various members of the U.S. research community, including solutions being developed at the University at Buffalo for the seismic retrofit of bridges and buildings. Figure 2.11 shows the possible inverted-V braced frame mechanism under loading.

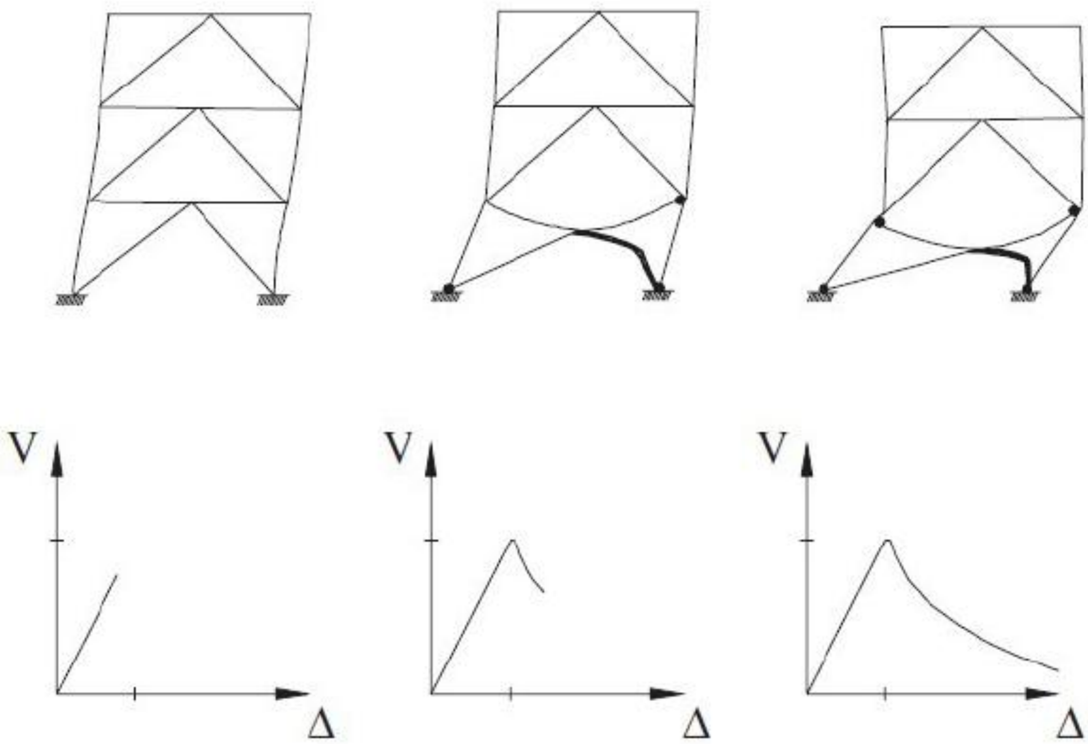


Figure 2.11 Inverted-V braced frame mechanism (Bruneau et al., 2005)

As a solution for this problem, the idea to link every beam to brace intersection point with columns, called “zipper columns”, was suggested by Khatib et al. (1988). In this case, when a brace buckles, the unbalanced vertical force was transmitted to the “zipper column” as tension force. The column re-distributed the force to the upper story braces as an extra compression force, forcing the upper story compression brace to buckle. A new unbalanced vertical force was then generated and transmitted to the next level through another “zipper column”. This mechanism, called the “zipper mechanism”, would repeat itself at all levels forcing all the compression braces to buckle almost simultaneously, resulting in a better energy dissipation distribution over the height of the building and avoiding concentration of damage in just one story. Plastic hinges also developed at the base of the columns and at the midspan of the beams. This was the plastic collapse mechanism of the "zipper frame". So Khatib et al. (1988) proposed zipper columns for decreasing the adverse effect of unbalanced force and called full-height zipper mechanism (Figure 2.12). This system had a good distribution of forces dissipation over all the height of the structures. At the same time, buckling of the compression braces caused more uniform distribution of damage that was the required objective. However, full-height zipper systems had some seismic drawbacks, for instances, compression member buckled directly and collapse could occur immediately, this system had adverse force redistribution capability. So full-height zipper mechanism limited the applicability.

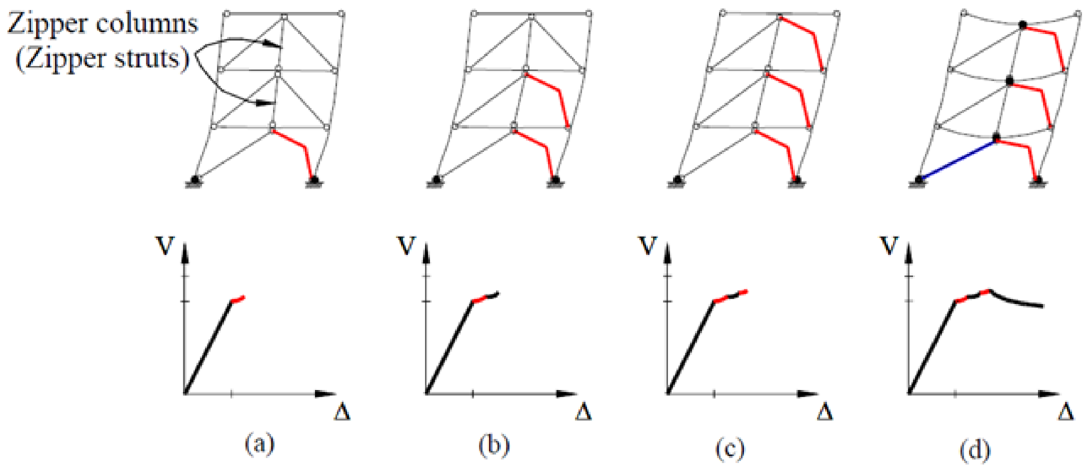


Figure 2.12 Full-height zipper mechanism (Khatib et al., 1988)

Leon and Yang (2003) used a new system labeled as "suspended zipper frame". This system was applied to one bay three storey steel structures. The aim of their study was to eliminate the drawbacks of a full-height zipper mechanism. For the design of suspended zipper frame, the top story braces were designed to remain elastic; all other compression braces were designed to buckle. The suspended zipper struts were designed to yield in tension. Table 2.1 shows the member size of the beam, column, and while Figure 2.13 reveals the elevation of 3 storey zipper braced frame. In their study, they proposed the design strategy for zipper braced frame. It was obtained more ductile suspended zipper frames than full-height zipper mechanism, also it was showed that the suspended zipper system had superior seismic performance and strength compared to ordinary zipper frames.

Table 2.1 Member size (Leon and Yang, 2003)

Storey	Braces	Columns	Beams	Zipper
3	HSS 12x12x5/8	W14x74	W12x45	W12x50
2	HSS8x8x1/2	W14x74	W12x53	W10x26
1	HSS8x8x1/2	W14x74	W12x53	-

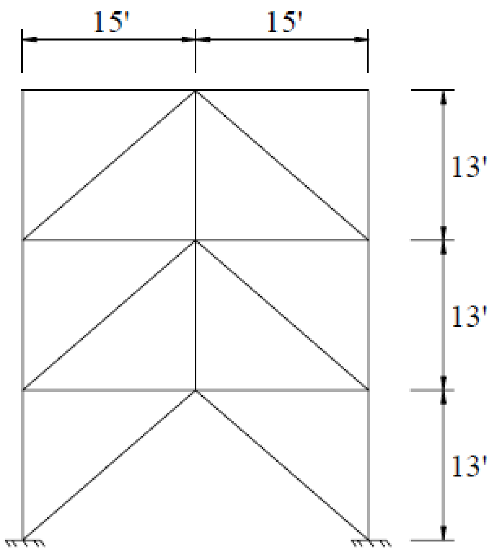


Figure 2.13 Elevation of 3-storey steel structure with zipper element (Leon and Yang, 2003)

Yang (2006) also revealed that the idea eliminating the drawbacks of full-height zipper mechanism and proposed design procedure for zipper braced frames targeted at obtaining ductile behavior, labelled suspension system as "suspended zipper frame". In this system, the brace of the top storey remained elastic when the compression braces had buckled and the zipper struts had yielded. They investigated the behavior and seismic performance of the zipper-braced systems by using nonlinear static analyses and nonlinear dynamic analyses. The analyses demonstrated that the design procedure produced more safe zipper-braced systems. Figure 2.14 illustrates the suspended zipper mechanism of the frame system.

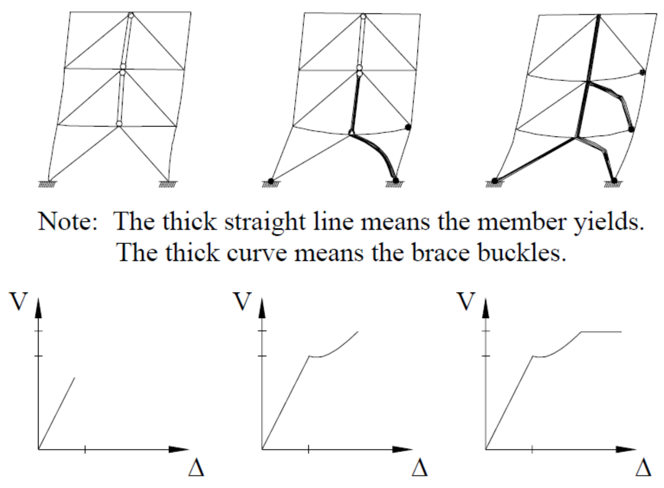


Figure 2.14 Suspended zipper mechanism (Yang, 2006)

Yang and Leon (2007) proposed a modified zipper braced frame structure consisting of an increased size in top-storey braces. This concept requires the top storey braces to remain elastic and prevent the full zipper mechanism formation. This modified configuration was mentioned as suspended zipper frames. The suspended zipper frame consisted of a partial height zipper braced frame and an elastic hat truss at the top floor with the aim to prevent the overall collapse of the structure. The suspended zipper columns were able to transfer the unbalanced vertical forces developed gradually due to the brace's inelastic behavior at the lower part of the structure to the top storey braces and supported the beams at mid-span. As a result, the beams could be design to hinge, which implied the reduced beam sizes and a more economical design. Figure 2.15 shows the structural behavior of the zipper frames.

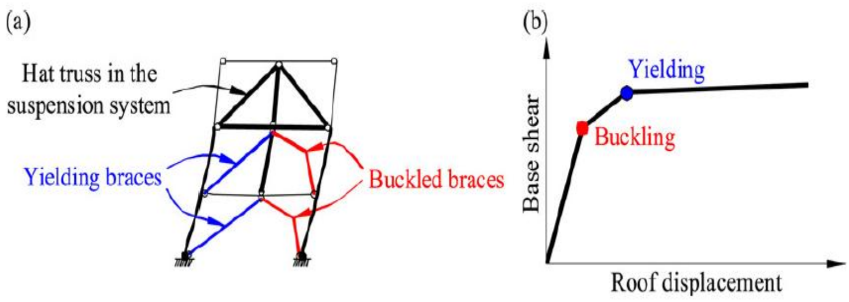


Figure 2.15 Expected behavior and performance of zipper frames (Yang et al., 2007)

Chen (2011) refined the design method for the zipper braced frame system which was initially proposed by Tremblay and Tirca (2003) and to study the system's behavior under seismic loads by means of accurate inelastic time-history analysis and to improve the overall performance of the CBF with strong zipper columns and to validate the design method, a 4-, 8- and 12-storey buildings were analyzed under three ensembles of ground motions. Results from the experimental tests emphasized the difference in behavior of slender, intermediate, and stocky tubular braces subjected to quasi-static cyclic loads. Based on these test results, analytical brace model were developed and two computer programs such as Drain2DX and OpenSees were selected for the numerical simulations. These analyses were performed at the design level while the structures still remained stable until the failure was initiated.

Yang et al. (2006) carried out an investigation on the seismic performance evaluation of the suspended zipper braced frame in two different phases. Hybrid and analytical models of the suspended zipper braced frames were generated and validated in the first phase. A probabilistic seismic performance evaluation method was developed and used in the second phase to examine the seismic risk of the suspended zipper braced frame. The method for seismic performance evaluation was computerized and a comparison between different bracing systems was analyzed on a test building. The results of such probability-based performance evaluation provided the information needed to demonstrate the advantage of using the suspended zipper braced frame.

Özçelik and Sarıtaş (2010) undertook a numerical study in order to evaluate and compare the seismic response of the steel frames with chevron and suspended zipper braces (Figure 2.16). For this purpose, three, nine and twenty-storey steel buildings were designed for both brace configurations. The designed buildings were analyzed under static and dynamic loadings. Member deformations, member forces, inter-storey drifts, top storey drifts, base shears were gathered for each analysis. They obtained that for low rise steel buildings, the suspended zipper braced frame demonstrated almost same behavior as inverted-V braced frames designed according to AISC Specifications and Seismic Provisions in terms of base shear capacity and interstorey drift demands without requiring overly stiff beams. For moderate rise steel buildings, the suspended zipper braced frames demonstrated similar behavior as chevron braced frames except for a slight strength drop after first buckling point. For high rise steel buildings, the suspended zipper braced frames demonstrated rather poor performance in terms of base shear capacity compared to chevron braced frame.

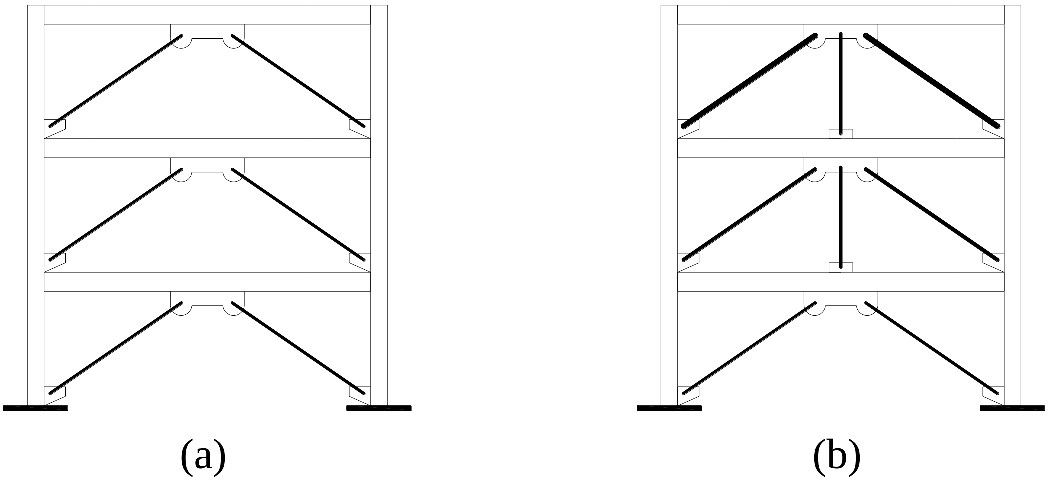


Figure 2.16 View of (a) chevron brace and (b) suspended zipper brace configuration (Özçelik and Sarıtaş, 2010)

Kim et al. (2008) presented two design methods of the zipper column to salvage inverted-V braced frames and evaluated based on a case study. From the capacity design perspective, it was pointed out that it was reasonable to design the zipper column to be elastic for the maximum forces imposed by the braces during cyclic yielding and buckling. But, the use of rigorous capacity design procedure was too conservative for most of the zipper columns since the braces in each story would not buckle simultaneously. Considering this, a simple static method which simulated the zipper column at each storey for the unbalanced load coming from just one storey below was first proposed. This method aimed at invoking at least two-storey buckling mechanism. The dynamic design method was also suggested considering both the effect of the brace slenderness and higher mode effects on the postbuckling behavior by incorporating the refined physical theory brace model and the modal pushover analysis in the design procedure. This method was theoretically more appealing but much more effort-demanding. In Figure 2.17, the postbuckling vertical balance force distribution at a joint is shown. Inelastic dynamic analyses for 15-storey frame building showed that both the static and dynamic design methods proposed led to significantly improved seismic performance as compared to the frame without zipper column. The dynamic method showed only slightly improved (or more uniform) distribution of interstorey drifts over the building height. The simple static design method proposed equally worked well in this case study. Further studies based on extensive inelastic dynamic analysis with including diverse structural configuration and earthquake input were also reported to be needed to critically evaluate the effectiveness of the two suggested design methods.

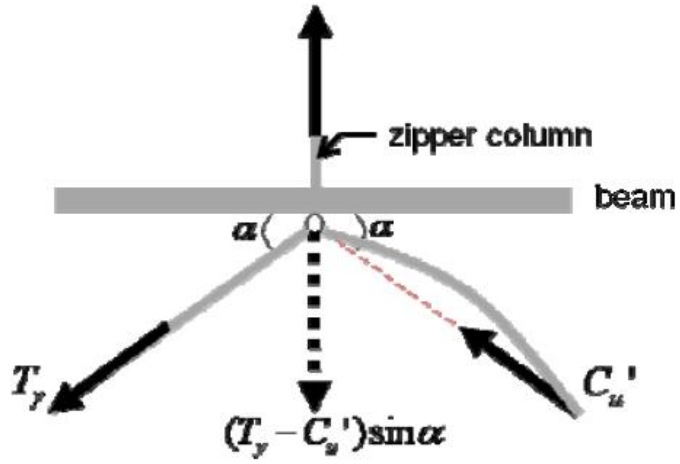


Figure 2.17 Postbuckling vertical unbalance force (Kim et al., 2008)

Zahrai et al. (2013) studied the seismic behavior of the braced frames with zipper-struts. They utilized a finite-element method for the numerical modeling of the zipper-strut-equipped model. The purpose of using the finite-element method was evaluating the behavior of shear links. Moreover, the study was examined the relationship between the shear links and zipper-struts. Therefore, the models were experienced nonlinear static (Pushover) analysis in two configurations; one with moment-resisting connections and other with pinned connections. As a result, additional zipper-strut in the systems increased the ductility coefficient for the model structures. Furthermore, the models were evaluated by means of the time-history analysis under the scaled earthquake records because of investigating the energy parameters of the models. The zipper-strut-equipped system showed a stronger tendency to form shear links. The zipper-strut-equipped system had also greater dissipation capacity in plastic zone.

Yang et al. (2009) proposed a hybrid model in which the seismic response of concentrically braced steel frame as named the suspended-zipper-braced frame was investigated. They evaluated a hybrid simulation for testing the model structures. They also investigated experimentally performance of the suspended-zipper-braced frame considering the actual hybrid model. Therefore, simulated and actual models were compared with each other determining the correctness of the hybrid simulation. In addition, it was used an energy-based approach for measuring the errors. The results showed that the deployed hybrid simulation method could be used to accurately model the seismic response of a complex structural system such as the zipper-braced frame. The simulation methods could also model the seismic response of the complex structural systems with similar accuracy. Therefore, the study indicated good agreement. The seismic response of the system was identified by the simultaneous yielding and buckling of the braces along the height of the frame. At the top of the frame, the resulting vertical component of the unbalanced brace forces was transferred and then to foundation through the frame columns. The hybrid model demonstrated the expected redistribution of force along the frame height although most of the frame inelastic deformation occurred in the first story. The results indicated good match and hybrid simulation showed outstanding calibration when compared with purely analytical simulation. Figure 2.18 reveals the details of the model structures.

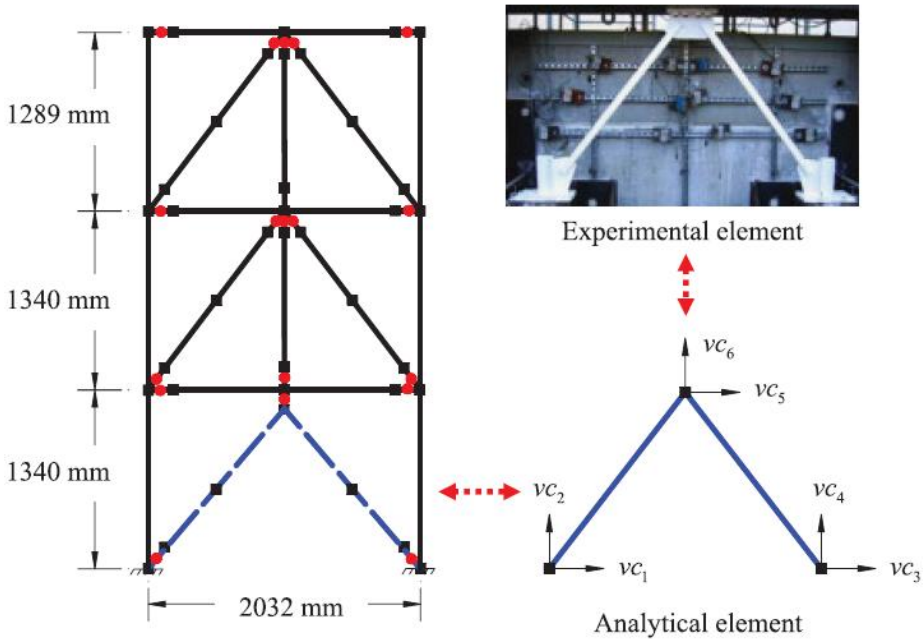


Figure 2.18 Dimensions of the $\frac{1}{3}$ scale model and hybrid model components (Yang et al., 2009)

Stavridis et al. (2010) modeled a three-storey suspended zipper steel frame to evaluate the seismic performance of the model structure. For this analytical and experimental studies were taken into account. The frame was an example of concentrically braced frames. The system transferred the unbalanced forces stimulated on the beams owing to the buckling of the lower-storey braces with zipper struts. The experimental study was used the hybrid test technique. In their study, only the bottom-storey braces of the three-storey frame were physically tested while the behavior of the other of the frame was modeled using a general structural analysis software. The calibration of the computer model for the analytical substructure as well as for the entire frame was embraced, and also this study included the selection of an appropriate damping matrix, and the modeling of the buckling behavior of the braces and bracing connections. The hybrid tests were confirmed the analytical model of the total frame and when the braces yielded and buckled, the analytical model of the frame was able to accurately capture the material and geometric nonlinearities that developed.

The suitability hybrid testing in improving analytical models was also illustrated in the study of Stavridis et al. (2010). It was provided information that not to be acquired from only analytical study with hybrid testing enhanced analytical models and modeling assumptions. As a result, the experimental study demonstrated the braces were distributed nonlinearity over the first two stories as intended in the design by the suspended zipper frame and did not have catastrophic damage under the design seismic excitations although the system had the significant inelastic deformations. Figure 2.19 shows the model structures as well as test arrangement.

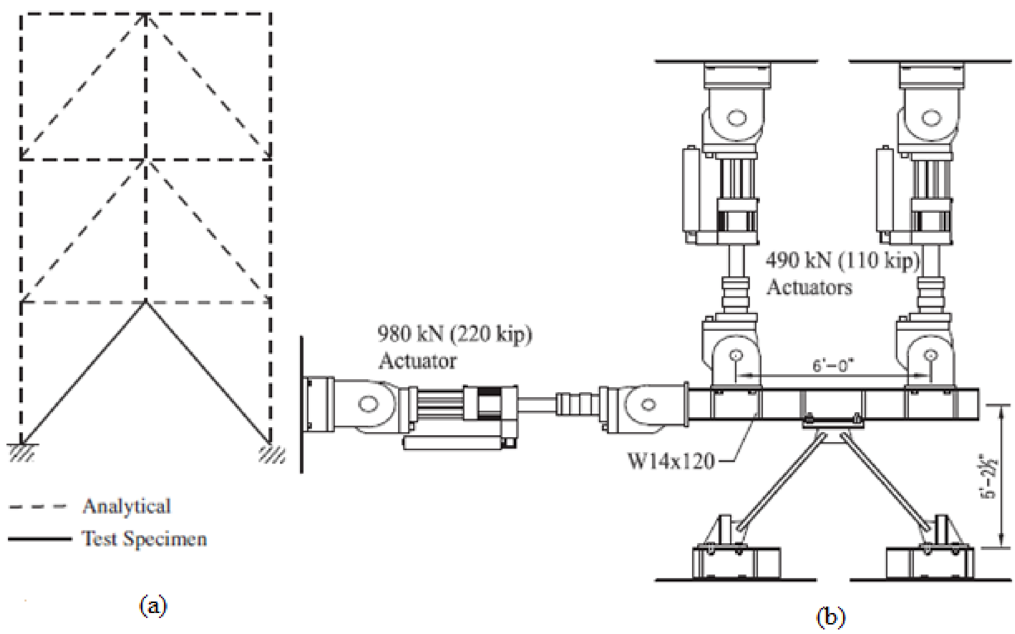


Figure 2.19 View of (a) structural partitioning and (b) test setup for hybrid tests (Stavridis et al., 2010)

Yang et al. (2008b) conducted on an experimental pushover test a special inverted-V-braced steel frame with zipper struts. The model was one-third-scale model of a special inverted-V-braced steel frame with zipper struts. Zipper struts were added at the intersections of the braces above the first floor as vertical members. And zipper struts were designed to carry upward the unbalanced loads resulting from buckling of the braces. The analyses were conducted considering two-dimensional and one three-dimensional frame models. Therefore, the study illustrated the capability of zipper struts for stimulating buckling in all stories except the top one, redistributing the loads in the structure and minimizing strength losses. Figure 2.20 shows the sidesway mechanisms for different frame systems.

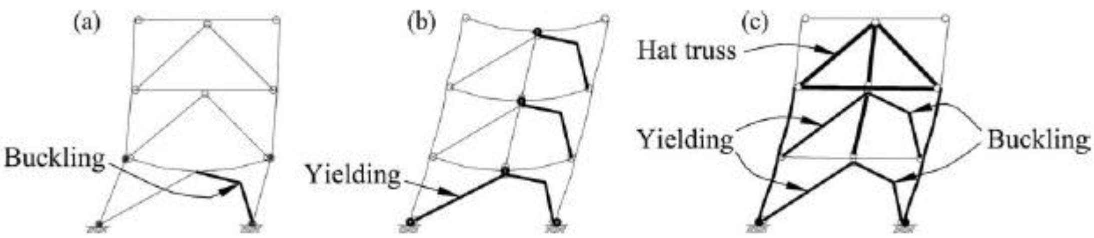


Figure 2.20 Sidesway mechanisms for the frame with (a) conventional inverted-V, (b) conventional zipper, and (c) zipper having hat truss (Yang et al., 2008b)

Nouri et al. (2009) examined the applicability of the suspended zipper system for the seismic rehabilitation of steel buildings. Inverted-V braced frames were modeled as 3-, 6-, 9-, and 12-storey steel frames. In such a case, the structures needed to be rehabilitated and zipper-struts were used for retrofitting the frame buildings. It was reported that the suspended zipper system demonstrated remarkable effect in case of 3-, 6-, and 9-storey inverted-V braced frames. The use of the suspended zipper braces increased lateral resistance of the structures up to life safety performance level. On the other hand, for high-rise buildings (such as 12 storey frame), this system did not show good performance. To overcome this drawbacks, the brace bay could consist of small “units” over the height of the entire structure, which each of them was a zipper-braced bay with a few stories. By using this method, the lateral resistance of 12 storey inverted-V braced frames was increased up to safety life level.

CHAPTER 3

METHODOLOGY

3.1 Analytical Model of Structures

As frame models, four and eight storey reinforced concrete (RC) buildings were used in order to compare the seismic response of the original structures and retrofitted structures considering the addition of inverted-V and zipper braced frames. The buildings were modeled considering a serious of planar frames connected at each floor level by rigid diaphragms. Therefore, only two dimensional analyses was conducted. The column foundations were considered as fixed for all cases. These frames were designed as three bays on each direction and the structures were considered as regular in shape and symmetric in plane in order to carry out the analysis on two-dimensional models which ease the interpretation of the results of analysis. Typical floor plan and elevation of the case study RC buildings are given in Figures 3.1, 3.2, and 3.3.

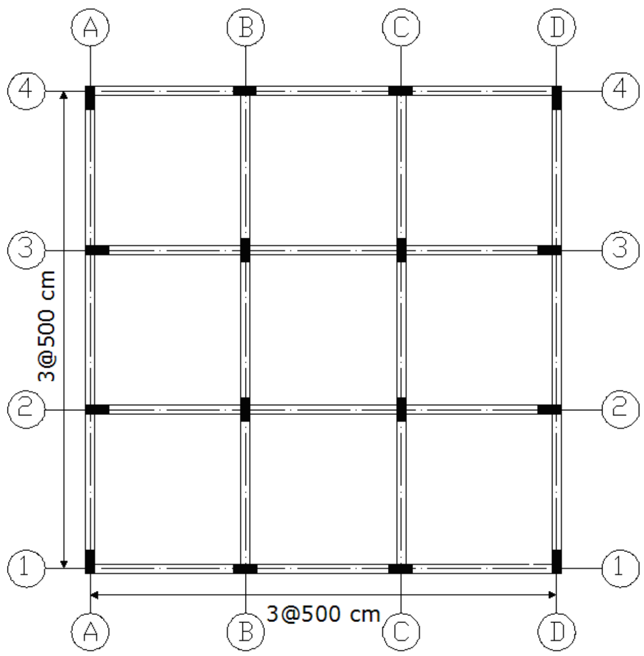


Figure 3.1 Plan view of the RC buildings

The selected structures (four-storey and eight-storey buildings) were designed in accordance with TS 500 (2000). Each storey had a height of 3 m and all slap thicknesses were 12 cm. The exterior and interior frames of the buildings comprised three bays. The dimensions of the beams were 60 cm in height and 25 cm in width. The long sides of the columns were placed at the exterior axes, however, and in the interior axes, the short directions of the columns were located the direction parallel to x axis. At four-storey building, the dimensions of the columns were the same dimensions (25x55 cm) at all storeys. For the eight-storey buildings, the dimensions of the columns were different for each storey. They were varied from first storey to fourth storey as shown in Table 3.1. The design live load and additional dead load for the building were taken as 2.00 kN/m^2 and 2.88 kN/m^2, respectively. Concrete and both longitudinal and transverse reinforcing steel classes were described as C16 and S220, respectively.

Table 3.1 Dimensions of the columns in the 8 storey buildings

Storey level	1	2	3	4	5	6	7	8
Dimensions of the columns (cm)	30x70	30x70	25x65	25x65	25x65	25x60	25x60	25x60

In the retrofitting of the RC buildings, the structural steel braces in the configuration of inverted-V and zipper bracing were used. For the brace elements, pipe section was utilized. At four-storey building, the dimension of inverted-V was as follows: outside diameter was 8.9 cm and wall thickness was 0.8 cm. That for the zipper element on the top floor was as: outside diameter was 7.3 cm and wall thickness was 0.6 cm. The other zipper elements were as: outside diameter was 6 cm and wall thickness was 0.6 cm. For eight-storey buildings, the dimension of inverted-V was as follows: outside diameter was 11.4 cm and wall thickness was 0.8 cm. That for the zipper element on the top floor was as: outside diameter was 8.9 cm and wall thickness was 0.8 cm. The other zipper elements were as: outside diameter was 7.3 cm and wall thickness was 0.6 cm. The modulus of the elasticity of steel used was 200 GPa and its yield stress was 345 MPa. In two retrofitting cases, brace system was inserted in the middle bays of each frame considering inverted-V and zipper configurations. All the steel elements have the same cross section and material properties. For the three-bay retrofitted RC frame models with two different numbers of storeys (4 and 8) were considered as shown in Figure, 3.2 and 3.3, respectively. The buildings were assumed to have a uniform mass distribution over the height and non-uniform lateral stiffness distribution.

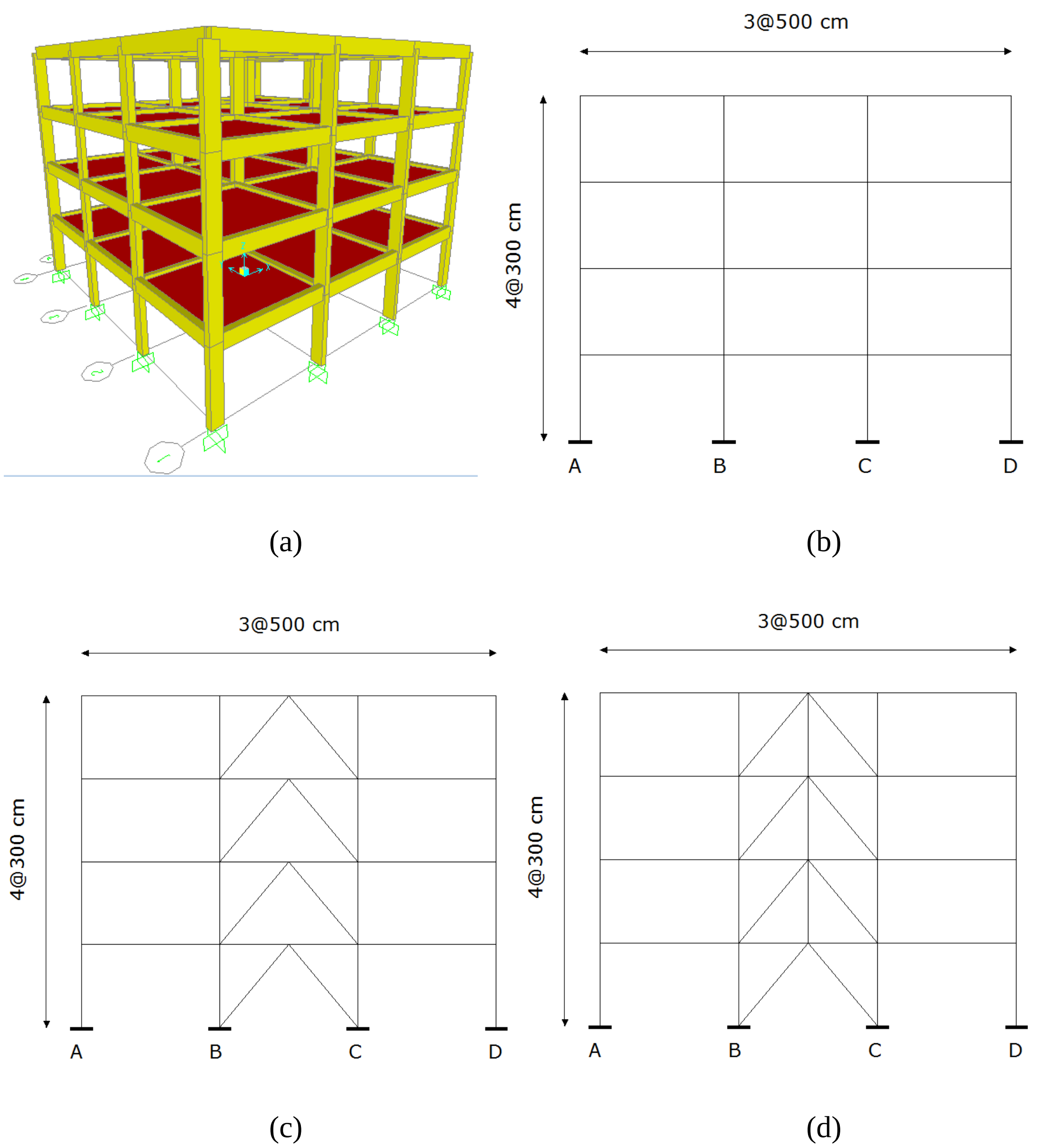


Figure 3.2 4 storey RC building a) 3-dimensional view, b) elevation of the frame, c) the frame retrofitted with inverted-V brace, and d) the frame retrofitted with zipper brace

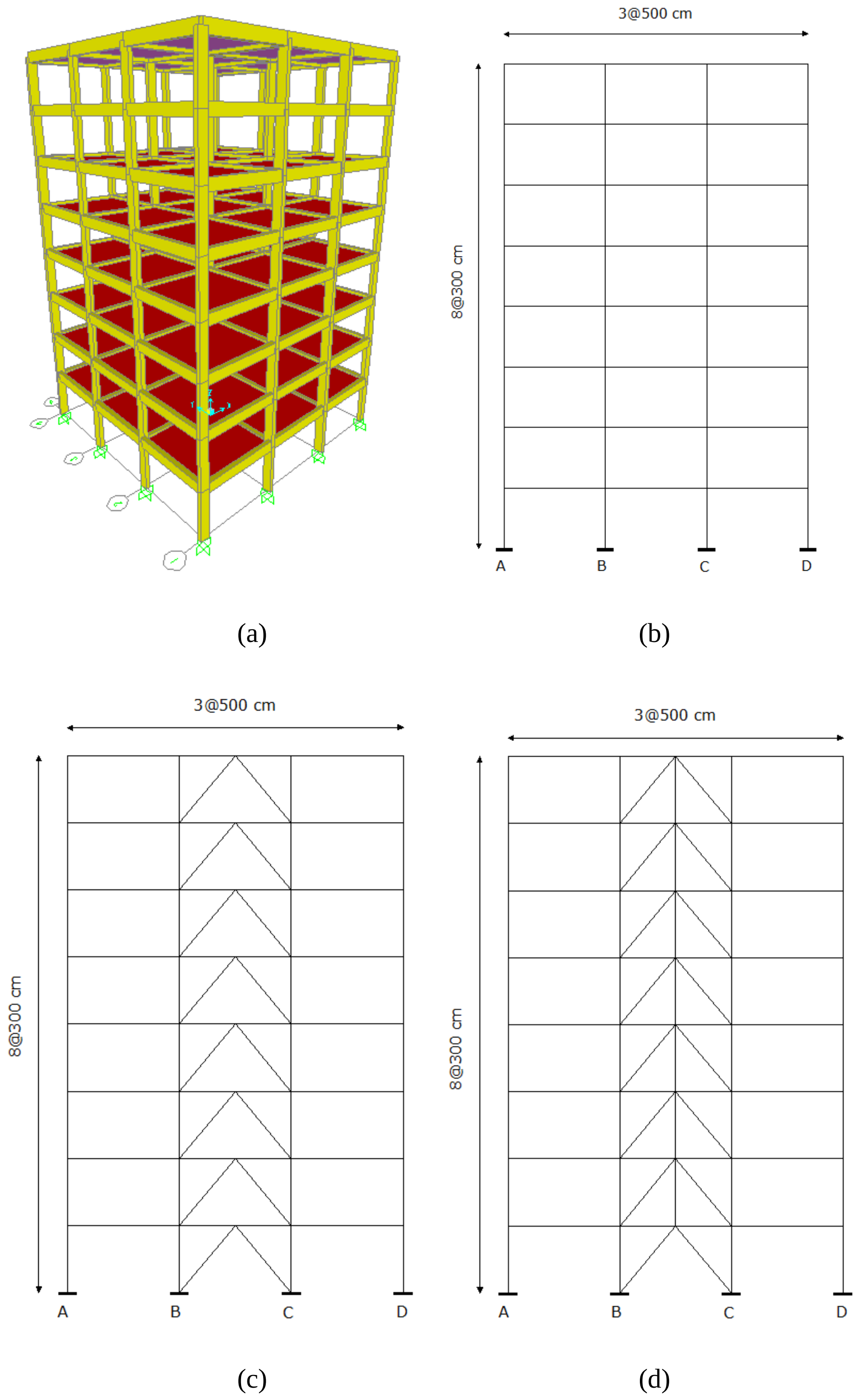


Figure 3.3 8 storey RC building a) 3-dimensional view, b) elevation of the frame, c) the frame retrofitted with inverted-V brace, and d) the frame retrofitted with zipper brace

3.2 Nonlinear Analysis Methods

Nonlinear static and dynamic analyses were carried out using the finite element program of SAP 2000 non-linear version 14 (CSI, 2009) to determine the seismic performance of the existing RC frames and those with inverted-V and zipper braced frames. Nonlinear static pushover analysis is the most extensively used method to evaluate the nonlinear behavior of the buildings. Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. It consists of series of sequential elastic analysis, superimposed to approximate, a force-displacement curve of the overall structure. The equivalent static lateral loads nearly represent earthquake induced forces. In this study, a plot of the total base shear versus displacement in a structure was formed by this analysis that would reveal any premature failure.

The analysis was conducted up to failure, hence it was possible to specify yielding point of the system. On the frames, the plastic rotation was also monitored, and lateral inelastic forces versus displacement response for the complete structure were analytically computed. According to FEMA 356 (2000), the hinge properties of the structural components were determined considering component type and failure mechanism. After defining the plastic hinge properties in the model, the structures were subjected to monotically increasing lateral forces until a specified displacement was reached. The capacity curves related to base shear versus roof displacement for 4 and 8 storeys structures with inverted-V and zipper braces were achieved at the end of the pushover analysis. Subsequently, the target displacements which represented the maximum displacement likely to be experienced during the design earthquake were also computed.

In order to specify the actual nonlinear behavior of buildings, besides carrying out pushover analysis, nonlinear time history analysis was performed. In this method, the buildings were subjected to real ground motion record. Hence, inertial forces were determined from the ground motions and the response of the structure either in deformations or in forces were calculated as a function of time. The seismic behavior of the original, inverted-V, and zipper braced frames were investigated under different earthquake ground accelerations. In the nonlinear time history analysis, analytical models consisting the nonlinear behavior of the structural members were subjected to earthquake ground accelerations. For the nonlinear dynamic analysis of the frames, a set of natural ground accelerations were generated as spectrum compatible were utilized (PEER, 2011). The design code spectrum and elastic spectra of the scaled natural ground accelerations are given in Figure 3.4. Moreover, the characteristic properties of the natural ground motions such as the magnitude (M_w), the peak ground acceleration (PGA), the peak ground velocity (PGV), the peak ground displacement (PGD), and the characteristics of the site where acceleration recorded are listed in Table 3.2.

Table 3.2 Characteristics of the selected ground accelerations

Earthquake record	Year	Magnitude (M_w)	V_{s30} (m/s)	PGA (g)	PGV (cm/s)	PGD (cm)	Scale factor
Morgan Hill	1984	6.19	239.7	0.376	58.2055	26.3717	10.391
Erzincan	1992	6.69	274.5	0.480	51.7942	18.8926	1.178
Hector Mine	1999	7.13	301	0.5065	92.244	77.617	19.901

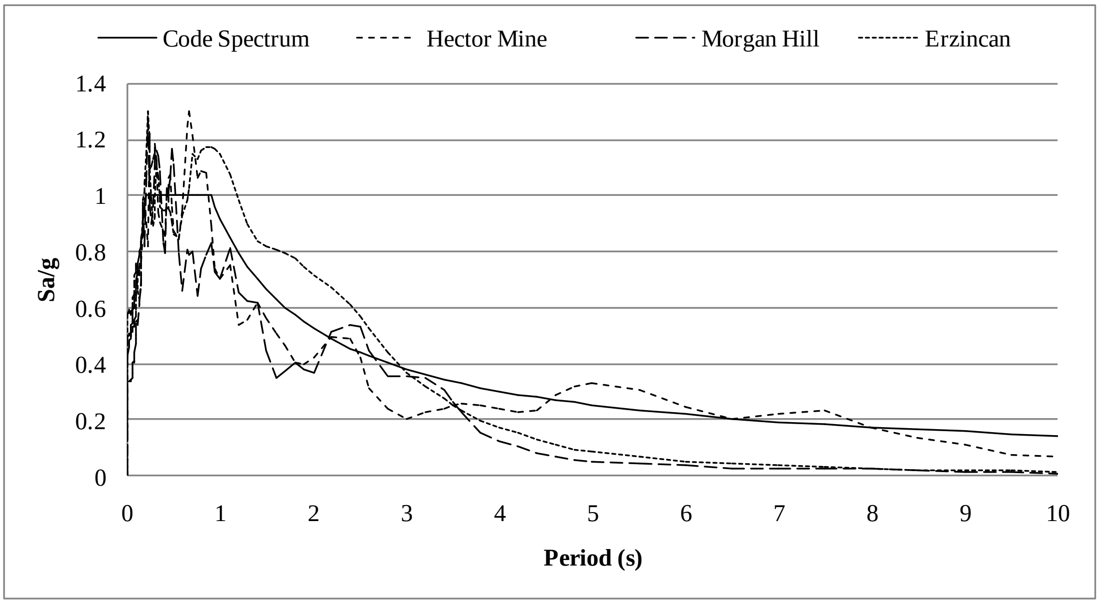


Figure 3.4 Elastic spectral accelerations of the ground motions

In nonlinear static and nonlinear time-history analyses, the post-yield behavior by assigning concentrated plastic hinges to frame was simulated. Elastic behavior occurred over member length, and then deformation beyond the elastic limit occurred entirely within hinges, which were modeled in discrete locations. Inelastic behavior was obtained through integration of the plastic strain and plastic curvature which occurred within a specified hinge length, typically on the order of member depth (FEMA 356, 2000). To capture plasticity distributed along member length, a series of hinges were modeled. Multiple hinges were also coincide at the same location. Plasticity were associated with force-displacement behaviors (axial and shear) or moment-rotation (torsion and bending). The nonlinearity was taken into account by adopting plastic hinges with hysteretic relationships based on FEMA 356 (2000) at each end of the beam and column members. For the column members, axial force and biaxial moment hinges (PMM) and for the beams, flexural moment hinges (M3) were considered. Table 3.3 shows the dynamic properties of the existing RC frames and those retrofitted with inverted-V and zipper braces.

Table 3.3 Dynamic properties of the existing, inverted-V, and zipper braced frames

Type of frame	T_1 (s)	T_2 (s)	T_3 (s)
4 storey existing frame	0.38	0.12	0.07
4 storey inverted-V braced frame	0.24	0.08	0.05
4 storey zipper braced frame	0.24	0.08	0.05
8 storey existing frame	0.67	0.22	0.13
8 storey inverted-V braced frame	0.44	0.15	0.08
8 storey zipper braced frame	0.44	0.15	0.08

At the four storey buildings, free vibration periods for existing frame, inverted-V and zipper braced frames were determined. The first three modes of free vibration period for original frame was calculated as T_1 = 0.38 s, T_2= 0.12 s, and T_3 = 0.07 s. However, the free vibration periods for inverted-V and zipper braced frames were very close to each other like that T_1 = 0.24 s, T_2= 0.08 s, and T_3 = 0.05 s. At the eight storey buildings, the first three modes of free vibration period for original frame was calculated as T_1 = 0.67 s, T_2= 0.22 s, and T_3 = 0.13 s. As to free vibration periods for inverted-V and zipper braced frames were very close like that T_1 = 0.44 s, T_2= 0.15 s, and T_3 = 0.08 s. As a result, with the addition of steel braces, stiffness of system was increased and free vibration period of the system was reduced.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 General

In this section, the results for the existing frames, frames retrofitted with inverted-V and zipper braces obtained nonlinear static and time-history analyses were given and discussed comparatively. In this study, a total of 6 different cases were considered and structural performance of original frame (OF), inverted-V braced frame (IVBF), zipper braced frame (ZBF) having different number of storeys under the effect of earthquake loadings were evaluated. Performance characteristics in terms of capacity curves, interstorey index, variation of storey displacement, roof drift and inter-storey drift ratio were given below.

4.1.1 Capacity Curves

The capacity curves (pushover curves) were evaluated for different frame types. Figures 4.1 and 4.2 show the comparison of the capacity curves of original systems, frames with inverted-V and zipper braces. The capacity curves resulting from the analysis were obtained for the original RC frame and retrofitted cases. The significant improvement in the seismic performance of the original frame was observed when the suitable retrofitting strategy was selected. As expected, the use of different retrofitting cases resulted in different seismic performance levels.

As seen from the figures, the frame with inverted-V and zipper braces had considerably greater capacity in comparison to the original frame. For example, for the four storey building, the maximum base shear of the original frame was about 118 kN while that of the retrofitted frames with inverted-V, and zipper braces were nearly 360 and 405 kN, respectively. This implied about 3 and 3.4 times higher lateral load carrying capacity for the retrofitted cases in comparison to the original frame. For the eight storey building, the maximum base shear of the original frame was about 164 kN while that of the retrofitted frames with inverted-V, and zipper were approximately 612 and 699 kN, respectively. This indicated that the retrofitting cases had about 3.73 and 4.26 times higher lateral load carrying capacity than the original frame. When comparing inverted-V and zipper braced frame, zipper braced frame had higher lateral load carrying capacity than inverted-V braced frame. However, as observed Figure 4.1, the capacity curves of the frames with inverted-V braced and zipper braces, lateral load carrying capacity from the peak roof drift ratio about 0.18 and 0.26 decreased because of diagonal elements of buckling at the four storey buildings. Moreover, both types of initial stiffness of the structure with the addition steel braced system was seen to increase about 3-4 times compared to initial stiffness of the existing structure.

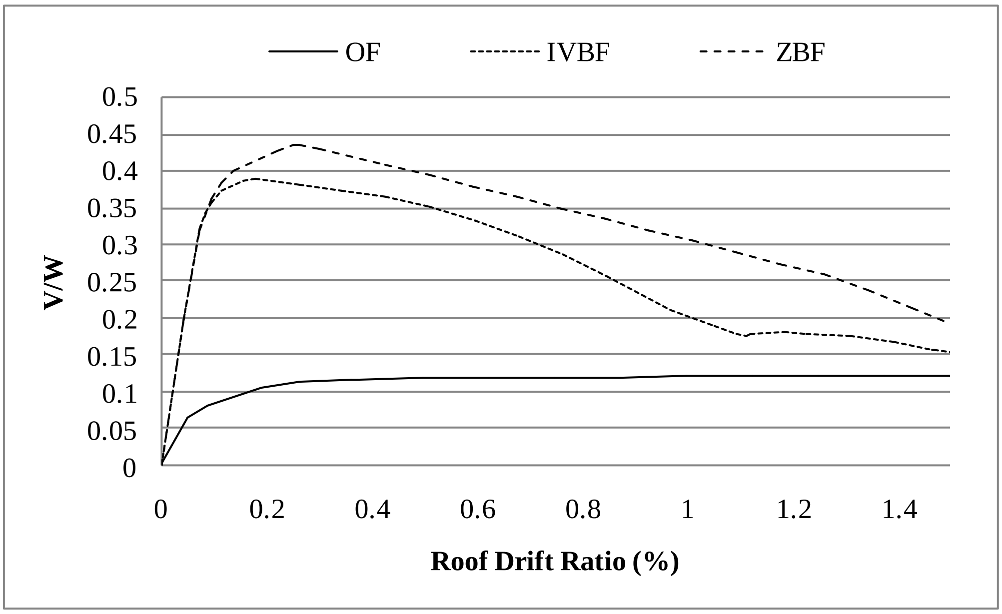


Figure 4.1 Capacity curves of 4 storey original and retrofitted frames in terms of the base shear/total weight versus roof drift ratio

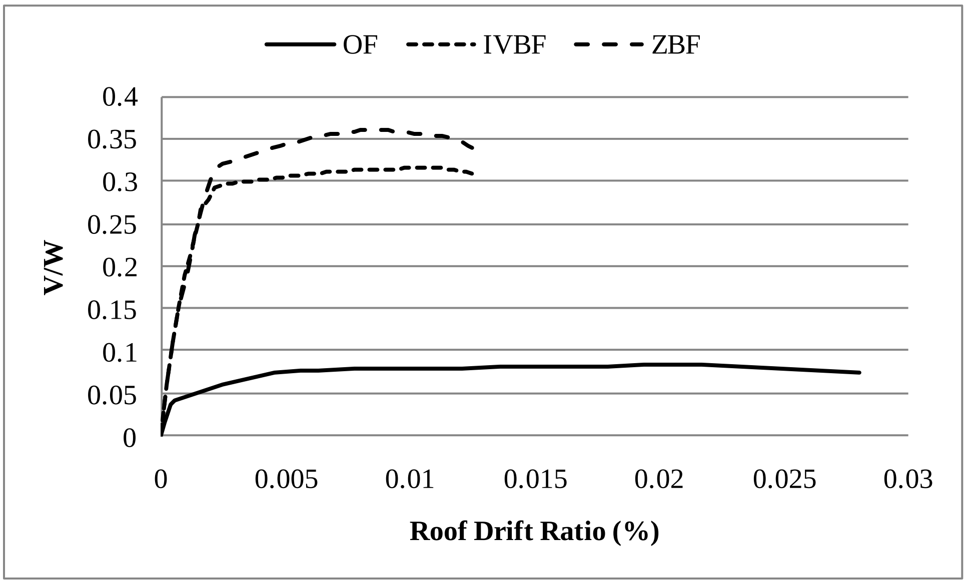


Figure 4.2 Capacity curves of 8 storey original and retrofitted frames in terms of the base shear/total weight versus roof drift ratio

It was clearly understood that the capacity curves in all circumstances for inverted-V and zipper braced frames were bilinear since at the beginning the structure was globally in the elastic stage and provided a linear elastic slope, and then when the base shear was exceeded, some structural members (beams and columns) would crack, some structural members (diagonal bracing) yielded and triggered to a change in the slope of the capacity curve.

Moreover, the behavior of inverted-V and zipper brace configurations is similar in the elastic region, however, the behavior in the inelastic region was varied. For example, the zipper braced frame was observed to be capable of more lateral load carrying and energy absorption capacity than inverted-V braced frame due to the provision of zipper elements distributed load more balanced between the braced members.

4.1.2 Interstorey Index

The maximum interstorey drift (δ_{max}) divided by the storey height (h) is defined as the maximum interstorey index. This index is a good indication of the damages experienced by the structural members.

The maximum interstorey index was assessed for existing frames and frames with inverted-V and zipper braces subjected to seismic excitations. Figures 4.3 and 4.4

compares the maximum interstorey index for 4 and 8 storey original and retrofitted frames, respectively. The retrofitting system showed a better performance compared to original one. It was also pointed out that the characteristic of the earthquakes used was very effective on the measured index.

Moreover, it was observed from the Figures 4.3 and 4.4 that there was a difference between the interstorey indexes of the existing and retrofitted frames. Therefore, the differences in the interstorey index for the retrofitted frames with inverted-V and zipper system were significantly smaller than the interstorey index for the existing frames. Furthermore, the retrofitting of RC frames with zipper system were performing better than those with inverted-V system.

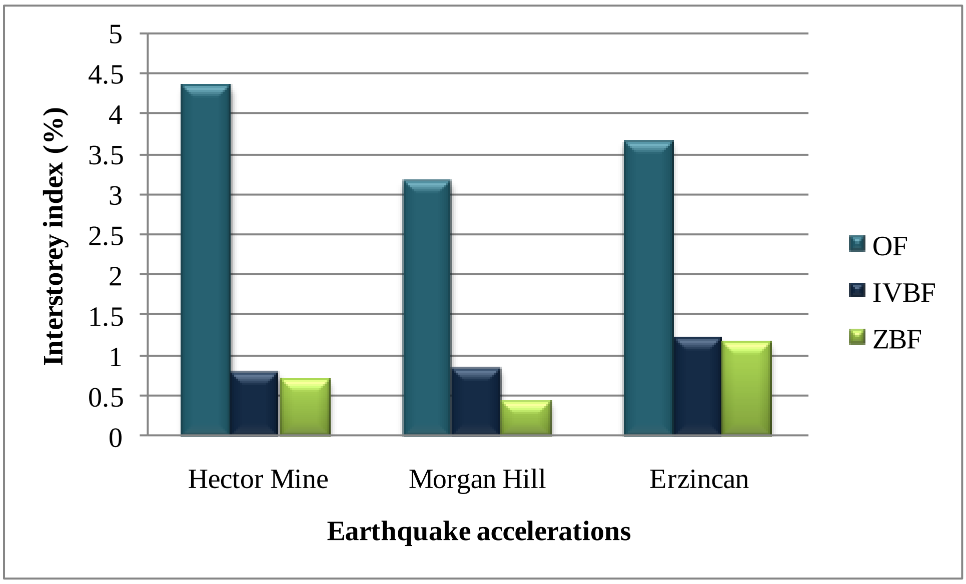


Figure 4.3 Maximum interstorey indexes for 4 storey RC frames

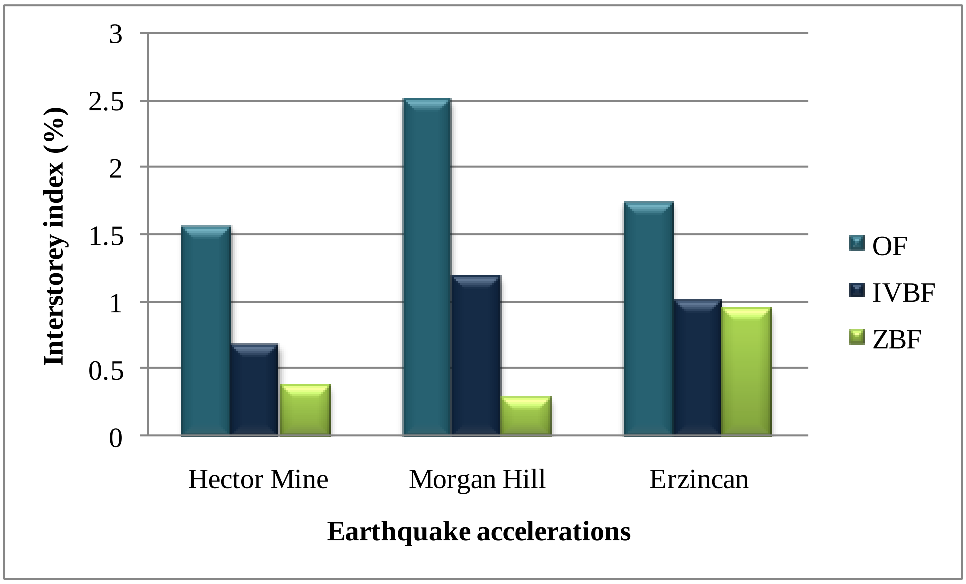
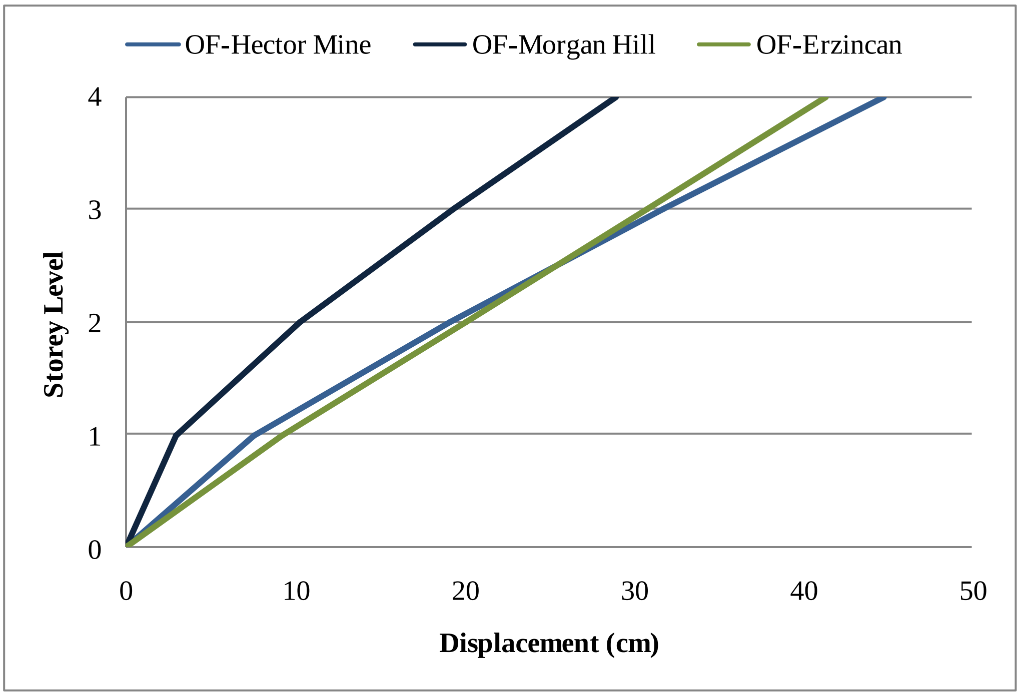


Figure 4.4 Maximum interstorey indexes for 8 storey RC frames

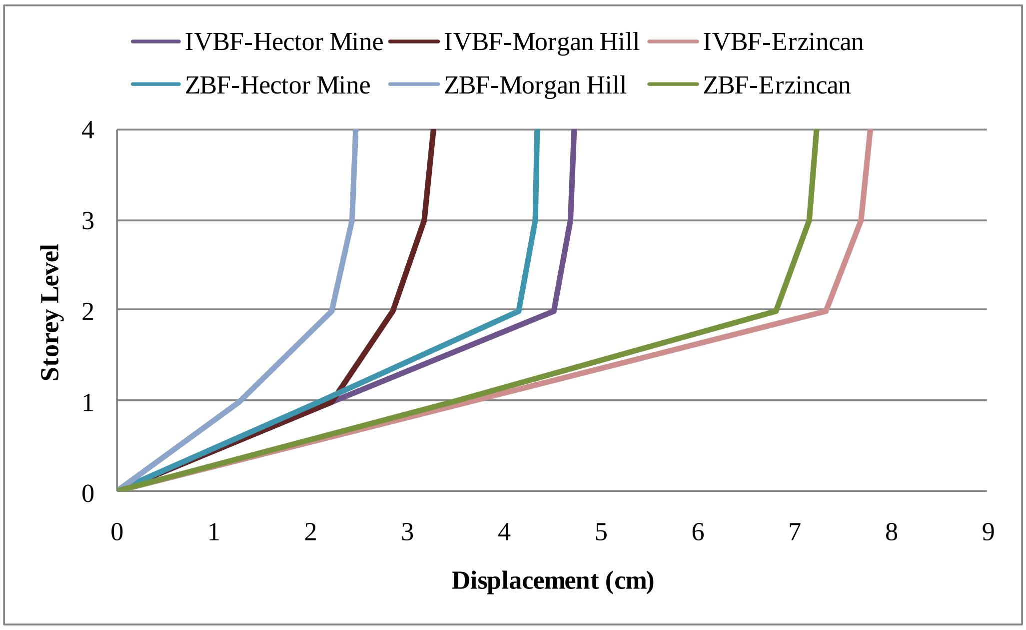
4.1.3 Variation of Storey Displacement

Figures 4.5 and 4.6 show the variation of storey displacements for the 4 and 8 storey original and retrofitted RC frames, respectively. As seen from the figures, the structures were subjected to three different ground motion records. It was observed that all structures under investigation were considerably affected by the given earthquakes. Moreover, the use of inverted-V and zipper braces for the purpose of retrofit decreased remarkably the value of the maximum storey displacements as compared to original frames, especially in the case of retrofitted with zipper systems.

The maximum storey displacement was also affected by the number of stories and frame type. For example, in the case of four storey frame with zipper system, the maximum storey displacement was smaller than other frames, by increasing number of storeys the maximum storey displacements were also increased.

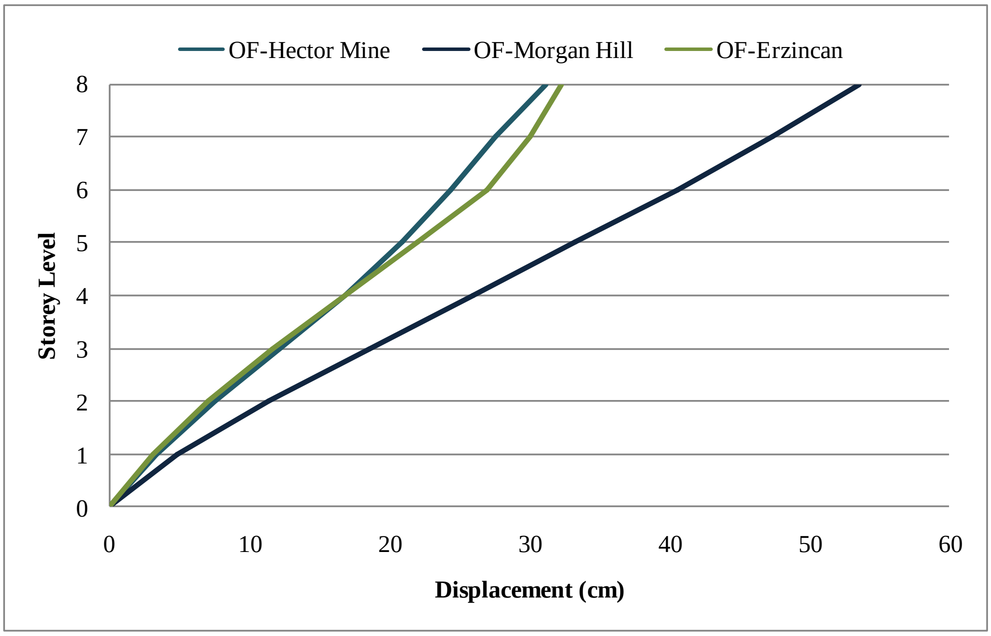


(a)

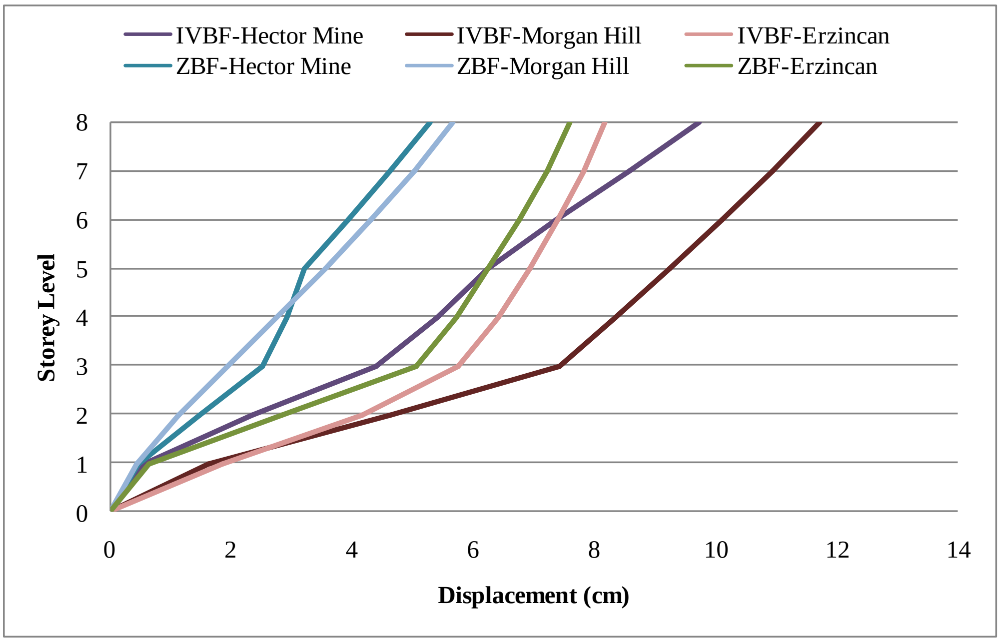


(b)

Figure 4.5 Variation of storey displacement in (a) the 4 storey original and (b) inverted-V and zipper braced frames



(a)



(b)

Figure 4.6 Variation of storey displacement in (a) the 8 storey original and (b) inverted-V and zipper braced frames

According to the findings obtained from the nonlinear time history analysis, both the braced frame systems had significantly lower storey displacements than the existing frame under the all earthquake ground motions. For example, as seen in Figure 4.5, at the four storey building, under the Erzincan earthquake, the maximum displacement of the existing frame was obtained as 41.3 cm while the maximum displacement of the inverted-V and zipper braced frames was obtained as 7.79 cm and 7.24 cm, respectively. In the case of 8 storey building (see Figure 4.6), under the Erzincan earthquake, the maximum displacement of the existing frame was achieved as 32.3 cm while the maximum displacement of the inverted-V and zipper braced frames was obtained as 8.2 cm and 7.6 cm, respectively. As a result, it was appeared that the use of the zipper braced frames was better than the inverted-V braced frame and in the case of using braced frames with inverted-V and zipper systems were decreased significantly the maximum displacement of the original frames.

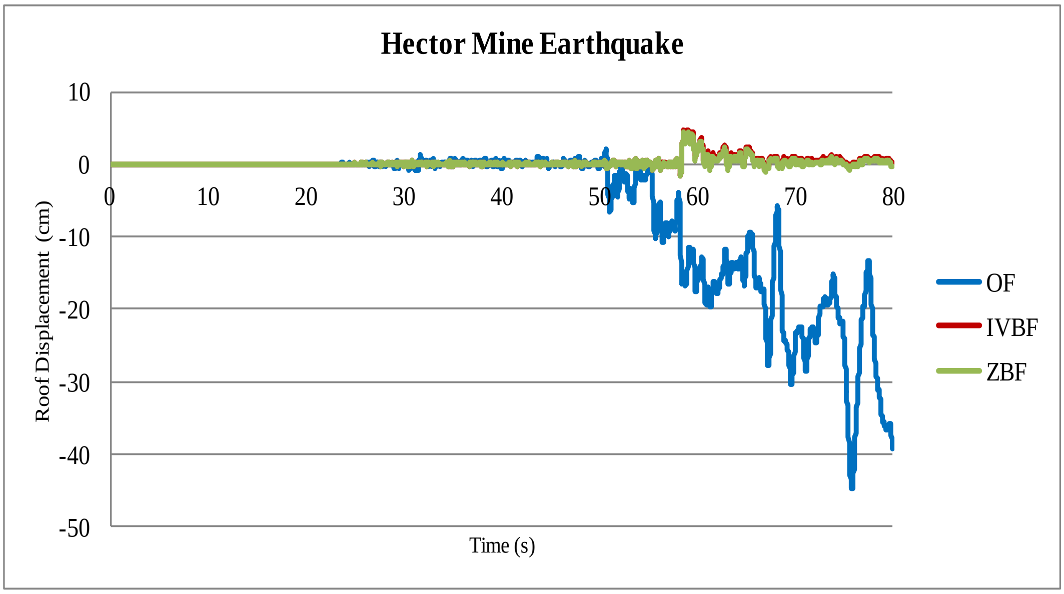
4.1.4 Roof Displacement Time History

Figures 4.7 and 4.8 show the time history of the roof displacement of the original frames and the braced frames under the three different seismic excitations (Hector Mine, Morgan Hill, and Erzincan earthquakes) for the 4 storey and 8 storey structures, respectively. The involvement of the structural steel braces in the retrofitted frames remarkably decreased the value of roof displacement compared to the original frames, especially in the case of frames with zipper system. The maximum roof displacement was also influenced by the number of storey and frame type. Furthermore, using the zipper braced frames had more downward trend for the maximum storey displacement of the roof level than the inverted-V braced frames for all cases.

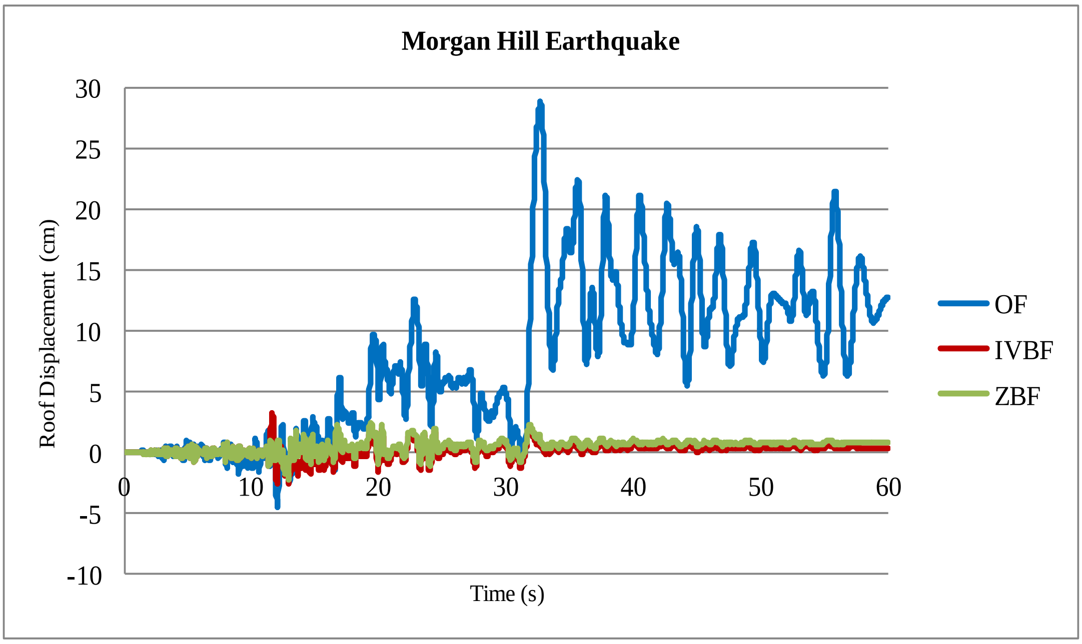
According to the results obtained from the nonlinear time history analysis, both of the braced frame systems decreased significantly roof displacement of the existing frame under the all earthquake ground motions. For example, as shown in Figure 4.7, for the four storey building, under the Hector Mine earthquake, the maximum roof displacement of existing frame was about 44.8 cm while the maximum roof displacement of the inverted-V and zipper braced frames was achieved as approximately 4.7 cm and 4.3 cm, respectively. In addition, as seen in Figure 4.8, for the eight storey building, under the Hector Mine earthquake, the maximum roof displacement of the existing frame was achieved as about 31.1 cm whereas the maximum roof displacement of the inverted-V and zipper braced frames was found as about 9.8 cm and 5.3 cm, respectively.

Therefore, it could be observed generally that the addition of brace systems reduced significantly the drifts in the frames. As seen the figures, the results showed that the use of zipper braced frames was better than that of the inverted-V braced frame and in the case of frames with inverted-V and zipper braces, the maximum roof drift demands were significantly smaller.

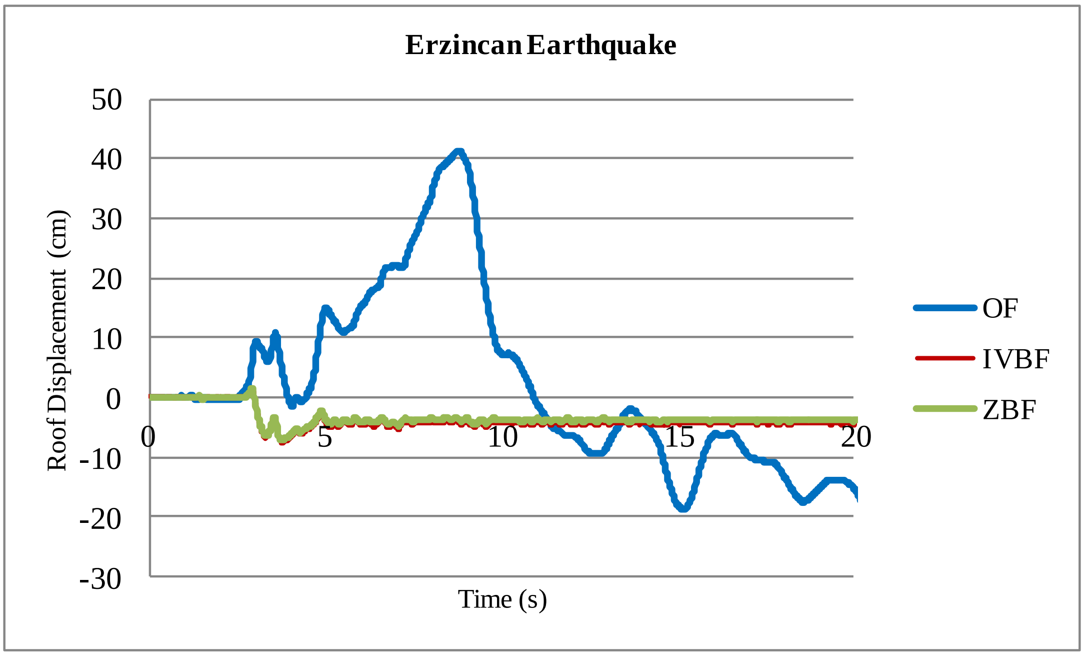
It was also evident from Figures 4.7 and 4.8 that the application of inverted-V and zipper braces into 4 and 8 storey buildings resulted in a marked reduction in the roof displacement up to about 91% and 89%, respectively. For example, for Morgan Hill and Erzincan earthquakes, the 4 and 8 storey retrofitted frames gave 82% and 76%, respectively. Moreover, the peak amplitude was 32.31 cm for the existing frame while minimum amplitude for the zipper brace frame occurred as 7.6 cm at the eight storey building for Erzincan earthquake. At this low level of drift, no damage was expected during this kind of a major earthquake. Moreover, all deformed shapes were given in Appendix A in order to compare the all cases.



(a)

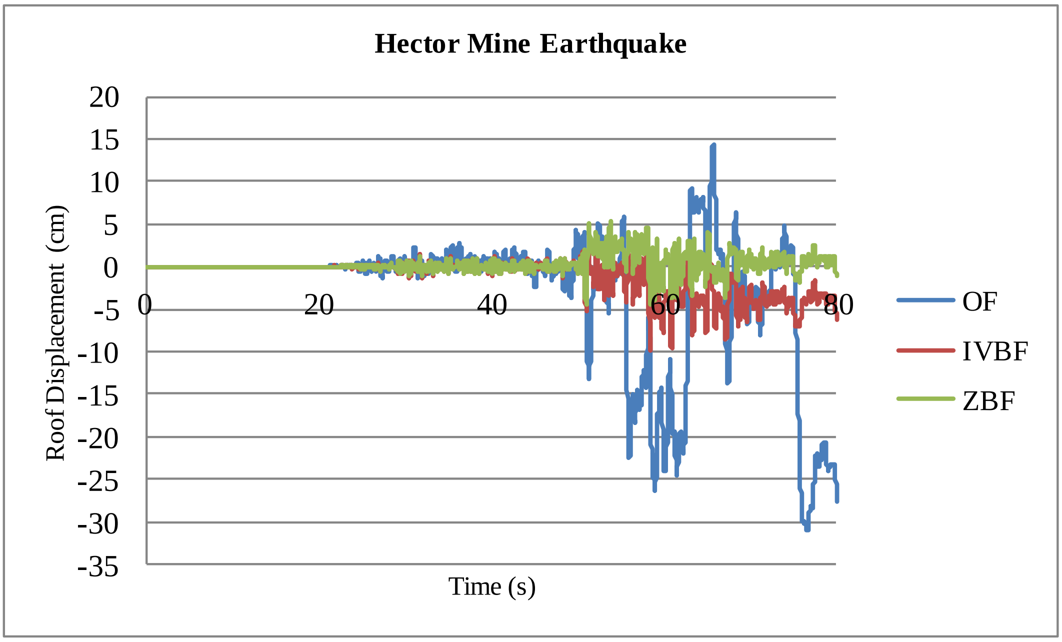


(b)

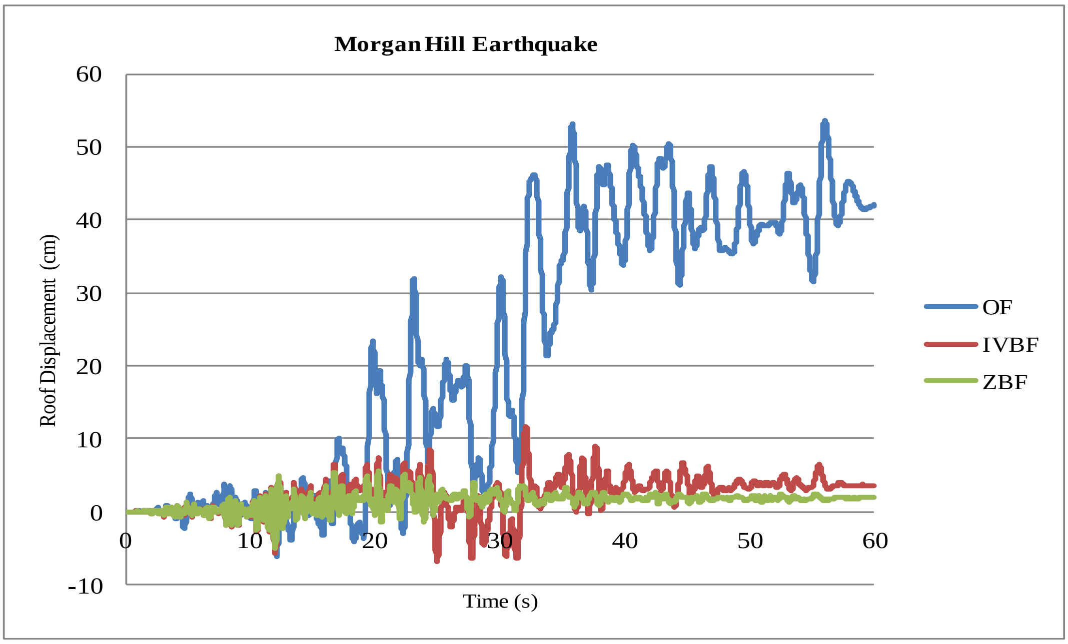


(c)

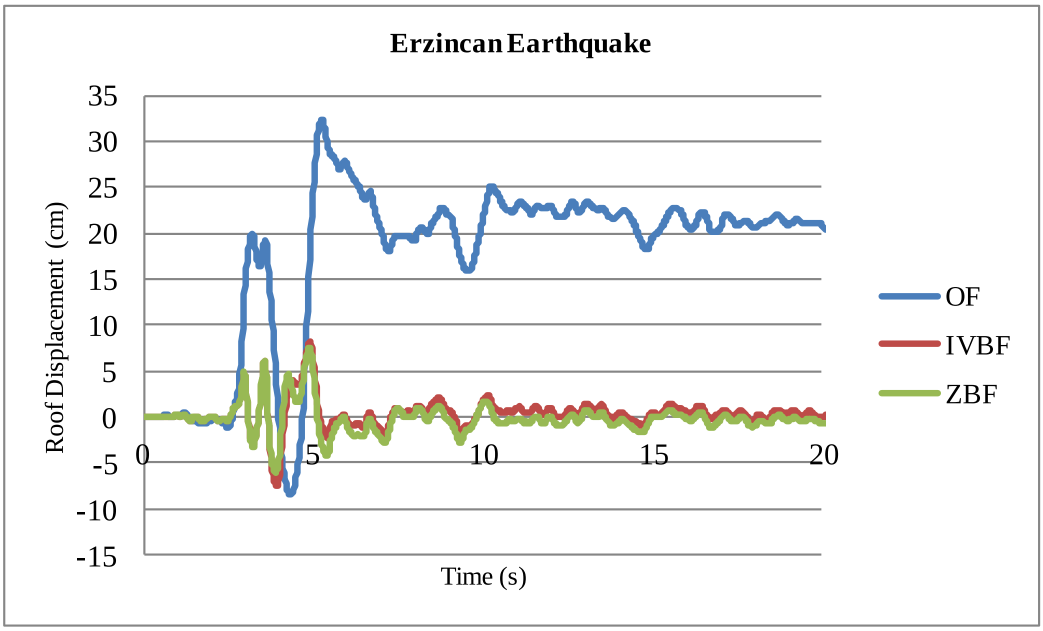
Figure 4.7 Roof displacement versus time for the original, inverted-V and zipper braced frames at the 4 storey structure: (a) Hector Mine earthquake, (b) Morgan Hill earthquake, and (c) Erzincan earthquake


Hector Mine Earthquake
Roof Displacement (cm)
20
15
10
5
0
-5
-10
-15
-20
-25
-30
-35
0
20
40
60
80
Time (s)
OF
IVBF
ZBF

(a)


Morgan Hill Earthquake
Roof Displacement (cm)
60
50
40
30
20
10
0
-10
0
10
20
30
40
50
60
Time (s)
OF
IVBF
ZBF

(b)



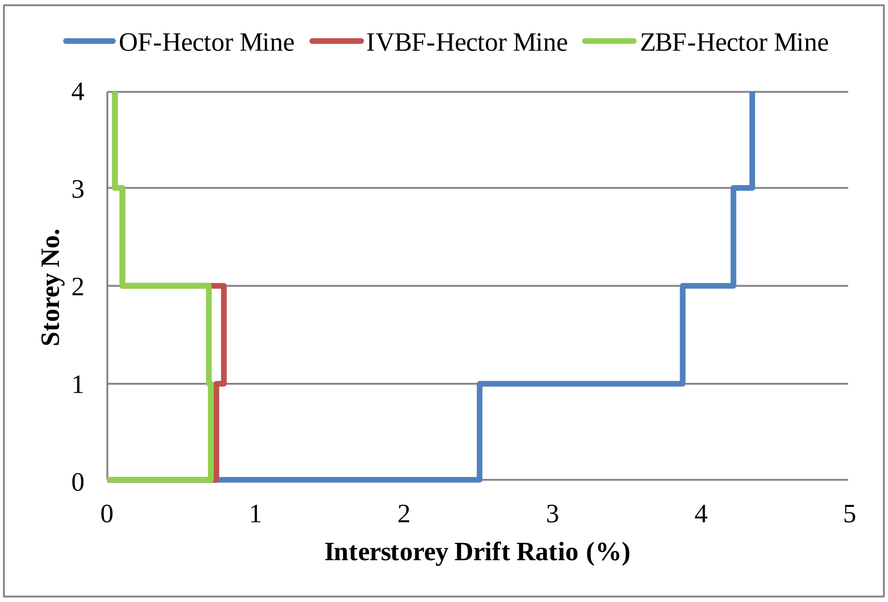
(c)

Figure 4.8 Roof displacement versus time for the original, inverted-V and zipper braced frames at the 8 storey structures: (a) Hector Mine earthquake, (b) Morgan Hill earthquake, and (c) Erzincan earthquake

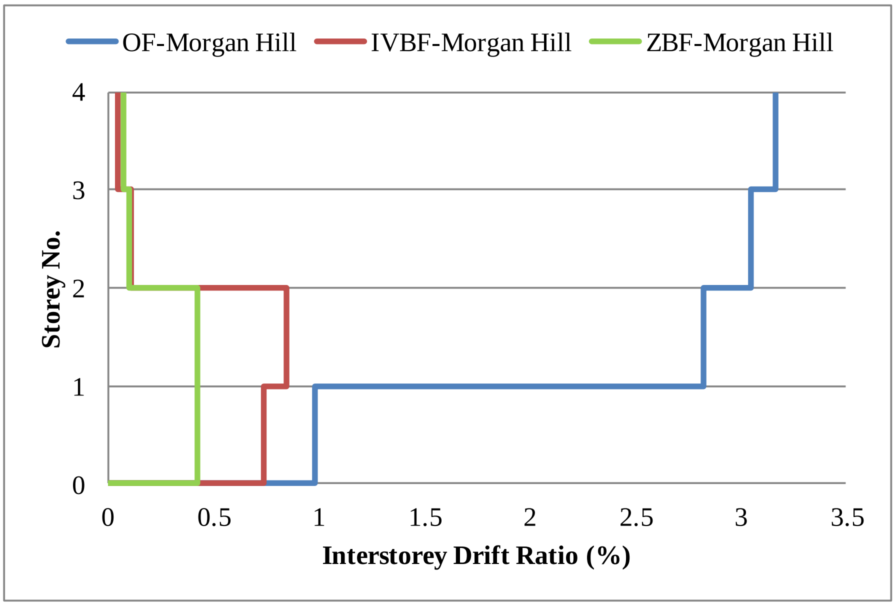
4.1.5 Interstorey Drift Ratio

The plots for the interstorey drift ratio of the original, inverted-V and zipper braced frames are given in Figures 4.9 to 4.12. According to the analysis of the results, using both the braced systems decreased significantly interstorey drift ratio of the existing frame under the all earthquake ground motions. For example, as seen in Figure 4.9, for the four storey building, under the Morgan Hill earthquake, the maximum interstorey drift ratio of the existing frame was achieved as 3.2% while the maximum interstorey drift ratio of the inverted-V and zipper braced frames was obtained as 0.84% and 0.42%, respectively. Moreover, as seen in Figure 4.11 for the eight storey building, under the Hector Mine earthquake, the maximum interstorey drift ratio of the existing frame was found as 1.56% while the maximum interstorey drift ratio of the inverted-V and zipper braced frames was obtained as 0.68% and 0.38%, respectively.

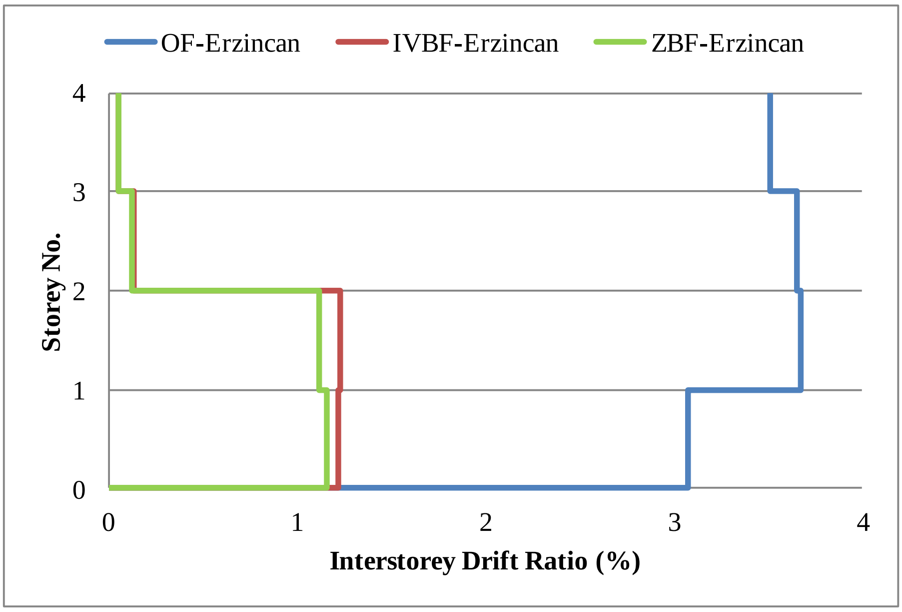
The interstory drift demands over height in the inverted-V and zipper braced frames were evaluated as seen in Figures 4.10 and 4.12. In general, it could be observed that the addition of braced systems reduced significantly the drifts in the frames. And these figures showed that the use of the zipper braced frame was better than the inverted-V braced frame and in the case of frames with inverted-V and zipper, the storey drift demands were significantly lower.



(a)

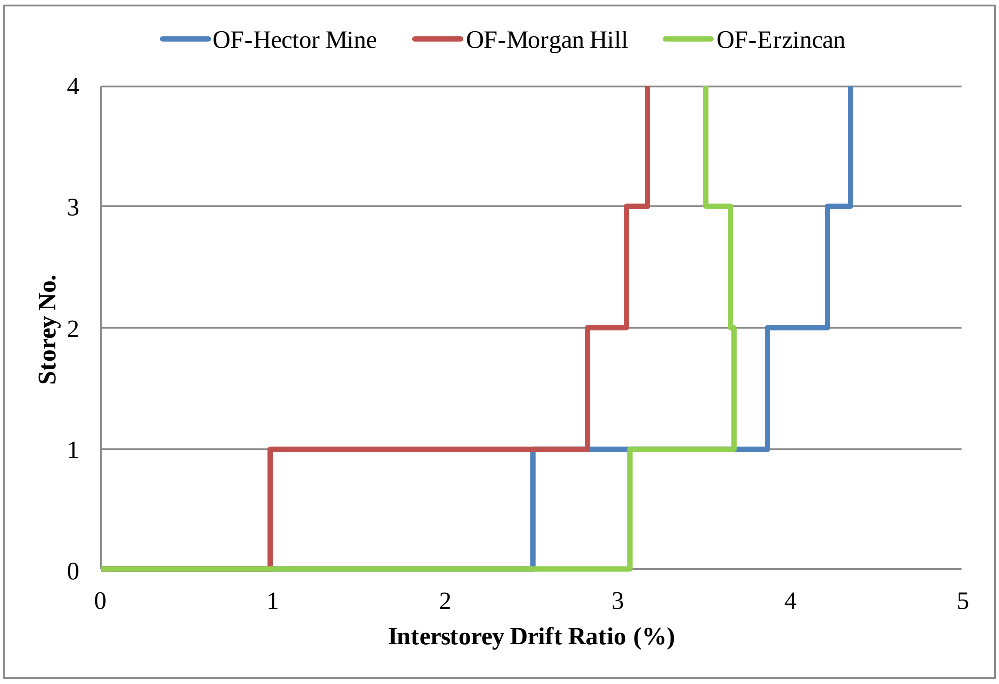


(b)

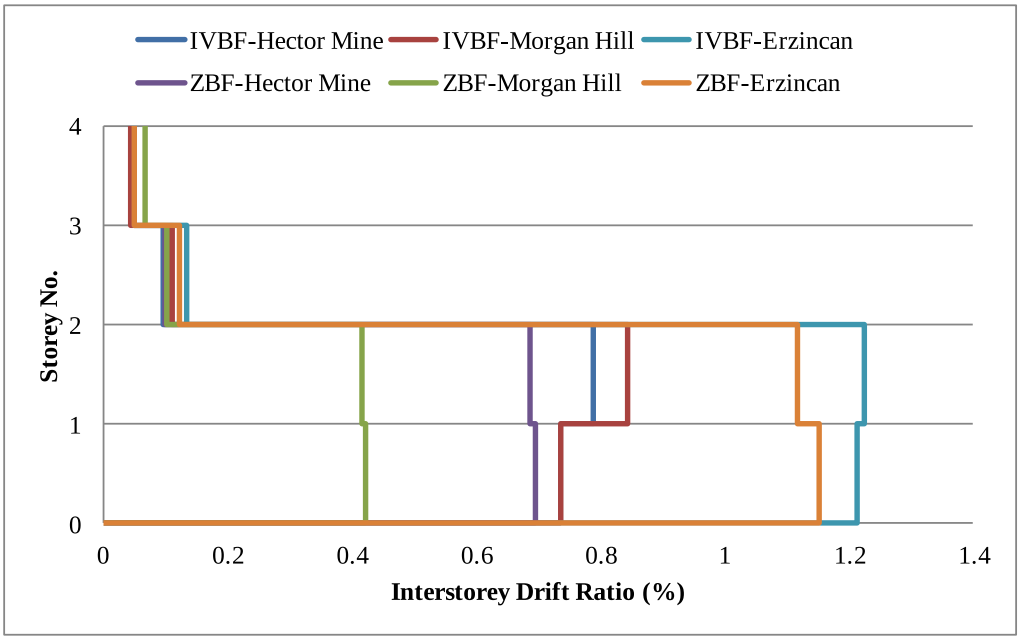


(c)

Figure 4.9 Maximum interstorey drift ratio for the original, inverted-V, and zipper braced frame under the given earthquakes for the four-storey structures

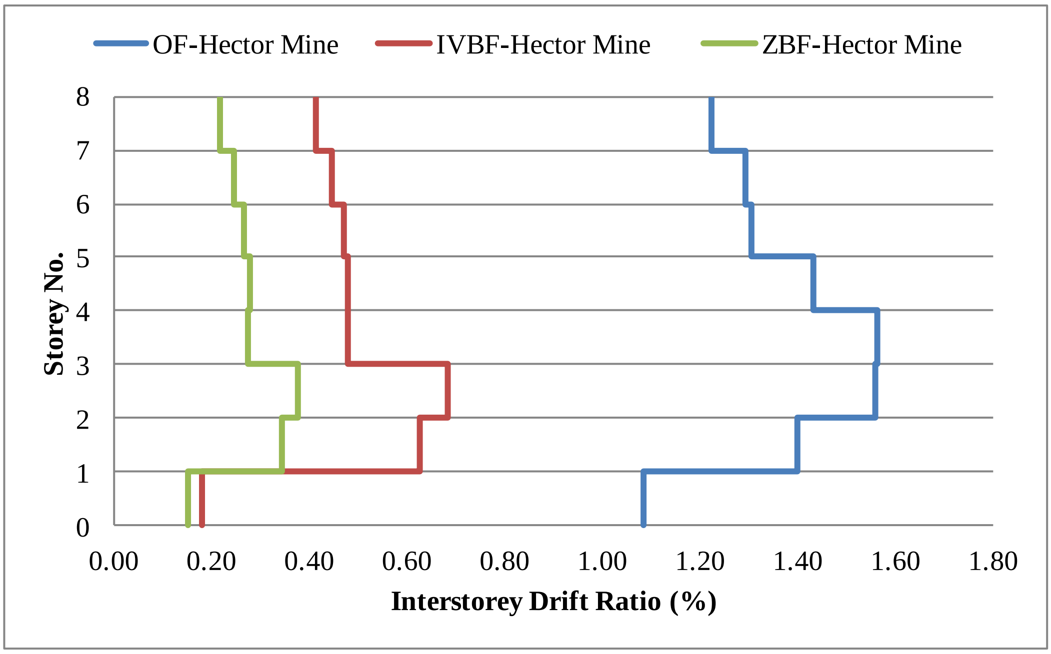


(a)

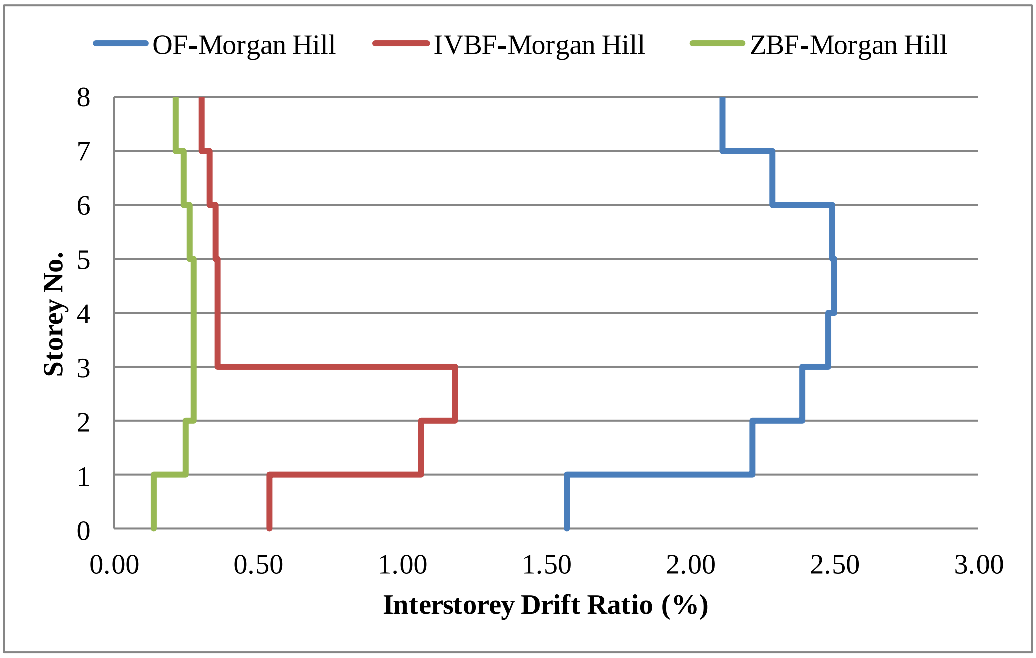


(b)

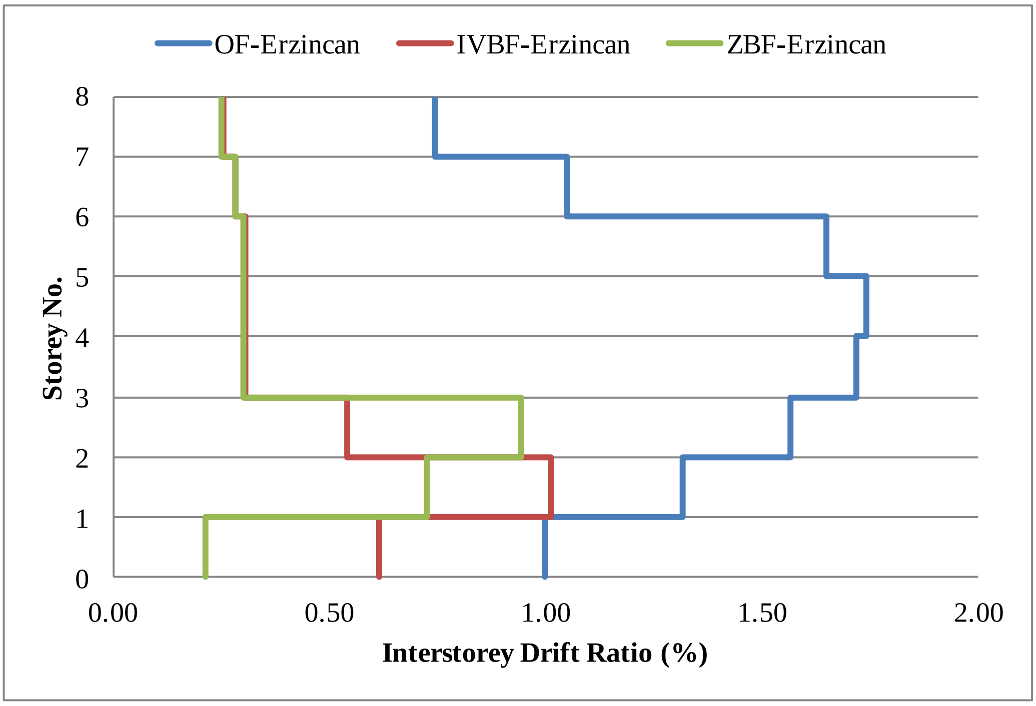
Figure 4.10 Effect of the earthquake accelerations on the 4 storey original and retrofitted RC structures



(a)

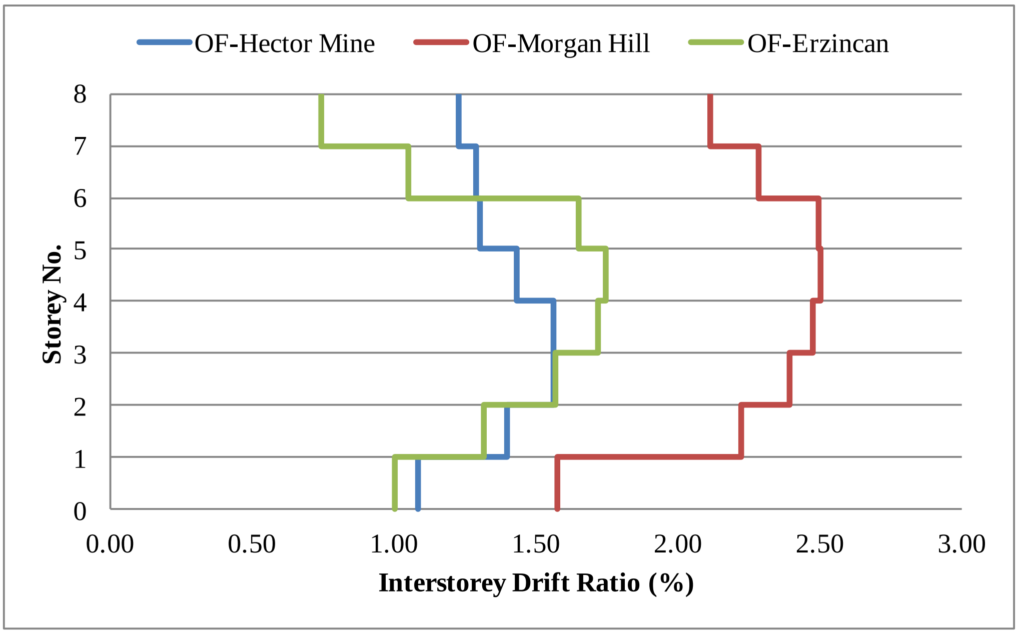


(b)

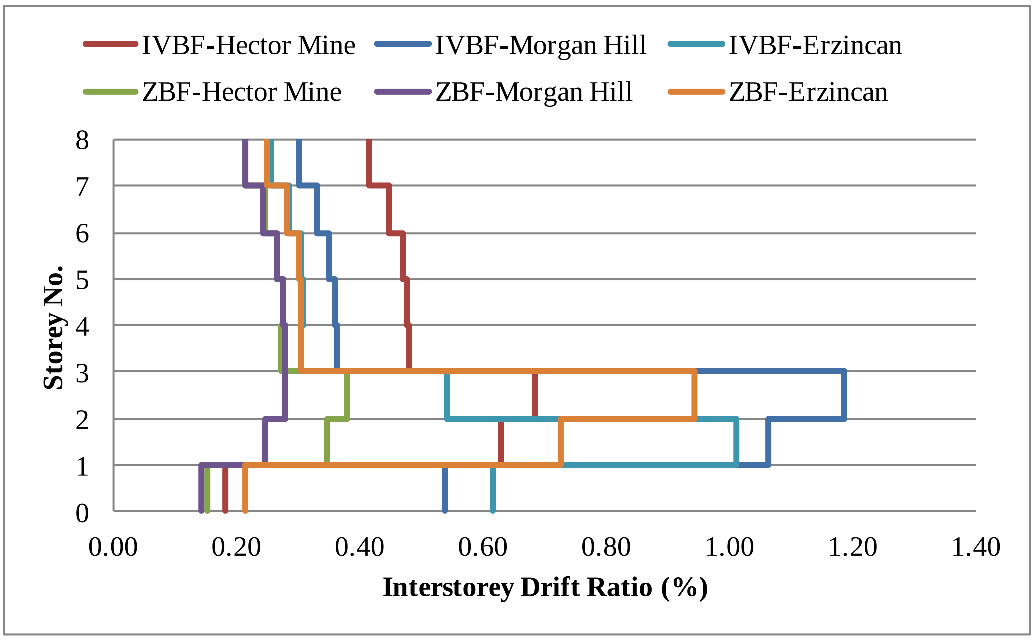


(c)

Figure 4.11 Maximum interstorey drift ratio for the original, inverted-V, and zipper braced frame under the given earthquakes for the eight-storey structures



(a)



(b)

Figure 4.12 Effect of the earthquake accelerations on the 8 storey original and retrofitted RC structures

CHAPTER 5

CONCLUSIONS

This study investigated the structural performance of the existing buildings and those retrofitted with inverted-V and zipper braces. The performance properties were assessed based on nonlinear static and time history analyses. From the results of this study, the following conclusions can be drawn:

- From the capacity curves, it was observed that the base shear, which is the capacity of the structure to resist lateral loads, was considerably increased in the case of the retrofitted frames. The frame with inverted-V and zipper braces had considerably greater load carrying capacity in comparison to the original frame.

- The capacity curve for the four storey building indicated that the retrofitting cases with inverted-V and zipper brace systems had about 3.06 and 3.43 times higher load carrying capacity than the original frame, respectively. In the case of 8 storey building, these values were about 3.73 and 4.26. When comparing chevron and zipper braced frame, zipper braced frame had higher load carrying capacity than inverted-V braced frame.

- However, for the frames with inverted-V and zipper braces, the load carrying capacity from the peak base shear value decreased because of the diagonal element buckling, especially for the four storey buildings. Moreover, the initial stiffness of the retrofitted structures with the addition of the steel braces was observed to increase about 3-4 times compared to the initial stiffness of the existing structure.

- Based on the capacity curves, the behavior of the inverted-V and zipper braced frames was similar in the elastic region, however in the inelastic region it was varied. The zipper braced frame was appeared to be capable of more lateral load carrying and energy absorption capacities than the inverted-

V braced frame due to the contribution of zipper elements distributed load more balanced between the braced members.

- It was also observed that there was a considerable difference between the interstorey index of the existing and retrofitted frames. The interstorey index for the retrofitted frames with inverted-V and zipper systems were lower than that for the existing frames. Moreover, it was worthy noting that the retrofitted structure with zipper system were performing better than the retrofitted structure with inverted-V system.

- Analysis of the results indicated that the use of the inverted-V and zipper braces as a retrofit strategy decreased remarkably the value of maximum storey displacement in the case study. The maximum storey displacement was also affected by the number of storey and frame type. The maximum storey displacement of the frame with zipper system was smaller than other frames, by increasing the number of storeys, the storey displacements had a tendency to increase.

- The retrofitted frames had lower roof displacement compared to the original frames, especially in the case of frames with zipper system. Moreover, the variation of storey displacement of the retrofitted cases along the height of the structure was observed to be more uniform than the original one under all earthquake ground motions.

- The comparison of the interstorey drift ratio for the existing and retrofitted buildings also indicated that the later had significantly lower drift values than the former, depending mainly on the retrofit method, storey height, and earthquake acceleration.

REFERENCES

Badoux, M. and Jirsa, J.O. (1990). Steel Bracing of RC Frames for Seismic Retrofitting, Journal of Structural Engineering, ASCE, Vol. 116, No.1, Paper No. 24219.

Brodericka, B.M., Elghazoulib, A.Y., Gogginsc, J. (2008). Earthquake testing and response analysis of concentrically-braced sub-frames, Journal of Constructional Steel Research, Vol. 64, pp. 997–1007.

Bruneau, M. (2005). Seismic Retrofit of Steel Structure, 1st Canadian Conference on Effective Design of Structures McMaster University Hamilton, Ontario, Canada July 10 – 13.

Bruneau, M., Engelhardt, M., Filiatrault, A., Goel, S. C., Itani, A., Hajjar, J., Leon, R., Ricles, J., Stojadinovic, B. and Uang, C.M. (2005). Review of selected recent research on US seismic design and retrofit strategies for steel structures. Progress in Structural Engineering and Materials, Vol. 7, pp. 103–114.

Chen, Z. (2012). Seismic response of high-rise zipper braced frame structures with outrigger trusses, Master Dissertation, Concordia University Montreal, Quebec, Canada.

Federal Emergency Management Agency, FEMA-356. (2000). Prestandard and commentary for seismic rehabilitation of buildings. Washington (DC).

FEMA. (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings. Federal Emergency Management Agency, Washington, DC.

Ghobarah, A. and Elfath, H.A. (2001). Rehabilitation of a Reinforced Concrete Frame Using Eccentric Steel Bracing, Engineering Structures, Vol. 23, pp. 745-755.

Güneyisi, E. M. and Gültekin, A. (2012). Behaviour of RC framed building with different types of knee brace systems, 10th International Congress on Advances in Civil Engineering, Middle East Technical University, pp. 12, Ankara.

Hajirasouliha, I., Doostan, A. (2010). A simplified model for seismic response prediction of concentrically braced frames, Advances in Engineering Software, Vol. 41 pp. 497–505.

Ikeda, K. and Mahin, S.A. (1986). Cyclic Response of Steel Braces, ASCE, ISSN 0733-9445/86/0002-0342, Paper No. 20396, February, 1986.

Kalkan, O., Güneyisi, E. M. and Gültekin A. (2013). Y-tipi Çelik Çaprazlarla Güçlendirilmiş Betonarme Yapının Davranışının İncelenmesi, 6. Mühendislik ve Teknoloji Sempozyumu, Çankaya Üniversitesi, pp. 9, Ankara.

Khatib, I., Mahin, S., and Pister, K. (1988). Seismic behavior of concentrically braced steel frames. Report No: UCB/EERC-88/01, Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, CA.

Kim, J., Cho, C., Lee, K. and Lee, C. (2008). Design of Zipper Column in Inverted V Braced Steel Frames, The 14th World Conference on Earthquake Engineering, Beijing, China.

Leon, R. T., and Yang, C. S. (2003). Special inverted-V-braced frames with suspended zipper struts, International Workshop on Steel and Concrete Composite Construction, IWSCCC, National Center for Research on Earthquake, Taipei, Taiwan.

Lumpkin, E.J., Hsiao, P.C., Roeder, C.W., Lehman, D.E., Tsai, C.Y., Wu, A.C., Wei, C.Y., Tsai, K.C. (2012). Investigation of the seismic response of three-story special concentrically braced frames, Journal of Constructional Steel Research, Vol. 77 pp. 131–144.

Maheri, M. R. and Ghaffarzadeh, H. (2008). Connection overstrength in steel-braced RC frames. Engineering Structures, Vol. 30, 1938–1948.

Maheri, M. R. and Sahebi, A. (1997). Use of Steel Bracing in Reinforced Concrete Frames. Engineering Structures, pp. 1018-1024.

Mahmoudi, M., Zaree, M. (2010). Evaluating response modification factors of concentrically braced steel frames, Journal of Constructional Steel Research, Vol. 66 pp. 1196-1204.

Nouri, G.R, Kalesar, H.I., Ameli, Z. (2009). The Applicability of the Zipper Strut to Seismic Rehabilitation of Steel Structures, World Academy of Science, Engineering and Technology.

Özçelik, A.Y. and Sarıtaş, A. (2010). Comparison of Chevron and Suspended Zipper Braced Steel Frames, 9th International Congress on Advances in Civil Engineering, Karadeniz Technical University, Trabzon, Turkey.

Özel, A.E. and Güneyisi, E. M. (2011). Effects of eccentric steel bracing systems on seismic fragility curves of mid-rise RC buildings: A case study. Structural Safety, Vol. 33, pp. 82–95.

PEER, “Open System for Earthquake Engineering Simulation,” ttp://opensees.berkeley.edu/OpenSees/OpenSees.html

Pincheira, J.A. and Jirsa, J. O. (1995). Seismic Response of RC Frames Retrofitted with Steel Braces or Walls, Journal of Structural Engineering, Vol. 121, No.8.

Sabelli, R., Roeder, C.W., Hajjar, J.F. (2013). Seismic Design of Steel Special Concentrically Braced Frame Systems, A guide for practicing engineers, NEHRP Seismic Design Technical Brief No. 8, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 13-917-24.

SAP 2000 Advanced 14.0.0., Structural Analysis Program, Computers and Structures Inc. Berkeley, CA.

Stavridis, A. and Shing, P. B. (2010). Hybrid testing and modeling of a suspended zipper steel frame, Earthquake Engineering Structural Dynamics, Vol. 39, pp. 187–209.

Symth, A., Altay, G., Deodatis, G., Erdik, M., Franco, G., Gülkan, P., Kunreuther, H., Luş, H., Mete, E., Seeber, N. and Yüzügüllü, Ö. (2004). Probabilistic Benefit-Cost Analysis for Earthquake Damage Mitigation: Evaluating Measures for Apartment Houses in Turkey. Earthquake Spectra, Vol. 20 pp. 171-203.

Tagawa, Y., Aoki, H., Huang, T., Masuda, H. (1992). Experimental study of new seismic strengthening method for existing RC structure, 10th world conf. on earthquake engineering. pp. 5193–5198.

Tremblay, R. (2001). Inelastic Seismic Response of Steel Bracing", Journal of Constructional Steel Research Vol. 58, pp. 665–701.

TS 500. (2000). Turkish Standards Institute. Building code requirements for reinforced concrete. Ankara, Turkey

Yang, C.S, Leon, RT, DesRoches, R. (2006a). On the development of zipper frames by pushover testing, The fifth behavior of steel structures in seismic areas conference.

Yang, C.S, Leon, RT, DesRoches, R. (2006b). On the development of zipper frames by quasi-static testing and pushover analyses, Proceedings of the 8th U.S. national conference on earthquake engineering.

Yang, C.S. (2006). Analytical and Experimental Study of Concentrically Braced Frames with Zipper Struts. PhD thesis, Georgia Institute of Technology.

Yang, C.S., Leon, R.T. and DesRoches, R. (2008b). Pushover Response of a Braced Frame with Suspended Zipper Struts, Journal of Structural Engineering, Vol. 134, No. 10.

Yang, C.S., Leon, R.T. and DesRoches, R. (2010). Cyclic Behavior of Zipper-Braced Frames Earthquake Spectra, Earthquake Engineering Research Institute, Volume 26, No. 2, pp. 561–582.

Yang, C.S., Leon, R.T., and DesRoches, R. (2008a). Design and behavior of zipper-braced frames. Engineering Structures , 30 (4), 1092-1100.

Yang, T.Y., Stojadinovic, B., and Moehle, J. (2006). Hybrid simulation evaluation of innovative steel braced frame system, Proceedings of the 8th U.S. National Conference on Earthquake Engineering, 18–22 April 2006, San Francisco, California.

Yang, T.Y., Stojadinovic, B., and Moehle, J. (2009). Hybrid simulation of a zipper-braced steel frame under earthquake excitation, Earthquake Engineering and Structural Dynamics, Vol. 38 pp. 95–113.

Zahrai, S.M., Pirdavari, M., Farahani, H.M. (2013). Evaluation of hysteretic behavior of eccentrically braced frames with zipper-strut upgrade, Journal of Constructional Steel Research, Vol.83, pp. 10–20.

APPENDIX

Appendix A: Deflected shapes

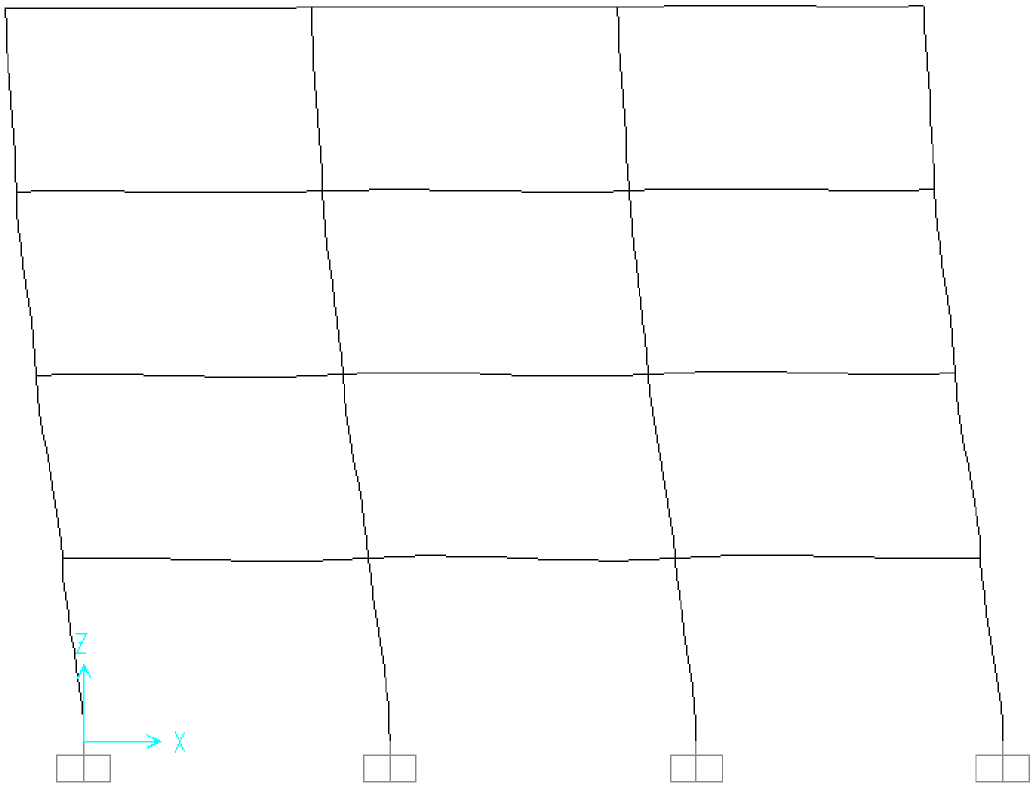


Figure A1 Mode shape of the four storey existing frame at T_1=0.38 s

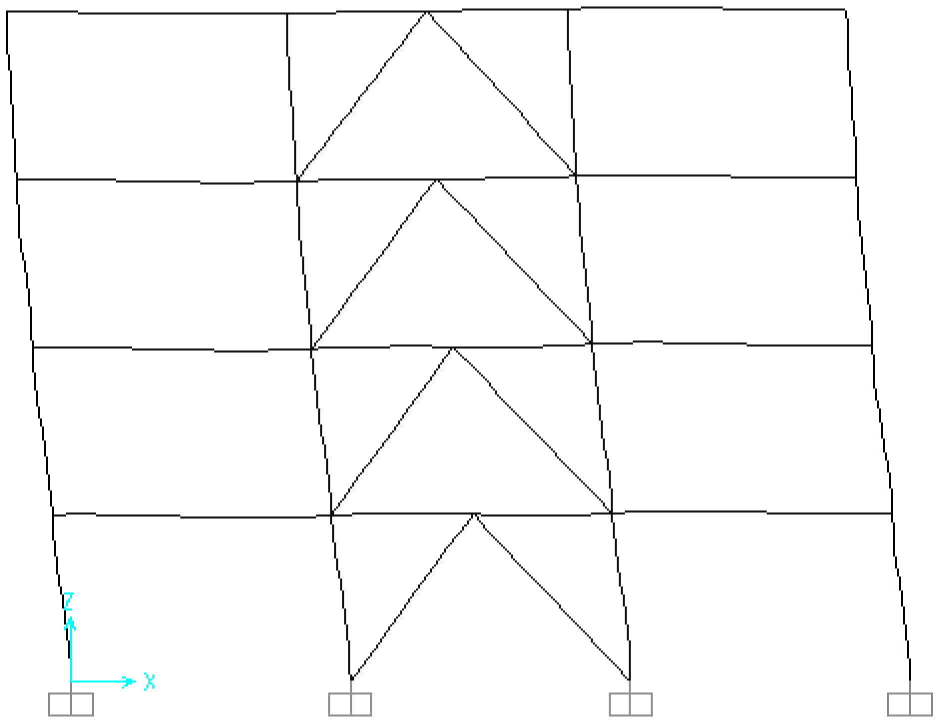


Figure A2 Mode shape of the four storey frame with inverted-V brace at T_1=0.24 s

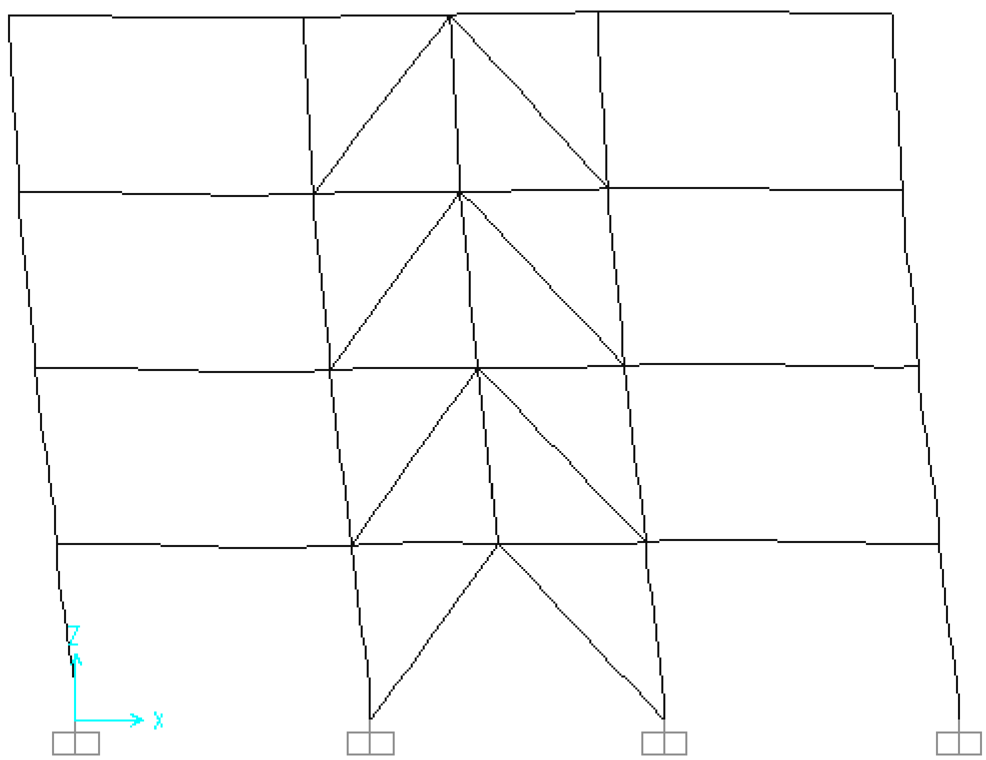


Figure A3 Mode shape of the four storey frame with zipper brace at T_1=0.24 s

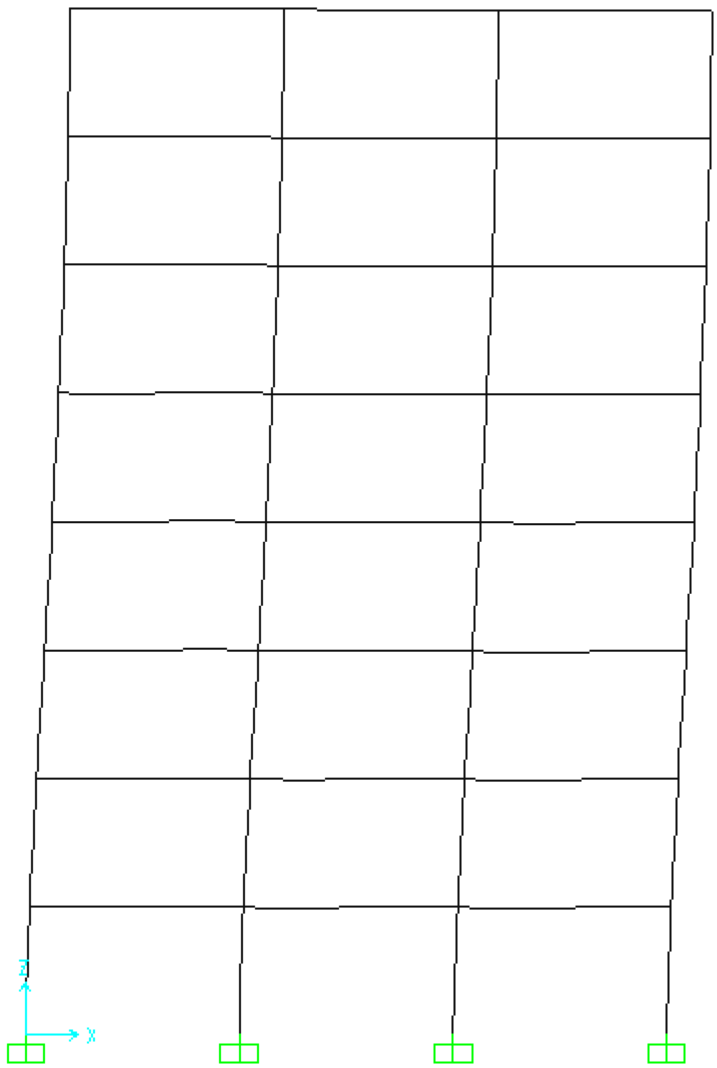


Figure A4 Mode shape of the eight storey existing frame at T_1=0.67 s

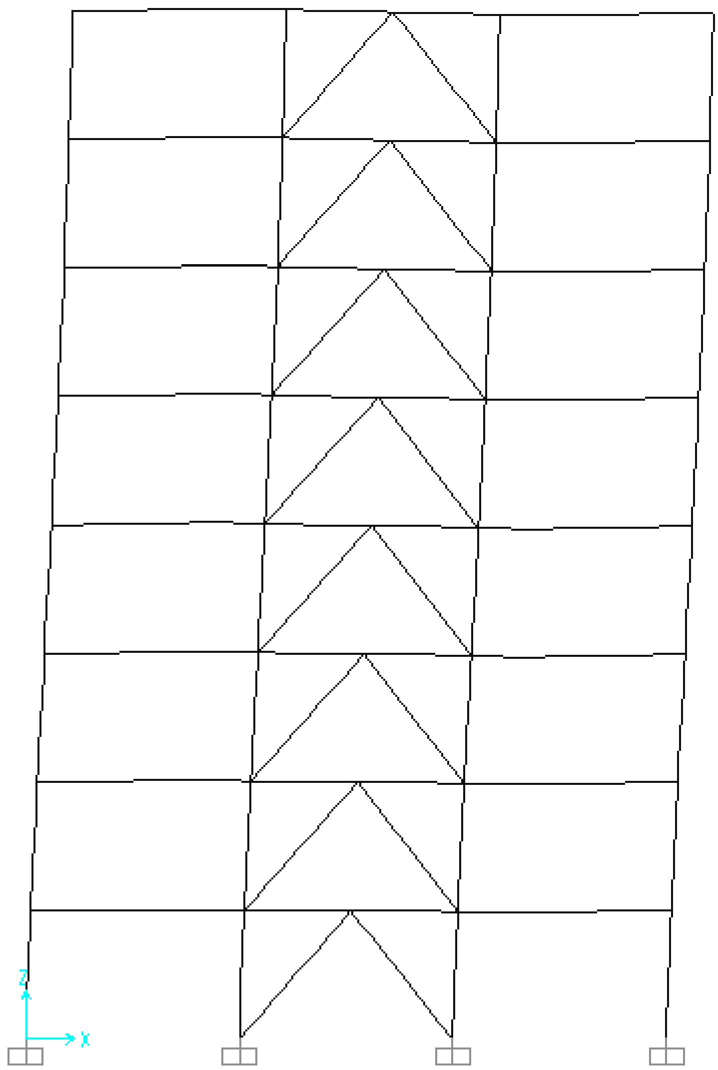


Figure A5 Mode shape of the eight storey frame with inverted-V brace at T_1=0.44 s

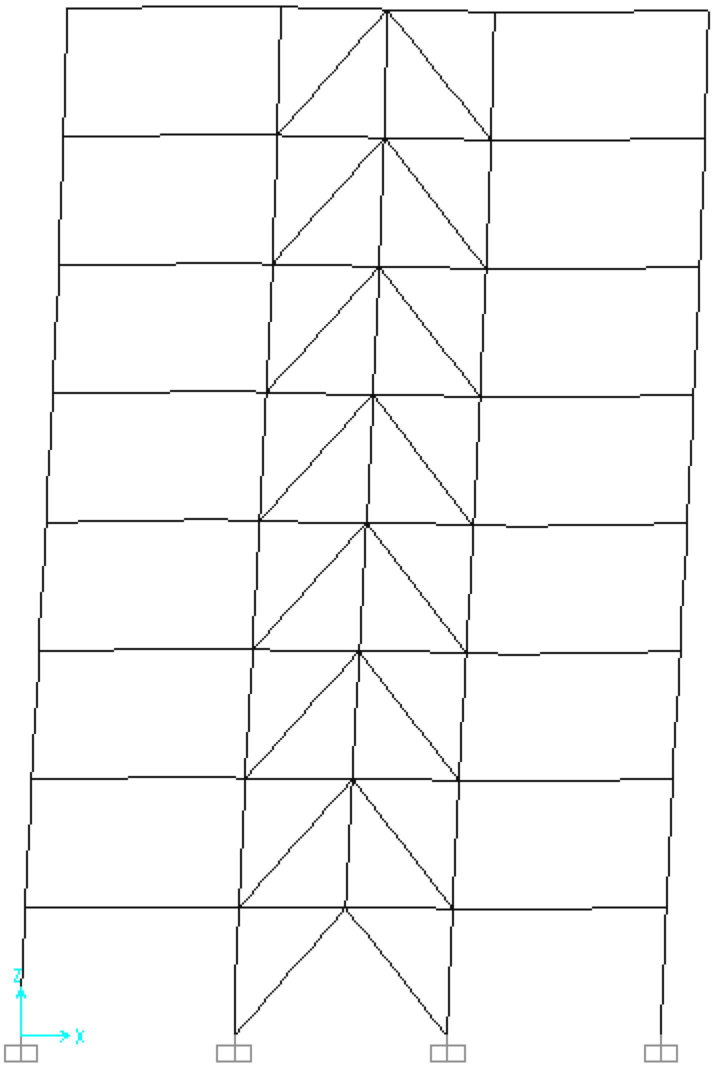


Figure A6 Mode shape of the eight storey frame with zipper brace at T_1=0.44 s

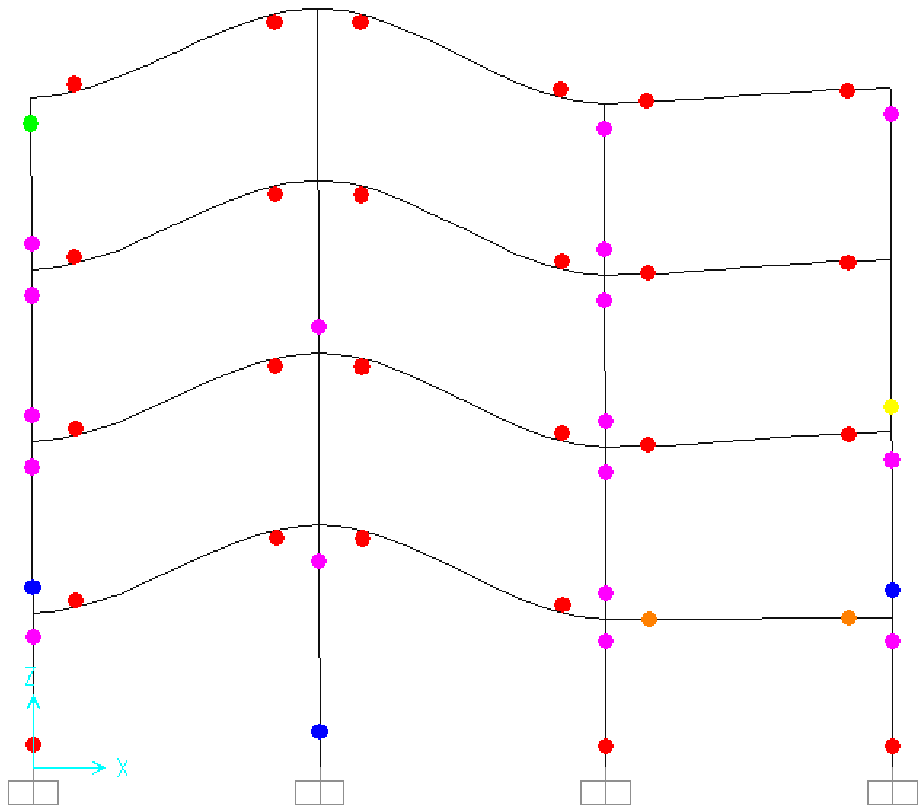


Figure A7 View of hinge formation of the four storey existing frame for the Hector Mine earthquake

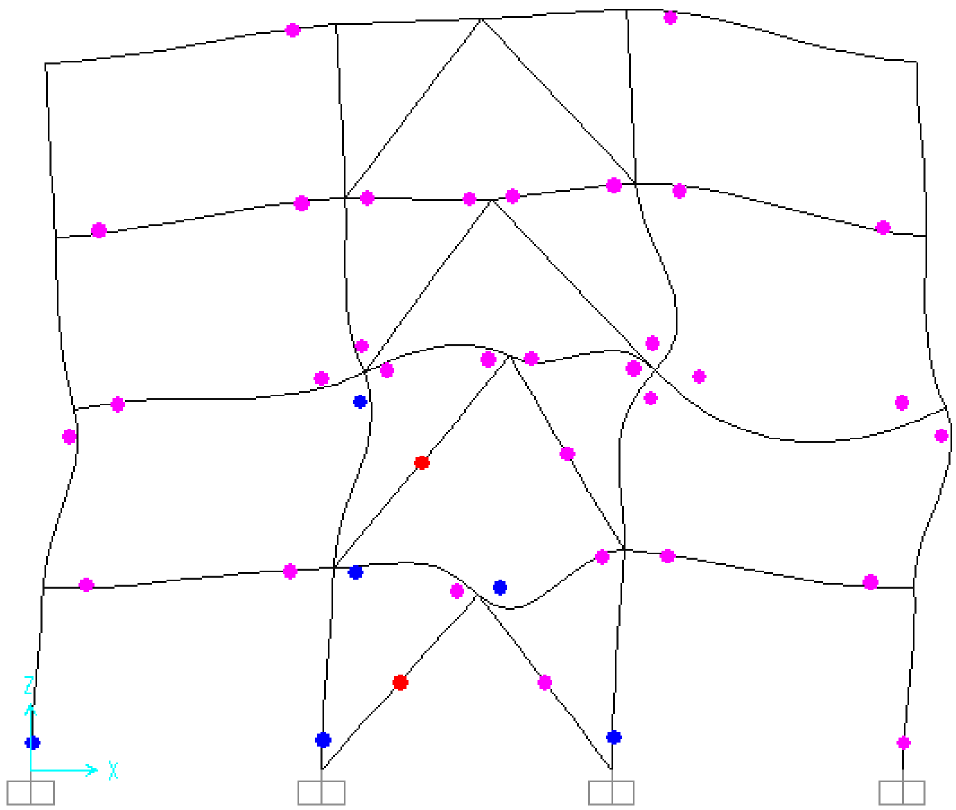


Figure A8 View of hinge formation of the four storey inverted-V braced frame for the Hector Mine earthquake

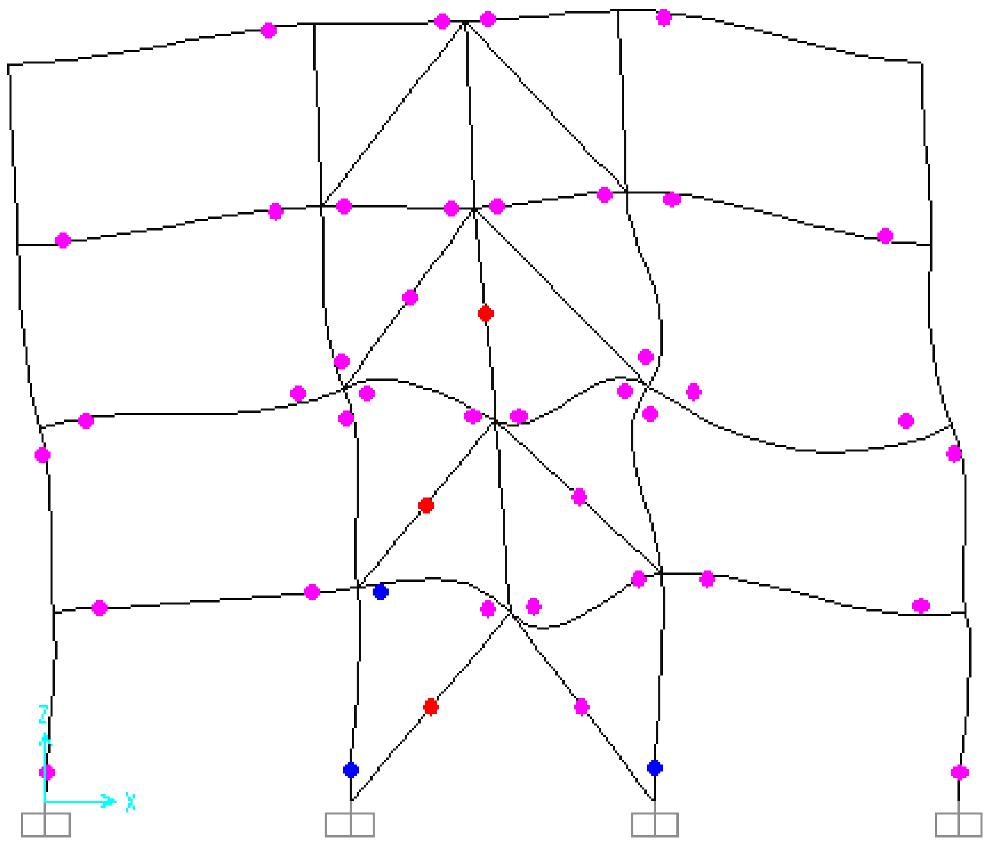


Figure A9 View of hinge formation of the four storey zipper braced frame for the Hector Mine earthquake

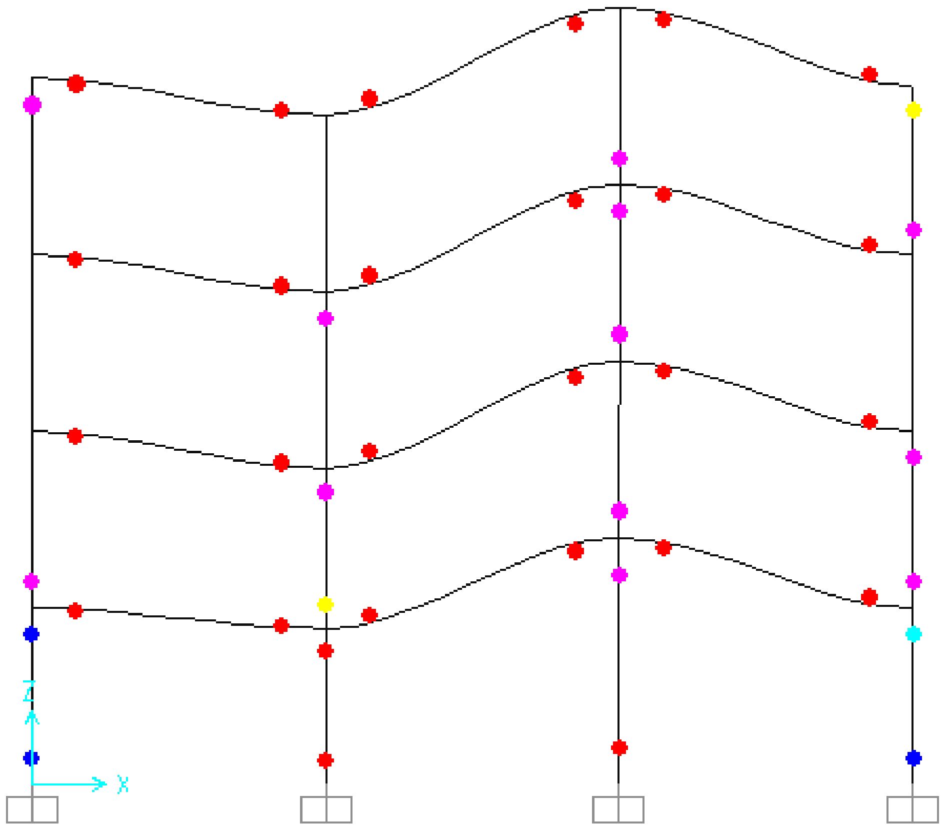


Figure A10 View of hinge formation of the four storey frame for the Morgan Hill earthquake

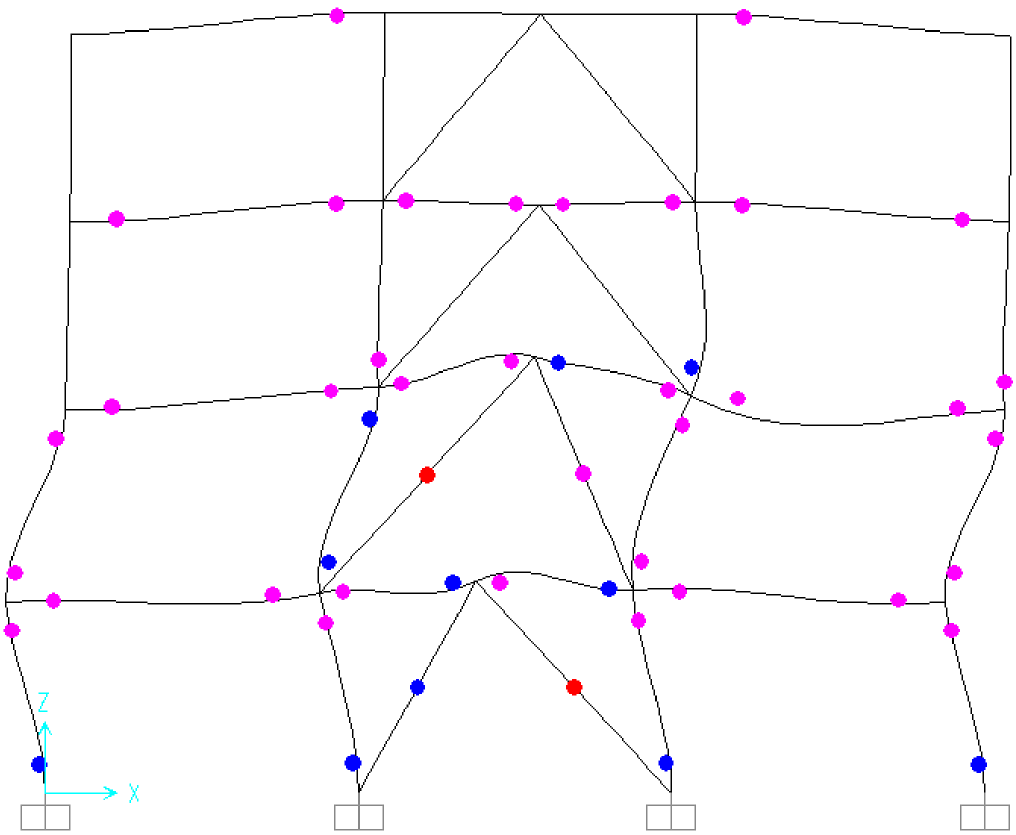


Figure A11 View of hinge formation of the four storey inverted-V braced frame for the Morgan Hill earthquake

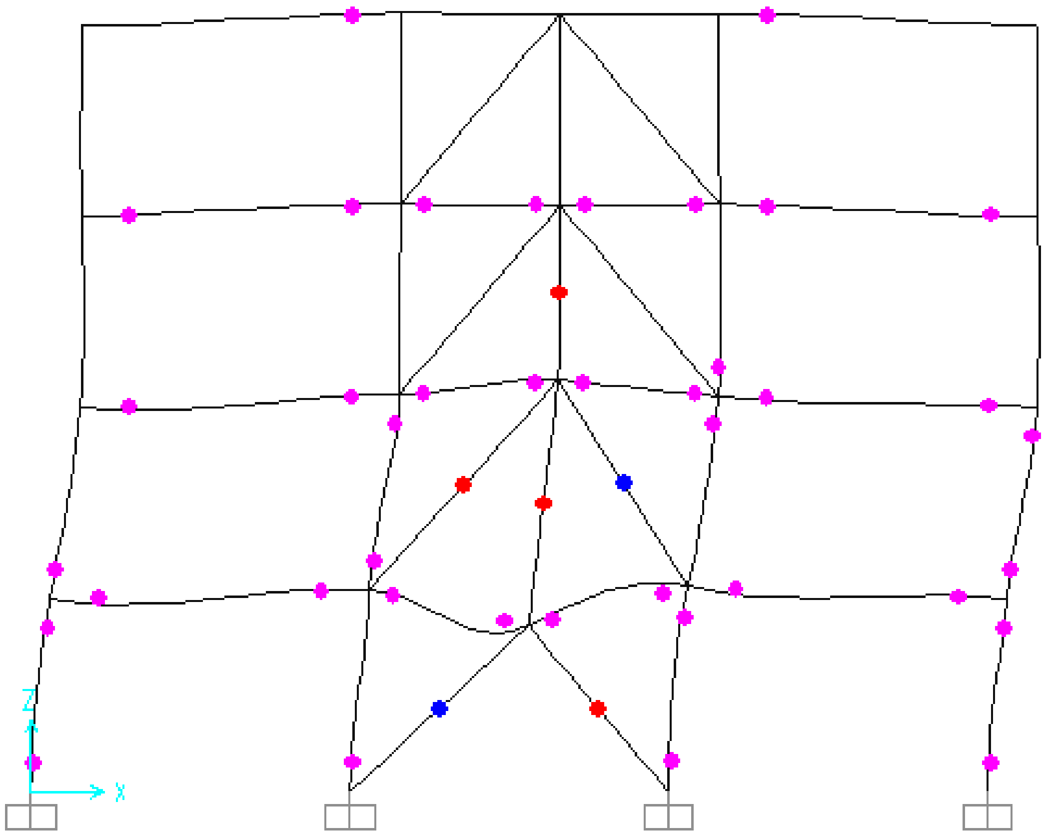


Figure A12 View of hinge formation of the four storey zipper braced frame for the Morgan Hill earthquake

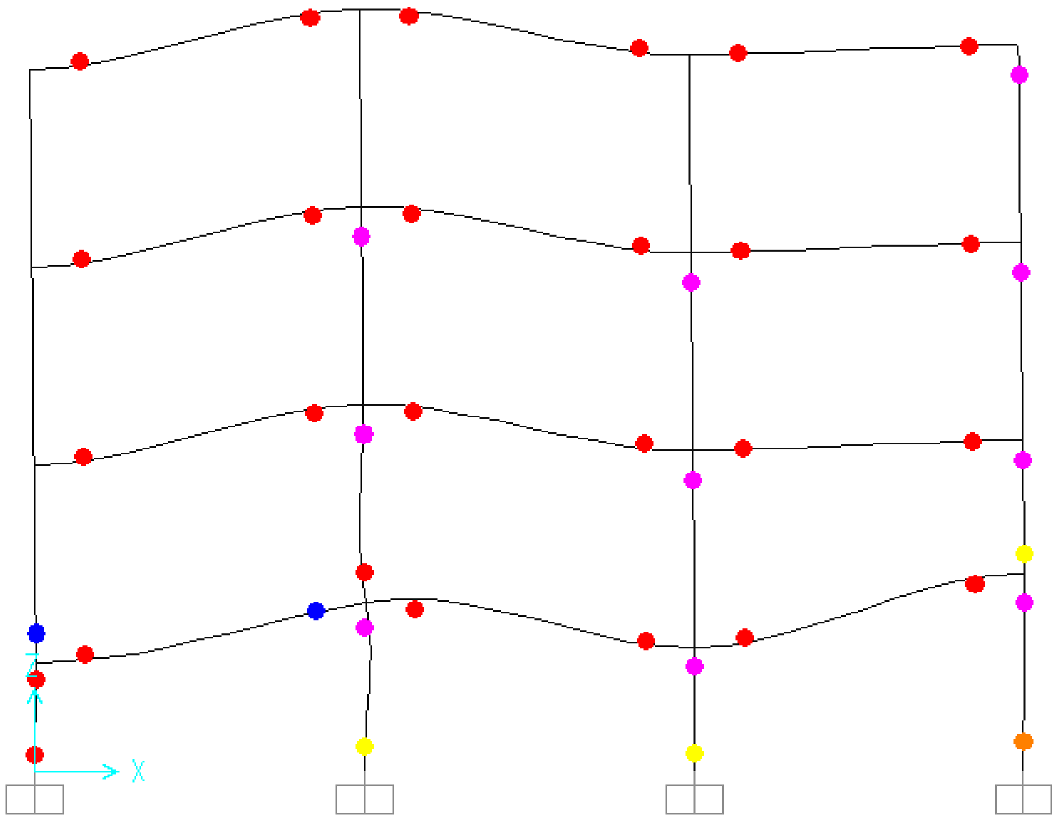


Figure A13 View of hinge formation of the four storey existing frame for the Erzincan earthquake

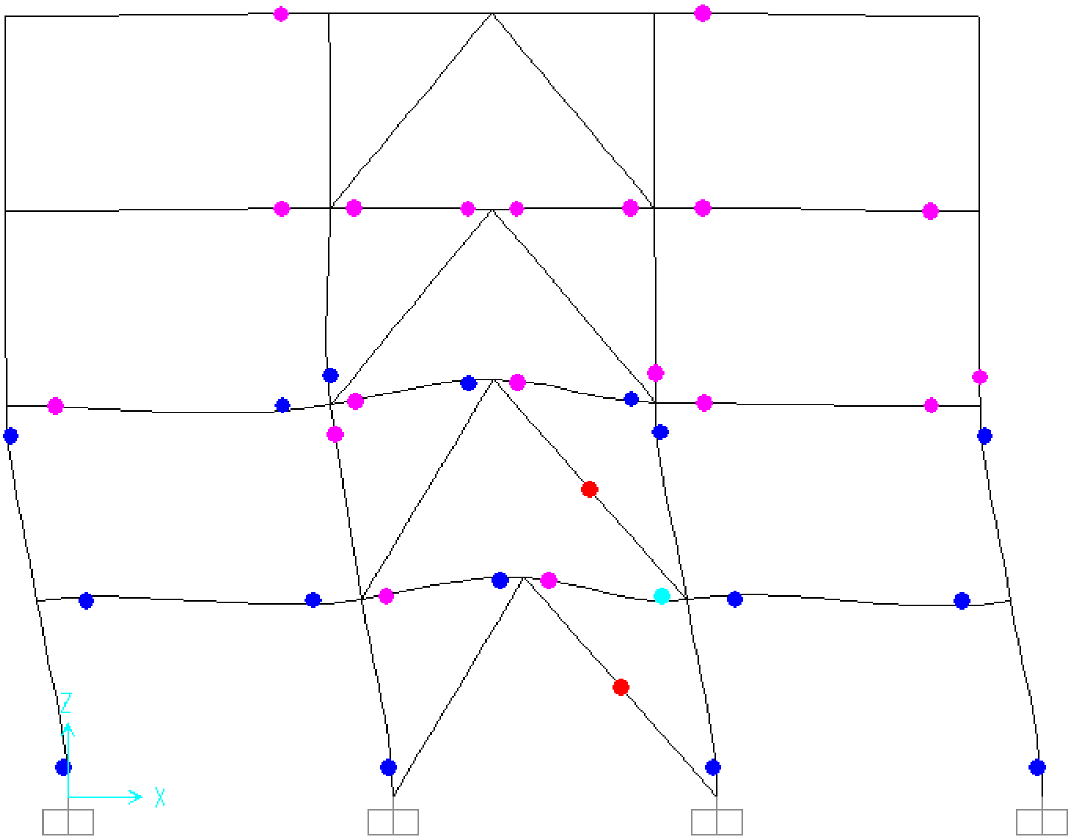


Figure A14 View of hinge formation of the four storey inverted-V braced frame for the Erzincan earthquake

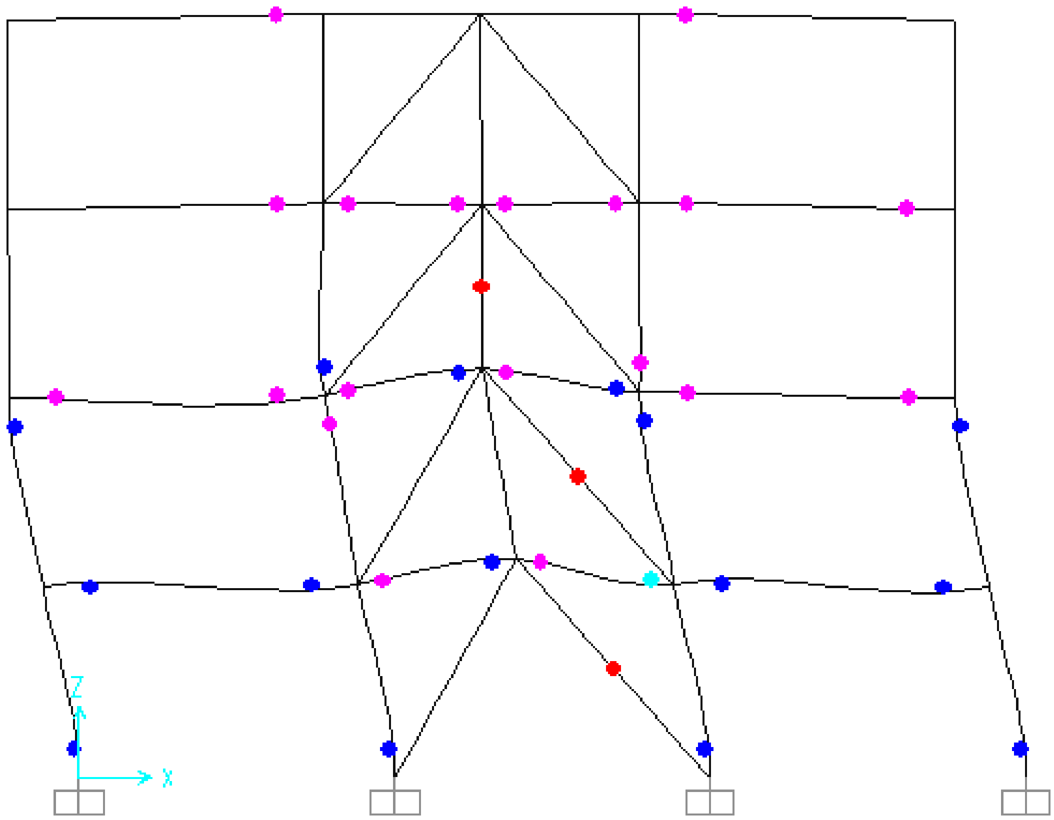


Figure A15 View of hinge formation of the four storey zipper braced frame for the Erzincan earthquake