HASAN KALYONCU UNIVERSITY
GRADUATE SCHOOL OF
NATURAL & APPLIED SCIENCES

SEISMIC PERFORMANCE IMPROVEMENT OF STEEL BUILDINGS BY USING ZIPPER BRACES

M.Sc. THESIS
IN
CIVIL ENGINEERING

By
AHMED SALIH ABDULQADER

JANUARY 2015

Seismic Performance Improvement of Steel Buildings by Using Zipper Braces

M.Sc. Thesis

In

Civil Engineering

Hasan Kalyoncu University

Supervisor

Assoc. Prof. Dr. Esra METE GÜNEYİSİ

By

Ahmed Salih Abdulqader

January 2015

HASAN KALYONCU UNIVERSITY

GRADUATE SCHOOL OF NATURAL & APPLIED SCIENCES

CIVIL ENGINEERING DEPARTMENT

Name of the thesis: Seismic performance improvement of steel buildings by using zipper braces

Name of the Student: Ahmed Salih ABDULQADER

Exam Date: 16. 01. 2015

Approval of the Graduate School of Natural and Applied Sciences.

Prof. Dr. Mehmet KARPUZCU
Director

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

Assist. Prof. Dr. Kasım MERMERDAŞ
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. Erhan GÜNEYİSİ	....
Assist. Prof. Dr. Kasım MERMERDAŞ	

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 these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Ahmed Salih

ABSTRACT

SEISMIC PERFORMANCE IMPROVEMENT OF STEEL BUILDINGS BY USING ZIPPER BRACES

ABDULQADER, Ahmed Salih

M.Sc. in Civil Engineering

Supervisor: Assoc. Prof. Dr. Esra METE GÜNEYİSİ

January 2015, 71 pages

Chevron brace (or inverted V brace) is a common type of the concentric braces. The seismic performance of such system is governed by the buckling of the first story brace in compression. At large, the force redistribution capability of this system has not performed well in severe earthquakes. To solve this problem, the concept of using a zipper brace system was proposed in the literature. In the current study, the effect of using zipper braces on the seismic behavior of 3, 6, and 8 story steel buildings was examined under different ground motions. The case study buildings had the same plan and three bays having equal spans in each direction. For this, two dimensional model for each building was considered in the analysis. To improve the structural response of the existing structures under earthquake loading, single and double zipper braces were used. The nonlinear static and dynamic analysis was performed on the structures with and without zipper braces. In the dynamic analysis, 1994 Northridge, 1999 Hector Mine, and 1999 Chi-Chi earthquake records were employed. It was observed that the use of zipper braces over the frame height was found to have a significant effect on the seismic behavior of the structures. Moreover, the results of analysis showed that the steel structures with zipper braces had lower displacement and drift ratio than the existing ones.

Keywords: Dynamic analysis, Seismic behavior, Static analysis, Structural performance, Zipper braced frame

ÖZET

FERMUAR ÇAPRAZ KULLANILARAK ÇELİK BİNALARIN DEPREM PERFORMANSININ İYİLEŞTİRİLMESİ

ABDULQADER, Ahmed Salih
İnşaat Mühendisliği, Yüksek Lisans Tezi
Danışman: Doç. Dr. Esra METE GÜNEYİSİ
Ocak 2015, 71 sayfa

Ters V çaprazlar merkezi çapraz sistemlerin bir tipini oluşturmaktadır. Bu tür sistemlerin deprem performansı çoğunlukla birinci kat çapraz elemanların basınçta burkulmasına bağlıdır. Genellikle, şiddetli depremler altında bu sistemlerde yük tekrar dağılımı yeterli şekilde gerçekleşmemektedir. Literatürde bu problemin çözümü için fermuar tipi çapraz kullanımı önerilmektedir. Sunulan bu çalışmada, fermuar çaprazların değişik deprem kuvvetlerine maruz 3, 6 ve 8 katlı çelik binaların deprem davranışına etkisi incelenmiştir. Örnek binalar her yönde üç eşit açıklığa ve aynı kat planına sahiptir. Bu nedenle analizlerde iki boyutlu modeller kullanılmıştır. Mevcut yapıların yapısal davranışlarının iyileştirilmesinde tek ve çift fermuar çapraz uygulaması yapılmıştır. Yapıların analizinde doğrusal olmayan statik ve dinamik analizler gerçekleştirilmiştir. Dinamik analizlerde 1994 Northridge, 1999 Hector Mine ve 1999 Chi-Chi deprem kayıtları kullanılmıştır. Yapı yüksekliği boyunca fermuar çaprazların kullanımının yapıların deprem davranışı üzerinde önemli etkisinin olduğu, ayrıca, çapraz kullanımı ile mevcut yapıların göreli kat ötelenmesi ve yerdeğiştirme taleplerinde azalmalar olduğu görülmüştür.

Anahtar Kelimeler: Dinamik analiz, Deprem davranışı, Statik analiz, Yapısal performans, Fermuar çaprazlı çerçeve.

To my dear parents, my Father, my Mother, my wife, brother, sisters, and all friends to give me full support, encouragement over the time

ACKNOWLEDGEMENTS

In the name of Allah, the Entirely Merciful, the especially merciful. First of all, I want to express my gratitude and thankfulness to the God almighty who is creator, the sovereign, and the sustainer of the universe and creatures. It is only through his mercy and help this work could be completed and I am hoping that this little effort be accepted by him.

I would like to express my deep gratitude to my Supervisor: Assoc. Prof. Dr. **ESRA METE GÜNEYİSİ** for suggesting the research project, and for her continuous guidance and encouragement during my work, wither them it would have been impossible for this study to be completed.

Special thanks to Hasan Kalyoncu University and the Iraq Ministry of Higher Education and Scientific Research for accepting me in the M.Sc. Follow ship program.

Also, I would like to express my special thanks to Res. Asst. **Ayşegül Gültekin**, who help me a lot during M.Sc. program, especially during design works.

Finally, I would like to express my sincere gratitude to anyone who helped me throughout the preparation of the thesis.

TABLE OF CONTENTS

LIST OF FIGURES

Page No.

LIST OF TABLES

Page No.

LIST OF SYMBOLS / ABBREVIATIONS

AISC	American institute of steel construction
CBF	Concentrically braced frame
EBF	Eccentrically braced frame
FEMA	Federal emergency management agency
L.L	Live load
MRF	Moment Resisting Frames
NDP	Nonlinear dynamic procedure
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
SCBF	Special concentric braced frame
ZBF	Zipper braced frame

CHAPTER 1

INTRODUCTION

1.1 Background and motivation

The zipper braced frame configuration (Figure 1.1) was first proposed by Khatib et al. (1988). The frame has geometry similar to that of the conventional inverted-V braced frame, except that a vertical structural element, the zipper column, is added at the beam mid-span points from the second to the top story of the frame. Like other concentrically braced frames, the zipper braced frame is very effective in providing stiffness to limit the story drifts under lateral loading (Khatib et al., 1988).

However, in seismically active zones, for economical reasons, the design philosophy allows the structure to absorb and dissipate earthquake energy through yielding of the structure. Since the braces have the largest stiffness in the building, they attract the largest lateral load and may buckle under severe lateral loading. Conventional steel braces have lower compression capacity then tension capacity. Thus, when the compression brace in the inverted-V sub-assembly buckles, the tension force in the other brace creates a large unbalanced vertical force at the mid span of the beam. This creates a design challenge (Khatib et al., 1988).

The zipper braced frame is designed to resist the unbalanced vertical load using the zipper columns. In the event of severe earthquake shaking, the compression brace in the first story of a zipper braced frame may buckle this action would increase the compression force in the second-story brace and consequently may cause it to buckle. Under increasing lateral deformations, the unbalanced vertical force would be transmitted upward through the structure and may lead to a mechanism in which all compression braces are buckled and beam plastic hinges are activated (Khatib et al., 1988).

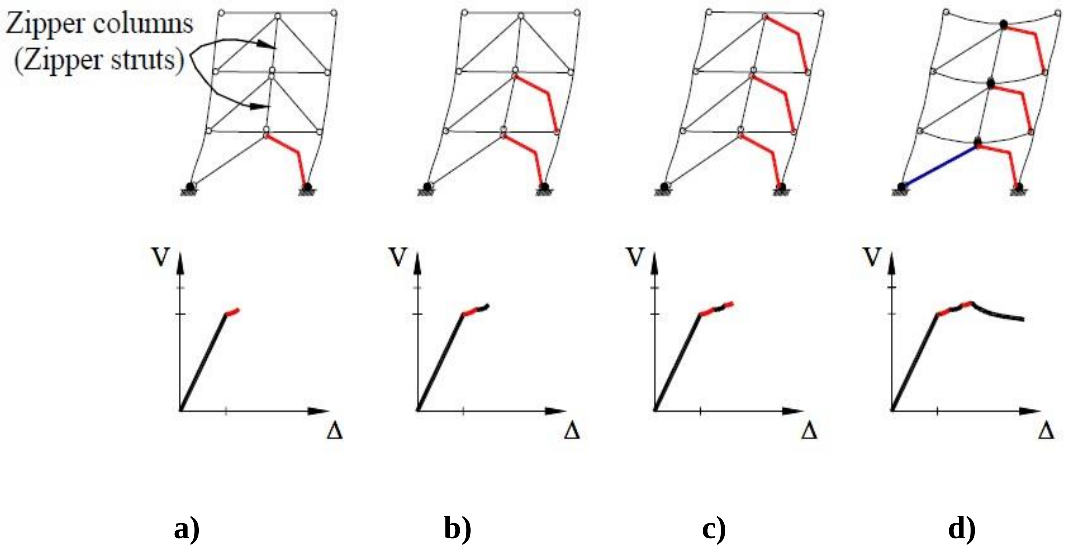


Figure 1.1 Full-height zipper mechanisms (Nouri et al., 2009)

This will result in a highly desirable distribution of inelastic response along the height of the frame. However, loss in structural stiffness may occur once the full zipper mechanism forms and the frame enters a softening response range. The reduced lateral load capacity and softening in the force-deformation response of the zipper braced frame after a full plastic mechanism has formed limits the applicability of the zipper braced frame configuration (Leon and Yang, 2003). They therefore modified the conventional zipper braced frame by increasing the member sizes of the braces at selected stories along the frame height such that they remain elastic and prevent the formation of the complete zipper mechanism. This configuration is named as the zipper braced frame (Figure 1.1). The primary function of the zipper column is to transfer the unbalanced vertical force to the upper story braces and support the beams at mid span.

Leon and Yang (2003) studied this configuration in which it provides a clear force path and makes the capacity design for the frame relatively explicit. And from the results of nonlinear static analysis, it seems that the pendulous zipper braced frame has a slightly larger strength and more ductility than the conventional one. The main disadvantage arises as the number of stories increases, and the magnitudes of the unbalanced vertical forces transmitted up to the top-story braces become very large, making the design of the top story braces very difficult. They have shown that by providing the support at mid span of the beams, a reduction of the beam sizes can be achieved, which may save material and makes the pendulous zipper braced frame more economical.

Many recent studies, such as the research of Tirca and Tremblay (2004), and Yang and Leon (2004, 2008) showed the behavior and seismic performance of the steel frames along with the zipper braced system. These researches used the nonlinear static analysis and nonlinear dynamic analysis. The results showed that the design procedure produced safe zipper braced system.

The pushover analysis, or as it is called nonlinear static procedure (NSP), is the most preferred one, as it has been developed since about twenty years. It is preferred due to the design and seismic performance evaluation purposes. This is because of the procedure which is relatively simple. It also considers the elastic behavior. Pushover analysis has been subjected of the mathematical model to increase the lateral loads till a kind of displacement is reached. It has also shown the structural response characteristics under the pressure of the seismic procedures. The nonlinear load deformation behaviors and individual components of the structure would be calculated in a mathematical model. The target displacement, on the other hand, would represent the maximum displacement that can be experienced during the earthquake design. According to the study, if the ratio of the strength exceeds the maximum value, the significant nonlinear duration as well as nonlinear dynamic analysis would be needed. Another point, the hinge mode effects must not be significant, which is determined by comparing different stories using only the first mode which in turn it will produce 90% of the mass of participation. The internal forces would be approximated of the expectations during the design of the earthquake FEMA 365 (2000).

A computational model that incorporates all the load-deformation characteristics of the individual components is required by a nonlinear dynamic procedure or time history analysis. This model would be subjected to earthquake shaking which are represented by ground motion time histories. This motion must be specific to the building. The results of the internal forces that are predicted by the model shouldn't be modified as the nonlinear response is modeled. In this case, the displacements can be compared to the acceptance criteria directly. Moreover, it has been subjected a mathematical model to earthquake shaking that is represented by ground motion time histories to achieve forces and displacements, which would 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 and individual components of the structure. And, the calculated internal forces would be reasonable approximations of those expectations during the design earthquake FEMA 356 (2000).

1.2 Objectives and research tasks

The principal purpose of this study is to compare the seismic performance of the original steel buildings with single and double zipper braces. As a case study, 3, 6 and 8 story steel buildings were considered. The effect of using 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 analysis. The seismic response of the original frames and those with zipper braces were evaluated using different earthquake ground motions, namely, 1999 Hector Mine, 1994 Northridge, and 1999 Chi-Chi earthquake records. The performance of the steel structures with and without zipper braces is evaluated in term of capacity curve, story drift, displacement time history, etc.

1.3 Layout of the thesis

This thesis comprises of five chapters: -

Chapter 1-Introduction: The objective of the thesis are summarized.
Chapter 2-Literature Review: This chapter explains the background about previous studies on concentrically braced frames, eccentrically braced frames, especially zipper braced frames, and steel bracing systems.
Chapter 3- Case study: This chapter covers the description of the 3, 6, and 8 story structures withand without single and double zipper braces. Moreover, the detail of the analysis methods and the characteristics of the earthquake records used are provided.
Chapter 4-Results and Discussion: This chapter presents and compares the results obtained from nonlinear static and dynamic analysis of each structural system.
Chapter 5-Conclusions: In this chapter, the conclusions are given in the light of findings from the overall results of the analysis.

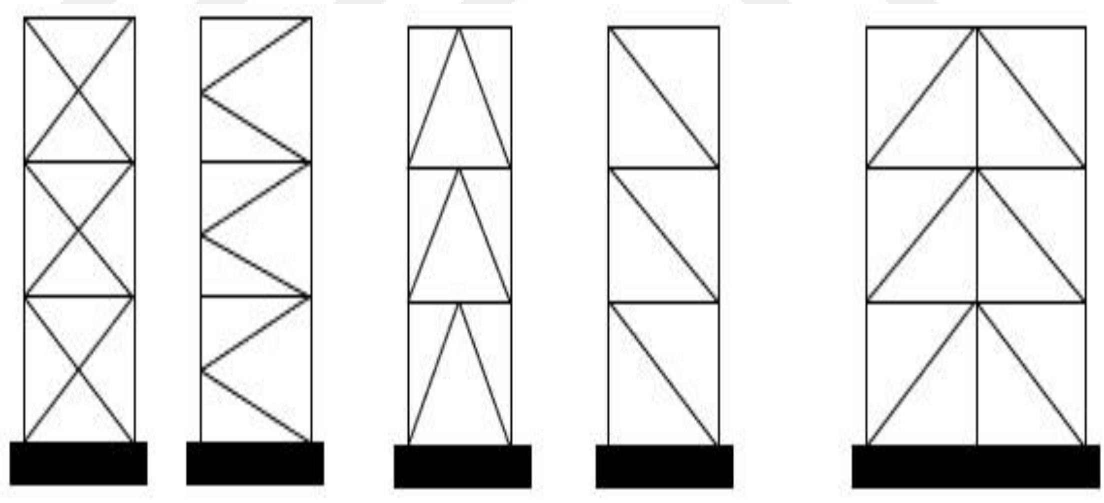
CHAPTER 2

LITERATURE REVIEW

2.1 Description of concentrically braced frames

Concentrically braced frame (CBF) is a cost-effective system for resisting lateral loads. This structural system is usually employed for low- and mid-rise steel framed buildings. This configuration shows a concentration of damage within a single floor and the tendency of storey mechanism formation. For instance, extensive damage was found in CBF buildings during Tohoku earthquake on March 2011 (Lignos et al., 2011), Christ-church earthquake on 2010 (Bruneau et al., 2010), Loma Prieta earthquake (1989), Northridge earthquake (1994), Kobe earthquake in (1995) (Tremblay et al., 1996) and other events. Frequent damage was observed in braced frames where braces were proportioned to resist tension only, where connections were weaker than the braces attached to them, where braces framed directly into columns, and where braces were inclined principally in one direction. Under strong ground motions, braces in compression have buckled, and in consequence lose their buckling resistance strength. After buckling of braces occurred, beams were deflected downward as a result of the combined action of the gravity loading and the unbalanced force developed at the braces to beam intersection point due to the difference between the tensile and post-buckling capacity of brace members. In this case, strong floor beams are required to stabilize the system when the unbalance vertical load transferred from braces to beams has increased due to the attaining of the post-buckling strength in the compressive brace (Chen, 2012).

MacRae and Clifton (2013) studied a significant number of low and medium rise concentrically braced frame buildings. They typically used the welding. Although they were permitted to be designed for high ductility (up to a structural ductility factor of 4), same of the structures were designed for low ductility. The final results showed that in the 2010 and 2011 Canterbury earthquakes, the steel buildings on the whole behaved very well by not only satisfying their “life safety” even though the shaking was significantly greater than the design level. Because of different strengths in tension and compression, and their susceptibility to low cycle fatigue fracture, the formation of large local curvatures were generated by buckling in compression. They were generally not permitted to be major energy dissipating element in tall structures. They also needed to be designed for significantly greater strength than other systems showing more ductility. The bracing might be placed in different configurations such as X, K, inverted V, or diagonal bracing as shown in Figure 2.1. Balanced diagonal bracing was the most common for moderate rise structures because it provided the same strength in both directions. Frames with balanced diagonal bracing sustained buckling to the braces, but with appropriate bolted connections to the frame, the braces could be replaced after a major earthquake.

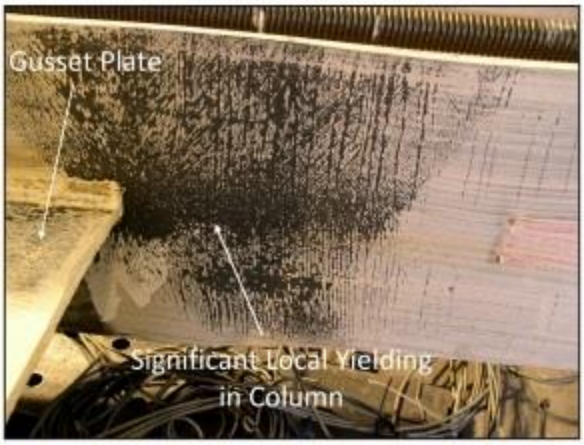


a) b) c) d) e)

Figure 2.1 Different bracing configurations for concentrically braced frames a) X, b) K, c) inverted V, d) diagonal, and (e) balanced diagonal bracing (MacRae and Clifton, 2013)

Sheng et al. (2002) conducted a parametric study on the inelastic compressive behavior and strength of gusset plate connections using the finite element program ABAQUS. Model configuration was isolated with the gusset plate in order to better understand the behavior of the gusset plates and the failure modes. The parameters included in their study were the length of the long, free edge of the gusset plate, gusset plate shapes (tapered and rectangular plates), types of connection between the splice member and the gusset plate (welded and bolted connections), rotational restraint provided by the bracing member along the element x-axis between the bracing member and gusset plate, splice member types and stiffness (tubular-section and tee-section), splice member length, and free edge stiffeners. Free edge length affected the strength of specimens significantly since the local buckling occurred along the free edge. The ultimate load of the specimen was decreased considerably with increased free edge length. Tapered plate and the types of splice member did not influence the ultimate load of specimens. The ultimate loads of specimens were increased by 10-20 % when the splice-to-gusset plate connections were welded. To improve connection behavior, they recommended adding the centerline stiffener as close to the beam and column boundaries as possible and adding free edge stiffeners. In addition, they suggested that the use of a bracing member that provided stronger out-of-plane bending rigidity. By comparing the analytical ultimate loads with design loads for alternative methods of gusset plate design, they found that the Whitmore method was ineffective at estimating the ultimate loads of the specimens, and the Thornton method and modified Thornton method provided conservative estimates of the ultimate load capacities. In Figure 2.2, the response of the braced frame under loading is illustrated (Sabelli et al., 2013).

a) Brace buckling deformation b) Deformation of gusset plate



c) Local yielding in beam and column

Figure 2.2 Various aspects of braced frame behavior (Sabelli et al., 2013)

Tremblay (2002) showed the results of several experimental studies on inelastic response of steel braces in cyclic loading. The study examined the seismic design of concentrically braced steel frames for which at ductile response under earthquakes. And, the parameters of the study included the buckling strength and compressive resistance. In addition, the maximum ductility was considered in the bracing systems. The test programs covered a wide range of brace properties, including the type, compressive and tensile strengths, and material properties. Brace resistance could be determined at various deformation considering actual yield strength of all the specimens and its effects in the design. It was observed that actual yield strength of the steel material exceeded the nominal properties in all specimens. These values could not be used directly in design in view of the long period of time. In particular, the bracing members are often made with small shapes which might exhibit relatively higher yield strength as seen in Figure 2.3. The compressive strength of the braces at first buckling, Cu, generally exceeded the value founded from column design curves. This indicated that both the compression and tension braces at a given floor of a symmetrical bracing bent could develop simultaneously a compression force equal to Cu and a tension force equal to AgFy, respectively. And, the reduced compressive strength equal to 0.8*Cu could be used when yielding developed in the companion tension braces. It was also reported that applied loading history affected the maximum tension force that would develop in a brace section and the 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. Moreover, it was concluded that fracture of bracing members was highly dependent on slenderness ratio and slender braces could sustain higher ductility levels prior to fracture.

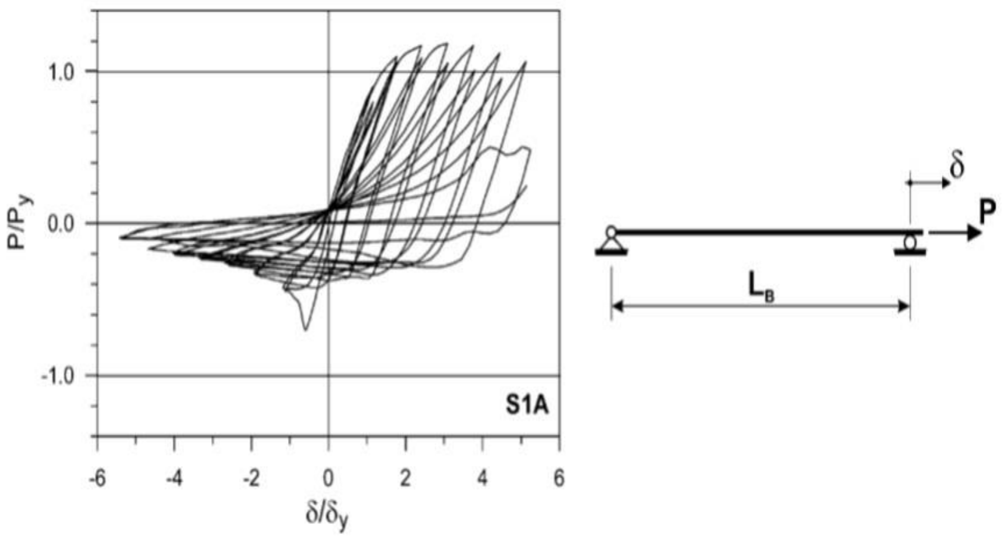
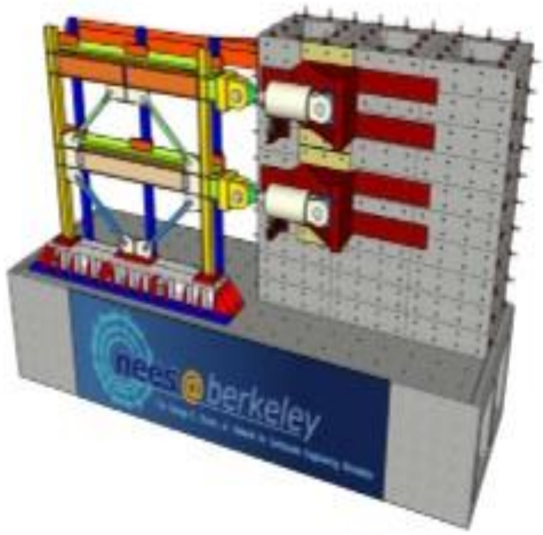
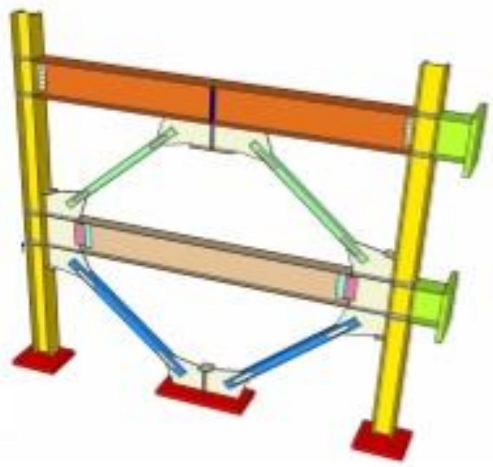


Figure 2.3 Typical brace hysteretic response under symmetrical cyclic loading (Tremblay, 2002)

Chen et al. (2008) examined a number of steel buildings that were modeled in the OpenSees platform. These buildings had steel braced frame system as a primary lateral resisting system and they were designed in accordance with NEHRP (1997) and ASCE-7-05 (ASCE 2005) guidelines. The scope of this investigation was to develop improved design guidelines towards performance-based design of steel braced frames. An emphasis was put on the response modification factor, R, as defined in ASCE-7 (2005). It was found that the structures designed with a low R factor exhibited to decrease seismic demands on the braces. This tendency resulted in a soft story failure mode. However, absolute floor accelerations were increased and this could have an important effect on the non-structural components as a part of the same buildings. The authors performed a parametric study to design four large-scale two-story braced frames and such configurations were tested experimentally, as shown in figure 2.4.



a)



b)

Figure 2.4 Views of a) tentative test setup and b) test specimen (Chen et al., 2008)

Kim and Choi (2005) studied the overstrength, ductility, and the response modification factors of 21 special concentric braced frames (SCBFs) and 9 ordinary concentric braced frames (OCBFs). The pushover analysis was conducted to evaluate the structures for various stories and span lengths. In addition, the results of the static pushover and nonlinear incremental dynamic analysis were compared. From the study, it was concluded that the response modification factor increased when the height of the structure decreased and the span length increased. Apart from three-story structures, the response modification factors of the most SCBF models was found to be smaller that given in the code-specified value of 6.0. Similarly, the response modification factors of all OCBF models was found to be smaller that given in the code-specified value of 5.0.

Sarand et al. (2013) examined a concentrically braced frame. The behavior of this system was controlled by the buckling of first story braces in compression, resulting in failure and loss of lateral resistance. The unexpected failure of the steel structures during past strong seismic excitation led to full fill adequate for modern structures in seismic areas. Concentrically braced frame showed a concentration of damage within a single floor and tendency of strong mechanism formation. The undesired effect of the unbalanced force could be reduced by adding zipper struts which was labeled as zipper frames. Suspended zipper frame had more ductile behavior and higher strength than ordinary zipper frame. The suspended zipper braced system improved the seismic behavior of the frames. In the zipper braced frames, all of the braces participated in seismic energy dissipation. Thus, better damage distribution was achieved and the frame exhibited more strength and ductility. This became more important as the number of stories increased. And, the suspended zipper frames appeared to reduce the tendency of frames to form soft stories and to improve the seismic performance without having to use overly stiff beams.

Özçelik (2011) conducted a nonlinear dynamic analysis in order to examine the six story concentrically braced frame (CBF) structures. An improved hysteresis model for brace members was proposed to use in the analysis. It was proposed that in order to prevent the steel beam failure of a chevron system, unbalance force after brace buckling could be considered. Moreover, the strength of the beam at the braced bay had no significant effect on the lateral strength of the non-moment concentrically chevron braced frame. For the analyzed frame, 1/3 of lateral strength was carried out by the columns in the non-moment concentrically chevron system. A non-moment resisting frame was designed while considering the ductility of braces and assuming a lateral force reduction factor as high as 10. This resulted in excellent behavior. In addition, chevron braced moment resisting frames were designed with ductile braces by assuming lateral force reduction factor as 12. This resulted in acceptable performance under severe ground motions.

Rai and Goel (2003) showed the extensive damage occurred on the steel concentric braced frame (CBF) in the recent earthquake (1994 Northridge earthquake). It was pointed out that the seismic performance of non-ductile CBFs could be improved by delaying the fracture of braces, also, further improvement could be achieved by redesigning the brace and floor beams to a weak brace and strong beam system, as in Special CBFs. However, CBF system had inherent redundancies in the form of the lateral and gravity columns, which could be substantial. It was revealed that the study building could survive another seismic event of similar intensity without collapse if left unrepaired, as shown (Figure 2.5).

a) b) c)

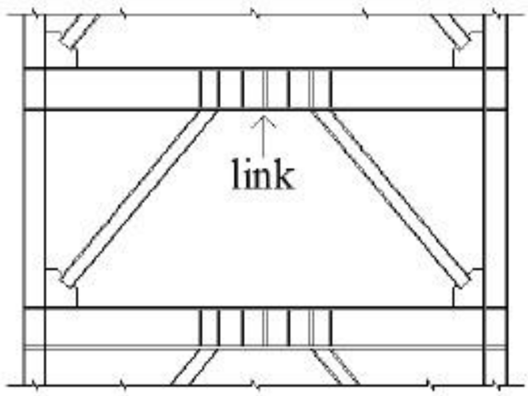
Figure 2.5 Typical examples of observed damage a) buckled brace, b) fractured brace, and c) failed brace to girder connection and lateral displacement of girder (Rai and Goel, 2003)

2.2 Description of eccentrically braced frames

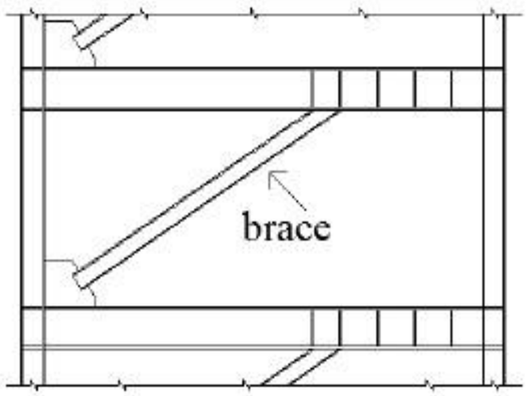
Eccentrically braced frames (EBFs) are one of the most commonly utilized systems of the lateral load resisting systems permitted in ANSI/AISC 341-10 Seismic Provisions for Structural Steel Building (AISC, 341). Moment resisting frames (MRFs) can be designed as a high level of ductility, making them an excellent option to dissipate energy for high seismic events. However, the high level of ductility results in a high cost. MRFs have a lower level of lateral stiffness than EBFs since they lack braces, and the low lateral stiffness of MRFs can cause story drift at levels exceeding drift limitations. As such, MRFs are designed based on the drift instead of the strength, resulting in reduced economy. Conversely, EBFs have a high level of lateral stiffness and a low level of ductility (Popov and Engelhardt, 1988).

For EBFs to be utilized in high seismic regions, a special detailing is required to ensure that the frames behave in the prescribed manner. In the 1970s, a new set of the frame configurations, shown in Figure 2.6, was proposed for the seismic design. The seismic-resisting EBF is the product of decades of the research. Figure 2.6a depicts a modified chevron configuration in which there is one mid-beam link per level; the braces of the above level could be inverted to form a modified two-story X configuration, which would reduce the axial load transferred to the beams (Popov and Engelhardt, 1988).

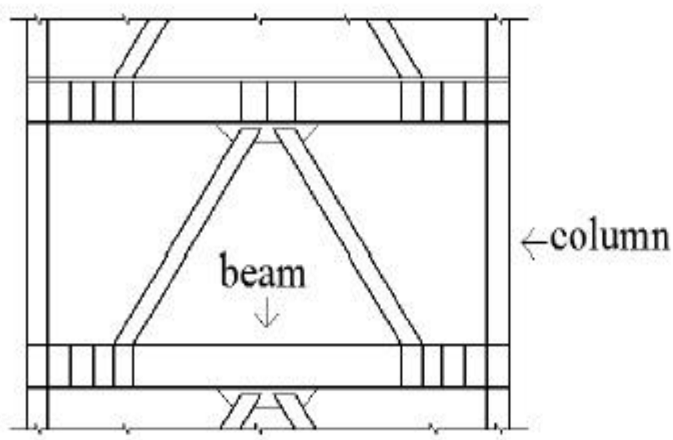
The frame configuration in Figure 2.6b depicts a column-link configuration in which the link is adjacent to one of the frame columns. Figure 2.6c indicates a second modified chevron configuration in which two links are created due to the brace-column eccentricity; in this case, one link is considered active and one passive. The passive link can introduce uncertainty in the inelastic behavior of the frame as the two links do not necessarily equally share the inelastic deformation, as the nomenclature suggests. The EBFs successfully combine the high level of ductility of MRFs and the high level of stiffness of CBFs by introducing eccentricity between a frame cross bracing and column. The cross brace of an EBF provides the elastic stiffness of CBF and the eccentricity of the cross brace creates a link that is responsible for the ductility, and therefore, the energy dissipation capacity of MRF (Popov and Engelhardt, 1988).



a)



b)



c)

Figure 2.6 Configuration of the eccentric brace frame (Popov and Engelhardt, 1988)

Hines (2009) carried out a study on the seismic performance of low-ductility steel systems considering the cost-effective design of the ductile systems for the seismic regions. It was reported that concentrically braced frames (CBFs) were prevalent in the moderate seismic regions. This was due to their high stiffness-to-weight ratio and the ease with which they could be designed and evaluated by the equivalent lateral force method. On the other hand, eccentrically braced frames (EBFs) could offer the same advantages as CBFs, along with providing significant ductility capacity, and greater flexibility with architectural openings. According to the discussion of a test theoretical for 9-story building, an EBF could be designed to conform with the AISC 2005 Seismic Provisions. The capacity design requirements could be kept in check by selecting the smallest possible links to withstand the wind forces. These links could be fabricated separately from the beams outside the link and bolted together as a single element. The extra fabrication effort required for the built-up link beams seemed to be reliable safety benefits of such robust seismic force resisting systems. Since the branding of eccentric bracing as a high-seismic, high-ductility system in the 1990s, however, the use of these schemes had tapered off to almost non-existent as shown in Figure 2.7.

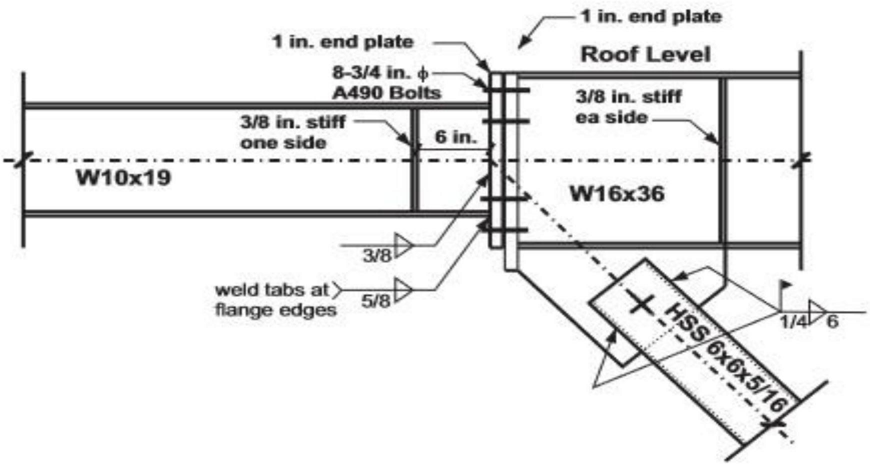


Figure 2.7 Shear link detail (Hines, 2009)

Shayanfar et al. (2008) also studied eccentrically braced frames (EBF). They investigated the stability and hysteretic behavior of the recommended details for double vertical links. To achieve this, the effects of length change of double link on overall behavior of the frames were studied. For this purpose, three eccentric braced frames with double-vertical links were chosen. The characteristics of the selected specimens are revealed in Table 2.1 and Figure 2.8. The chosen configuration showed the connection between the beam and column, and the connection between the column base and brace to the ground. The structural performance of EBFs having double vertical links were discussed comparatively.

Table 2.1 Characteristics of the selected specimens (Shayanfar et al., 2008)

Specimen		Link	e_1 (cm)	e_2 (cm)	t (cm)	h (cm)	L (cm)	N*	V_p (ton)
1		LPE200	20	35	37	320	450	1	29.56
2	a	LPE200	35	35	37	320	450	1	29.56
	b	LPE200	35	35	37	320	450	2	29.56
3		LPE200	50	35	37	320	450	3	29.56

* N is the number of stiffeners

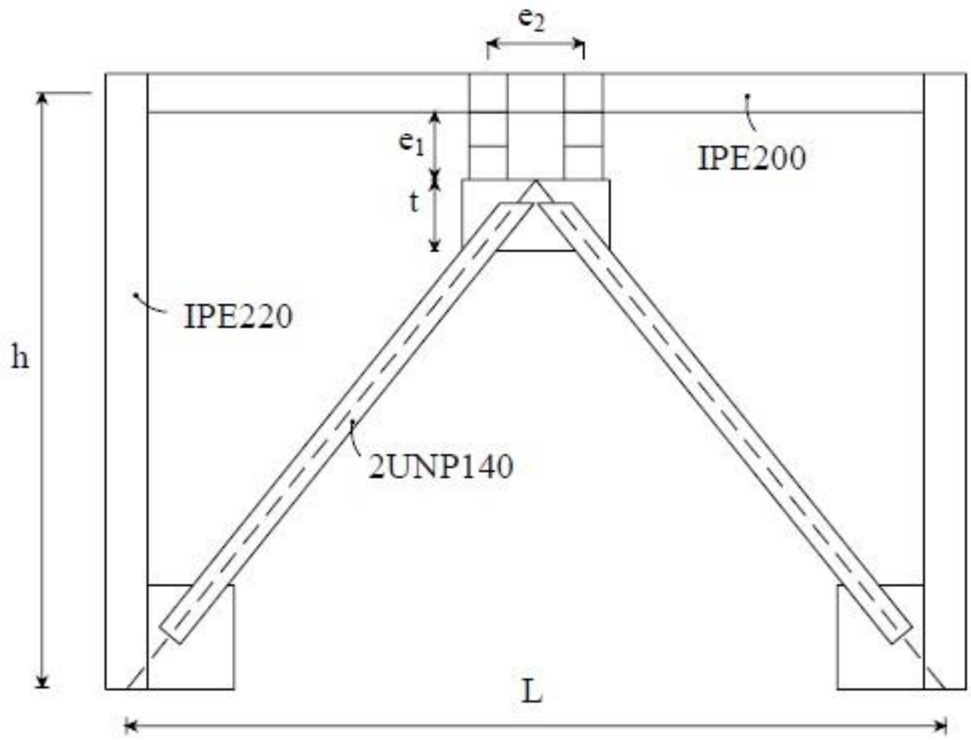
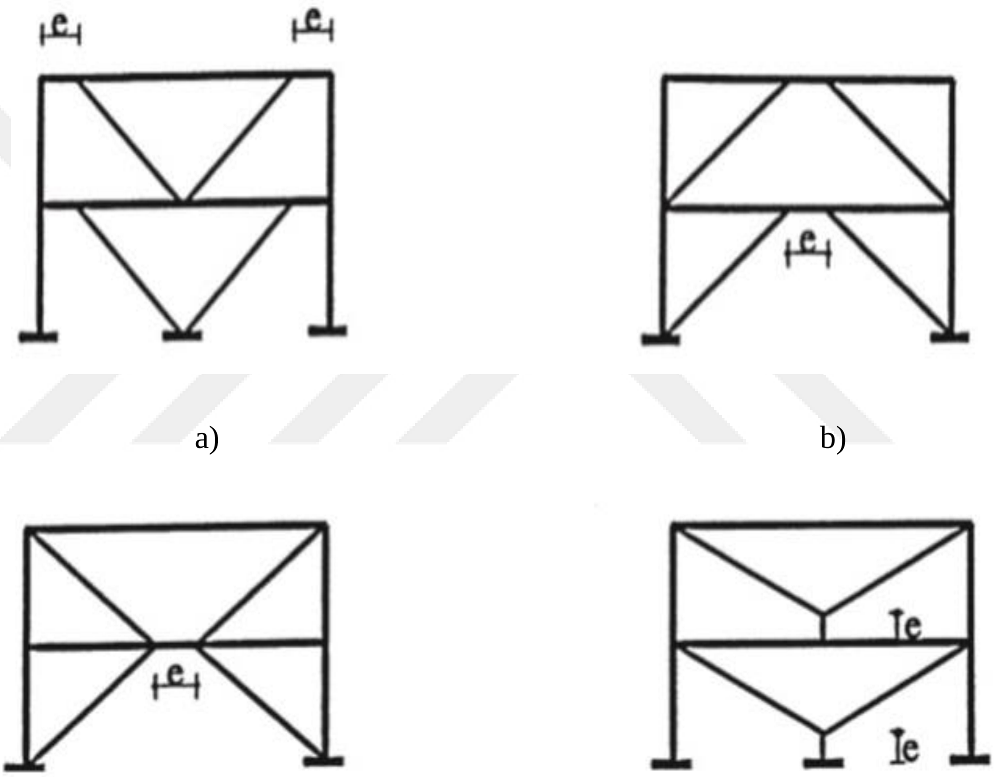


Figure 2.8 The parametric dimensions of the specimens (Shayanfar et al., 2008)

Ghobarah and Elfath (2000) designed on eccentrically braced frame (EBF) in which force was transferred to the brace members through bending and shear force. This was developed in the ductile steel link. In addition, the link was designed in yielding and dissipating energy. Well-designed links provided at stable source and energy dissipation. In their study, examples of different patterns including V-bracing, K-bracing, X-bracing, and Y-bracing are given in Figure 2.9. Most of these patterns might be used short beam segments as active links. For eccentrically braced steel frames, a vertical shear link in EBFs was implemented and used to study the performance of a non-ductile building by using one concentric and two eccentric steel bracing rehabilitation. The analysis of the results showed that the cases of the eccentric bracing had lower deformation and damage when subjected to the earthquake ground motions as compared to the behavior of the concentric bracing case.



a) b) c) d)

Figure 2.9 Different types of eccentrically braced steel frames: a) V-bracing, (b) K-bracing, c) X-bracing, and d) Y-bracing (Ghobarah and Elfath, 2000)

Chimeh and Homami (2012) had a study that steel frame structures having insufficient strength or stiffness against lateral forces were rehabilitated by means of the bracing systems. Braces could be added without substantially increasing the mass of the structures while the structures could be considerably strengthened and stiffened. The behavior of the rehabilitated structures by X braced frames, Chevron braced frames (Inverted-V- braced frames and V braced frames) withand without zipper columns and EBF with long and short link beams were compared with each other. The results showed that the zipper braced frame and short linked EBF were the most ductile systems while the EBF system exhibited the most efficiency.It was also found that the EBF had less stiffness than CBF in ductile behavior. This implied that eccentrically braced frames were a suitable combination of moment resisting frame and concentric braced frame as shown in Figure 2.10. In their study, each eccentrically braced frame was composed of four main elements, namely, 1- Link beam, 2- non link beam, 3- bracing, and 4-column. The main role of absorption and depreciation of inductive energy resulting from an earthquake governed by the link beam. On the other hand, link beams acted like fuses and showed ductile response.

Figure 2.10 Eccentrically braced frame (Chimeh and Homami, 2012)

Zahrai et al. (2013) studied the seismic behavior of eccentrically braced frames as shown in Figure 2.11. The zipper struts were connected at the mid-point of shear links in all storeys. With the goal of evaluating the behavior of shear links, the interaction of the shear links and zipper struts was also studied. Finally, the increase in the ductility coefficient with the use of the zipper strut was observered. The unbalanced shear force was distributed among all stories due to the added continuity among shear links. Moreover, a noticeable delay was occurred until the shear links provided enough rotation to meet collapse prevention acceptance criteria. Thus, it was clear that the shear was the dominant force for these links. Furthermore, the shear capacity of the frames which was obtained through the hysteric analysis showed that the ultimate shear capacity of zipper strut equipped frame was 2%

higher than that of EBF subjected to loads. This difference was as high as 8% for prototypes subjected to a single point load at top story. On the other hand, the participation of shear links in energy dissipation for zipper strut equipped frame was 6% higher than of the regular EBF. This highlighted obviously the influence of adding the zipper struts to the frame.

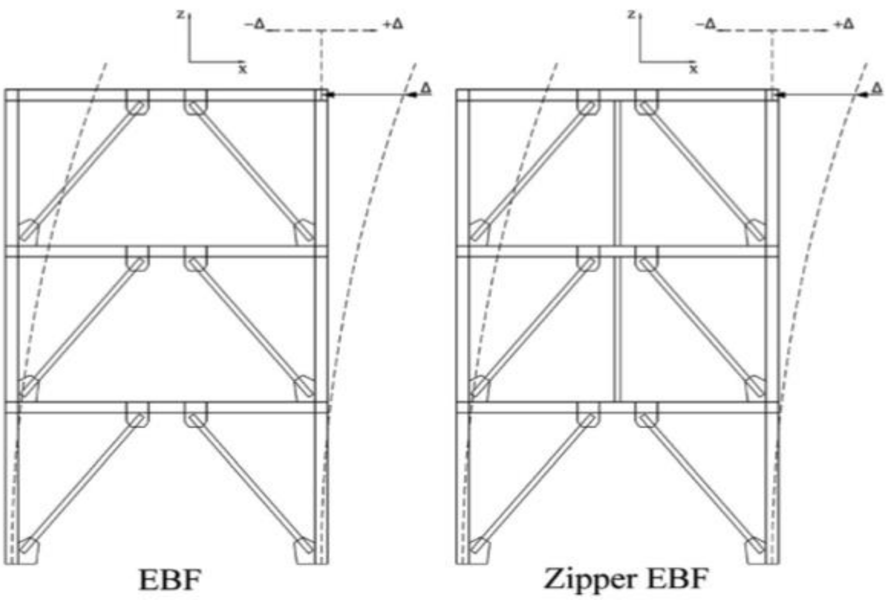


Figure 2.11 The method of imposing displacement on prototypes (Zahrai et al., 2013)

Koboevic et al. (2012) studied the seismic response of three and eight story eccentrically braced frame structures designed for the western and eastern North American locations. Non-linear time history analyse was performed by using the computer programs of the OpenSees software platform. The inelastic behavior of the EBF link element was predicted using the hysteretic material. The parameters for the steel material were determined from the calibration with the past test results of EBF specimens. Rotational zero length spring elements were also included at the brace ends to account for the end restraint conditions induced by the gusset plates. Realistic responses of the frame members, other than the link element, were also obtained by modeling them with eight non-linear beam-column elements together with 16 fibers for cross-section discretization. The study found a strong correlation between the plastic link rotations and the inter-story drifts. Moreover, the study confirmed that the flexural yielding of outer beams was acceptable for EBFs with short and intermediate links under certain circumstances.

2.3 Description of zipper braced frames

In zipper braced frames, when a brace buckled, the unbalanced vertical force was transmitted to the "zipper column" as tension force. The column redistributed the force to the upper story braces as an extra compression force. This forced the upper story compression brace to buckle. In this case, 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 the concentration of damage in just one story. Plastic hinges also developed at the base of the columns and at the mid span of the beams. This was the plastic failure mechanism of the “zipper frame”. So, the proposed zipper columns for decreasing the adverse effect of unbalanced force and called as the full-height zipper mechanism. This system had a good distribution of forces to dissipate the energy over the height of the structures. At the same time, the buckling of the compression braces caused more uniform distribution of damage that was the required objective. However, the full-height zipper systems had some seismic drawbacks, for example, compression member buckled directly and collapse failure occurred immediately. This system had adverse force redistribution capability (Khatib et al., 1988).

Kim et al. (2008) analyzed the zipper brace. In their study, simple static design was first presented, and then a dynamic design method (which could consider the effects of the brace slenderness and higher modes) was proposed by combining the refined physical theory and the modal pushover analysis. Inelastic analysis based on both the static and dynamic design methods proposed in this study showed that the seismic performance of the wise with zipper column had better than the case without zipper column. The simple static design method proposed to invoke at least two-story buckling mechanism equally worked well in this limited case study. 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 the most of the zipper columns since the braces in each story would not buckle simultaneously as given in Figure 2.12.

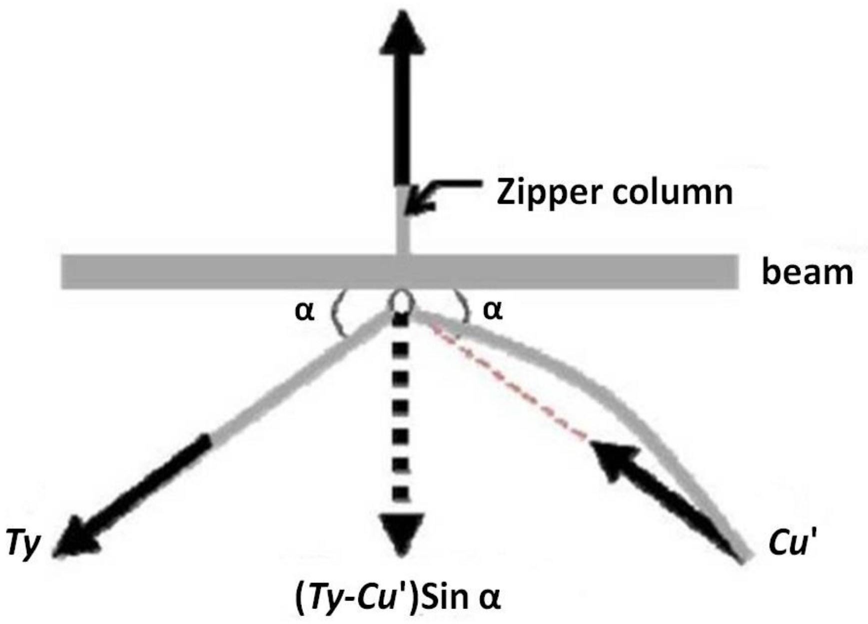


Figure 2.12 Post buckling vertical unbalance force (Kim et al., 2008)

Karimi et al. (2013) studied the behavior of zipper braced frame. For this, the zipper column was added at the beam mid span from the second to the top story of the frame as shown in Figure 2.13. The ductility and response modification factor of the frames with zipper braces were compared with those having concentric braces. It was observed that adding vertical zipper member resulted in positive effects on the behavior of the frames. Such modification caused to distribute unbalanced forces at the height of the frame suitably and also caused plastic hinges to be created in compressive members. Vertical displacement of the mid span point of the beam at the first floor, which showed the weakness of the Chevron bracing member, was modified and remained constant for all floors at the height of the frame. In addition, the lateral displacement focusing at the first floor was eliminated and then distributed at the height of the frame. Finally, it was clear that the amount of absorbing energy in zipper bracing members was more than chevron ones.

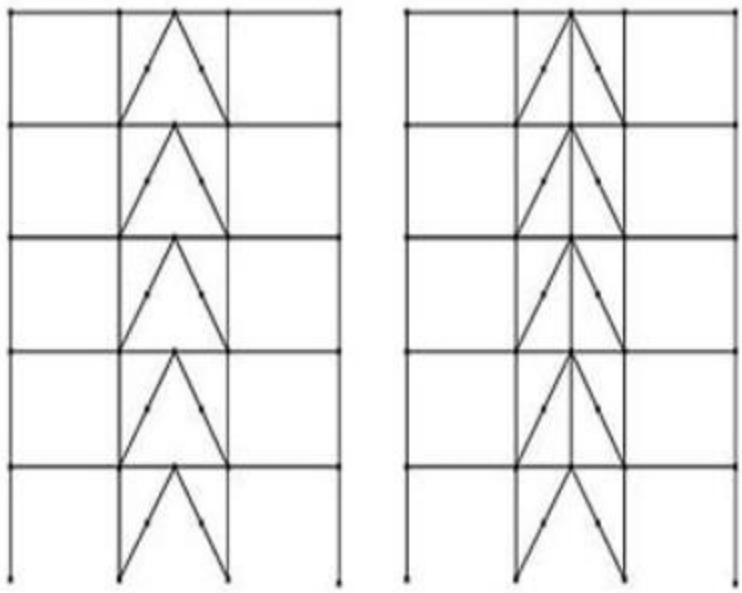
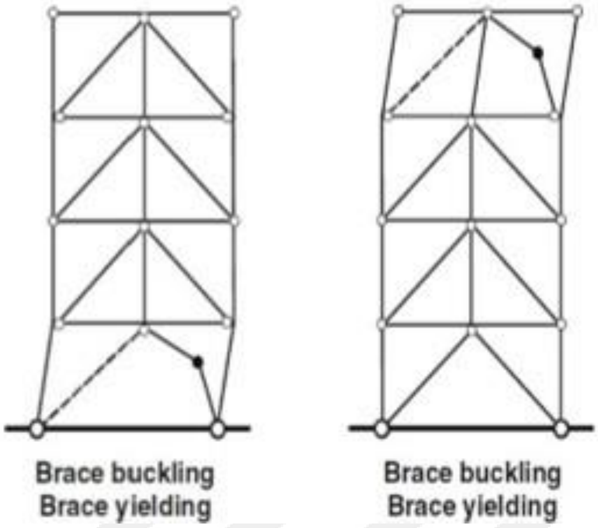


Figure 2.13 Brace configuration (Karimi et al., 2013)

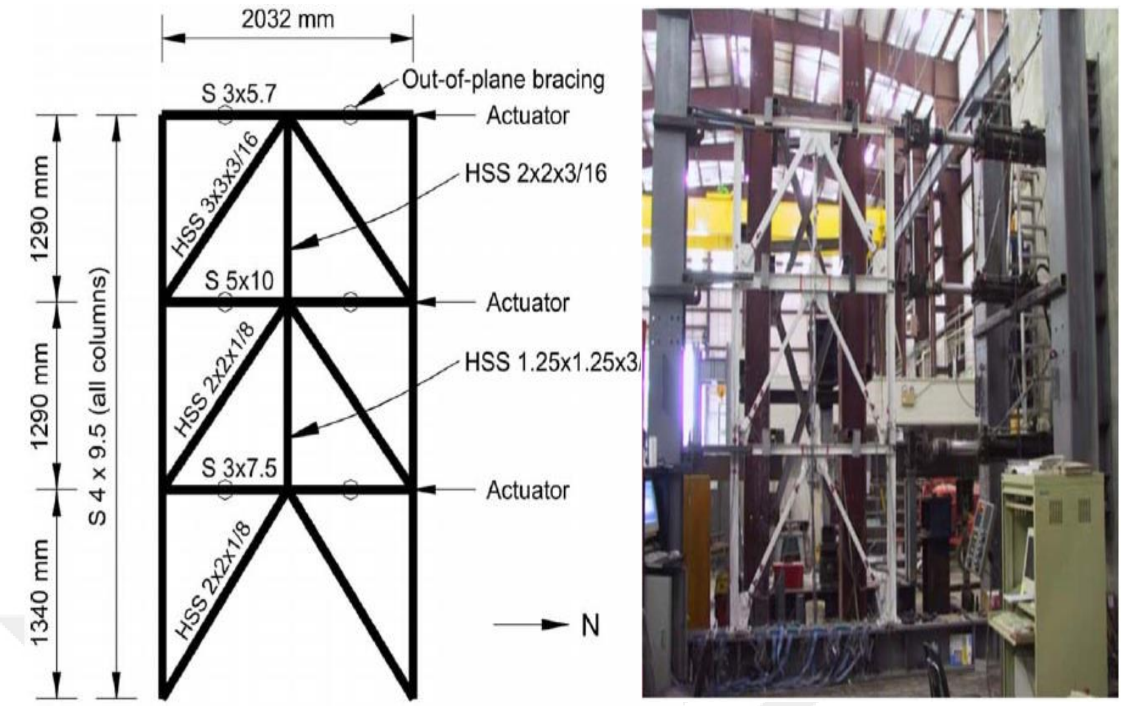
Tirca and Tremblay (2003) proposed a design method that relied on the ability of zippers to behave elastically. Based on their proposed design methodology, three zipper braced frame buildings (4, 8, and 12 story) were designed and investigated. Examination of the inelastic behavior of the aforementioned braced frames revealed that both critical scenarios of zippers acting in tension and compression could be treated separately. When the brace buckling initiated at the bottom story and propagated upward in the frame, zipper columns were subjected to tensile forces due to the subsequent buckling of braces as shown in Figure 2.14a. On the other hand, when the first buckled brace was located at the top floor, the buckling of braces propagated downward, and then the unbalance vertical forces projected from the braces to mid-span of the beams were transferred as compressive forces in zipper columns (Figure 2.14b). Therefore, the zipper columns were designed to carry the unbalanced load developed at the mid-span of the beams after braces buckled. To assess the force in zippers and their required compressive and tensile strengths, the following two scenarios were proposed: zippers could act in tension when the first brace would buckle at the base and zippers act in compression when the first brace buckles at the top of the structure. The zipper struts are designed to withstand both of the maximum compressive force and the maximum tensile force which would be induced by the internal forces which are equal to the probable buckling or post buckling capacity and the tensile capacity of braces. In order to make the zipper braced frame respond as predicted, the zipper columns must remain elastic throughout the entire seismic excitations.



a) b)

Figure 2.14 Behavior of zipper braced frame system with strong zipper columns: a) brace buckling initiated at the base and b) brace buckling initiated at the roof (Tirca and Tremblay, 2004)

Yang et al. (2008) the aimed to design a zipper braced frame to achieve more ductile behavior. Three zipper braced models (3, 6, 20 stories) were designed by using SAC model buildings. The models were showed to estimate the overstrength, inelastic strength, and deformation capacities for the entire structures. Furthermore, the models were evaluated using the nonlinear dynamic analysis. It was concluded that the design method produced safe designs such that the design became more conservative as the number of stories increased. The distribution of interstory drifts demonstrated the efficiency of the zipper struts in achieving uniform damage over the height of the structure, as shown Figure 2.15.



a) **b)**

Figure 2.15 Details of a) the elevation of the test frame and b) photo of the overall test setup (Yang et al., 2008a)

Razavi and Sheidaii (2012) studied the zipper elements which transferred unbalanced vertical forces of the lower stories to the upper ones. The tensile forces generated in these elements extremely increased in upper stories. Accordingly, these zipper elements needed an impractically large cross-section to be designed. This problem induced some limitations on the use of zipper bracing systems, especially in high-rise buildings. Therefore in their study, a novel approach was presented to resolve this problem. The proposed solution used cables with appropriate pre-stress ratios as zipper elements. Accordingly, the seismic behavior of cable zipper-braced frames with different pre-stress ratios was investigated. Moreover, it was shown that the use of the suggested system with appropriately pre-stressed cables enhanced the seismic performance of zipper-braced systems. It was concluded that the pre-stressed ratio and the number of story were very effective on the seismic performance of the frames with zipper elements. Therefore, the analysis was performed for 3, 6, 9, 12, and 15 story zipper-braced frames, with and without cables (Figure 2.16). These frames were investigated with two pre-stress ratios of 5% and 20% under seven scaled seismic ground motions. Cables were adjusted so that the relaxation did not occur in these elements. It was obvious that the maximum tensile force of zipper elements in upper stories was larger than those of lower ones. Conversely, the maximum compressive force in the upper zipper elements was smaller than lower ones except for mid and high rise models in which slight increment was appeared in the middle stories.

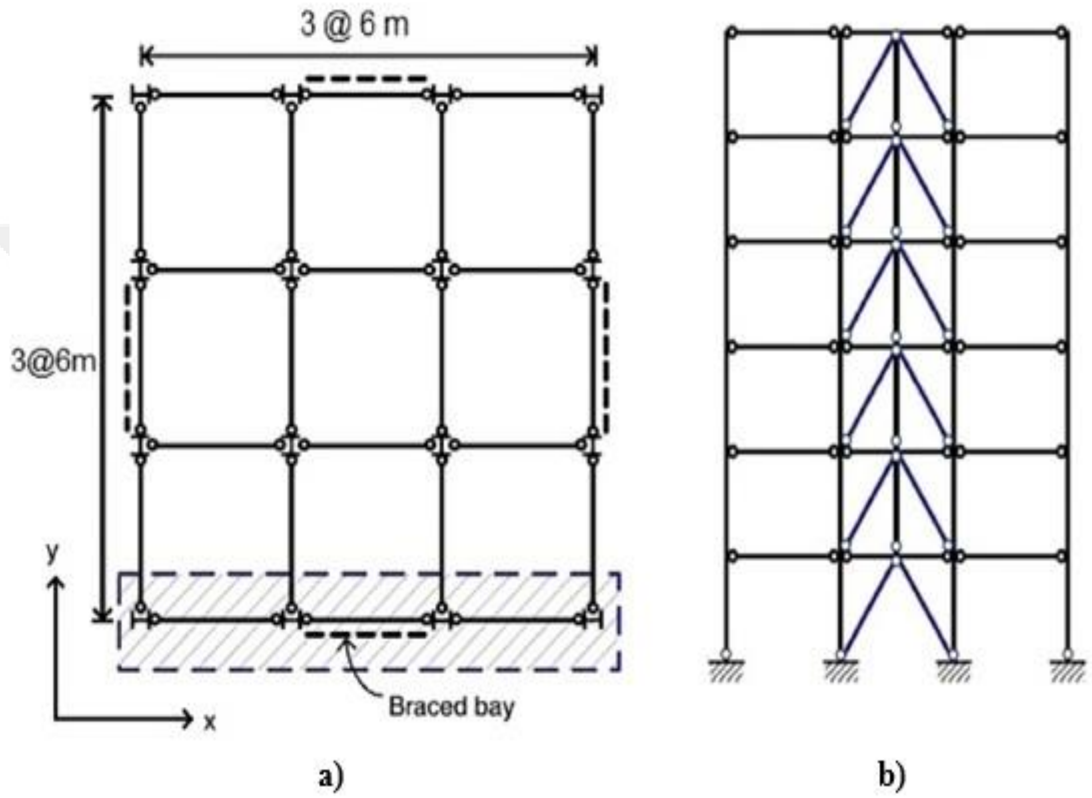
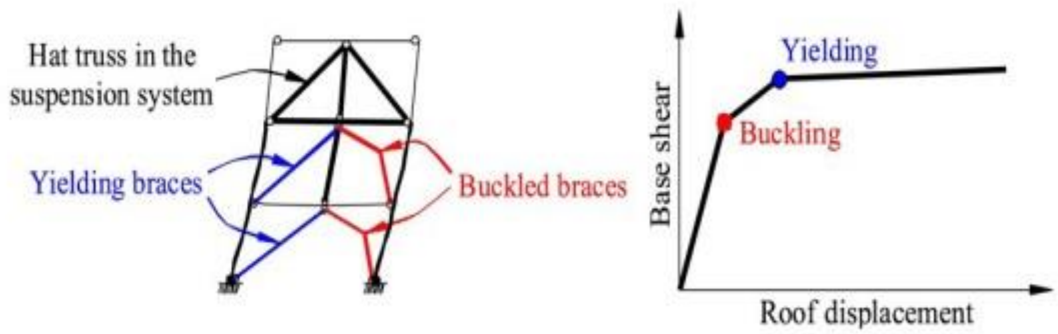


Figure 2.16 The model structures a) plan and b) elevation views (Razavi and Sheidaii, 2012)

Leon and Yang (2003) proposed a modified zipper braced frame called "suspended zipper frame". The modified system consisted of a zipper frame system with a hat truss located at the top floor level. The purpose of this modification was to keep the top level braces behaving in elastic range and to avoid the formation of a full-height zipper mechanism. In this approach, the failure was defined when the partial- height zipper mechanism was formed. In a suspended zipper frame, the top level braces remained in elastic range while all other compression braces in other stories buckled. The function of the suspended zipper columns was to transfer the unbalanced vertical loads developed due to the braces buckling at floors below and to support the beams at their mid-span. As a result, the beams could be designed to form plastic hinge at mid-span. Therefore, significant savings in the amount of steel was made for sizing beams to perform in the plastic range. Meanwhile, the system had a clear force path which made a capacity design for all the structural members' straight forward. The configuration and expected behavior of the suspended zipper frame was also explained in the study of Sarand et al, (2013) as shown in Figure 2.17.



a) **b)**

Figure 2.17 Behavior of zipper braced frame with suspended zipper strut (Sarand et al., 2013)

In the study of Naeimi et al. (2012), the zipper struts were used to enforce all compressive braces to buckle. The benefit of this behavior was that all stories had contribution in the energy dissipation. For instance, if the compressive brace of the first story could buckle, an unbalanced force would be imposed to the mid-span of the first floor beam (Figure 2.18). This unbalanced force was transferred to the intersection of the second story beam and braces and increased the compressive force in the brace. This led to the buckling of the compressive brace of the second story. This process continued until the buckling of the compressive brace. Although the buckling of all compressive braces resulted in a uniform distribution of the energy dissipation in the height of the structure, it was not always a good result. Due to the formation of the complete zipper mechanism in the height of the structure, overall instability and failure could occur in the system. This shortcoming limited the use of this system.

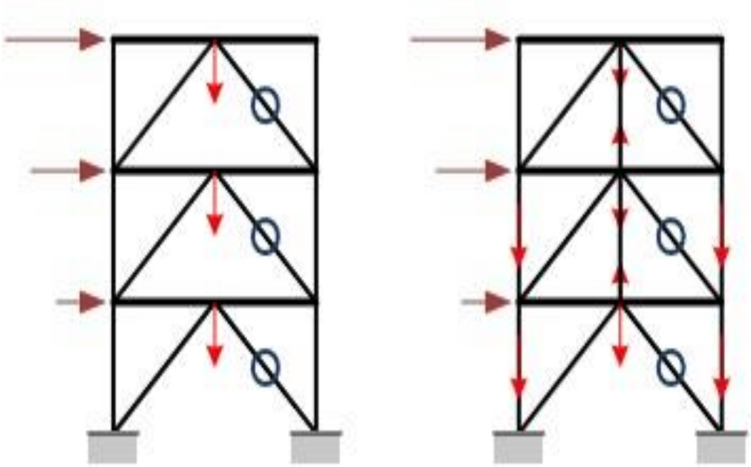


Figure 2.18 Transformation of vertical unbalanced force by the zipper strut (Naeimi et al., 2012)

CHAPTER 3

CASE STUDY

3.1 Analytical model of the structures

Three, six and eight story steel buildings were designed. Each story had a height of 3 m, the exterior and interior frames of the buildings comprised three bays. 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. For example, in the three-story building, the dimensions of the interior beam is W10x17 and exterior beam is W10x22. There was a difference in the dimensions of interior column in the first floor (W12x35), second floor (W12x30) and third floor (W12x22), also, in the exterior column of first floor (W10x45), second floor (W10x30), and third floor (W10x26). The column foundation was considered as fixed for all cases. Moreover, the structural steel braces were used as I section in the configuration of single and double zipper bracing of three-story building, the dimension of brace was W10x12. And, the zipper element (zipper column) on the top floor was selected as W8x13. Table 3.1 shows the member size of the three-story building.

Table 3.1 Selected member size of the three-story building

Story	Interior Column	Exterior Column	Interior Beam	Exterior Beam	Brace	Zipper Column
3	W12x22	W10x26	W10x17	W10x22	W10x12	W8x13
2	W12x30	W10x30	W10x17	W10x22	W10x12	W8x13
1	W12x35	W10x45	W10x17	W10x22	W10x12	-

For the six-storey buildings, the dimensions of the column and beam (internal and external) was varied. For example, the dimensions of the interior beam was W10x17 and the exterior beam was W10x26. Similarly, the dimensions of the column in the 1-2 story was W10x77, 3-4 story was W10x54, and 5-6 story was W10x33. Moreover, the structural steel braces were used as tube section in the configuration of single and double zipper bracing of the six-story building, the dimension of brace was selected as HSS7x2x1/2. And, the zipper element (zipper column) were used as I section, on the top floor was W10x12. Table 3.2 demonstrates the member size of the six-story building.

Table 3.2 Selected member size of the six-story building

Story	Column	Interior Beam	Exterior Beam	Brace	Zipper Column
6	W10x33	W10x17	W10x26	HSS7x2x1/2	W10x12
5	W10x33	W10x17	W10x26	HSS7x2x1/2	W10x12
4	W10x54	W10x17	W10x26	HSS7x2x1/2	W10x12
3	W10x54	W10x17	W10x26	HSS7x2x1/2	W10x12
2	W10x77	W10x17	W10x26	HSS7x2x1/2	W10x12
1	W10x77	W10x17	W10x26	HSS7x2x1/2	-

For the eight-story building, the dimensions of the column and beam were as follows, the dimension of the interior beam was W10x19 and the exterior beam was W10x26. The dimension of the column in 1 to 3 story was W10x77, 4-5 was W10x54, and 6 to 8 was W10x33, as shown in Table 3.3. And, the structural steel braces were used as I section in the configuration of single and double zipper bracing of the eight-story building, the dimension of brace was W10x22. And the dimension of brace was W8x67. The design live load for the building was taken as 2.00 kN/m^2. The modulus of the elasticity of steel used was 200 GPa and its yield stress was 235 MPa. All the steel elements had the same material properties. The steel frame models with three different numbers of the stories (3, 6, and 8) were considered as shown in Figures, 3.1. 3.2, and 3.3, respectively. As seen from the figures, each building was retrofitted by the zipper bracing system considering the single or double configuration. Thus, within the scope of this study, nine different frame cases were taken into account.

Table 3.3 Selected member size of the eight story building

Story	Column	Interior Beam	Exterior Beam	Brace	Zipper Column
8	W10x33	W10x19	W10x22	W10x17	W8x67
7	W10x33	W10x19	W10x22	W10x17	W8x67
6	W10x33	W10x19	W10x22	W10x17	W8x67
5	W10x54	W10x19	W10x22	W10x17	W8x67
4	W10x54	W10x19	W10x22	W10x17	W8x67
3	W10x77	W10x19	W10x22	W10x17	W8x67
2	W10x77	W10x19	W10x22	W10x17	W8x67
1	W10x77	W10x19	W10x22	W10x17	-

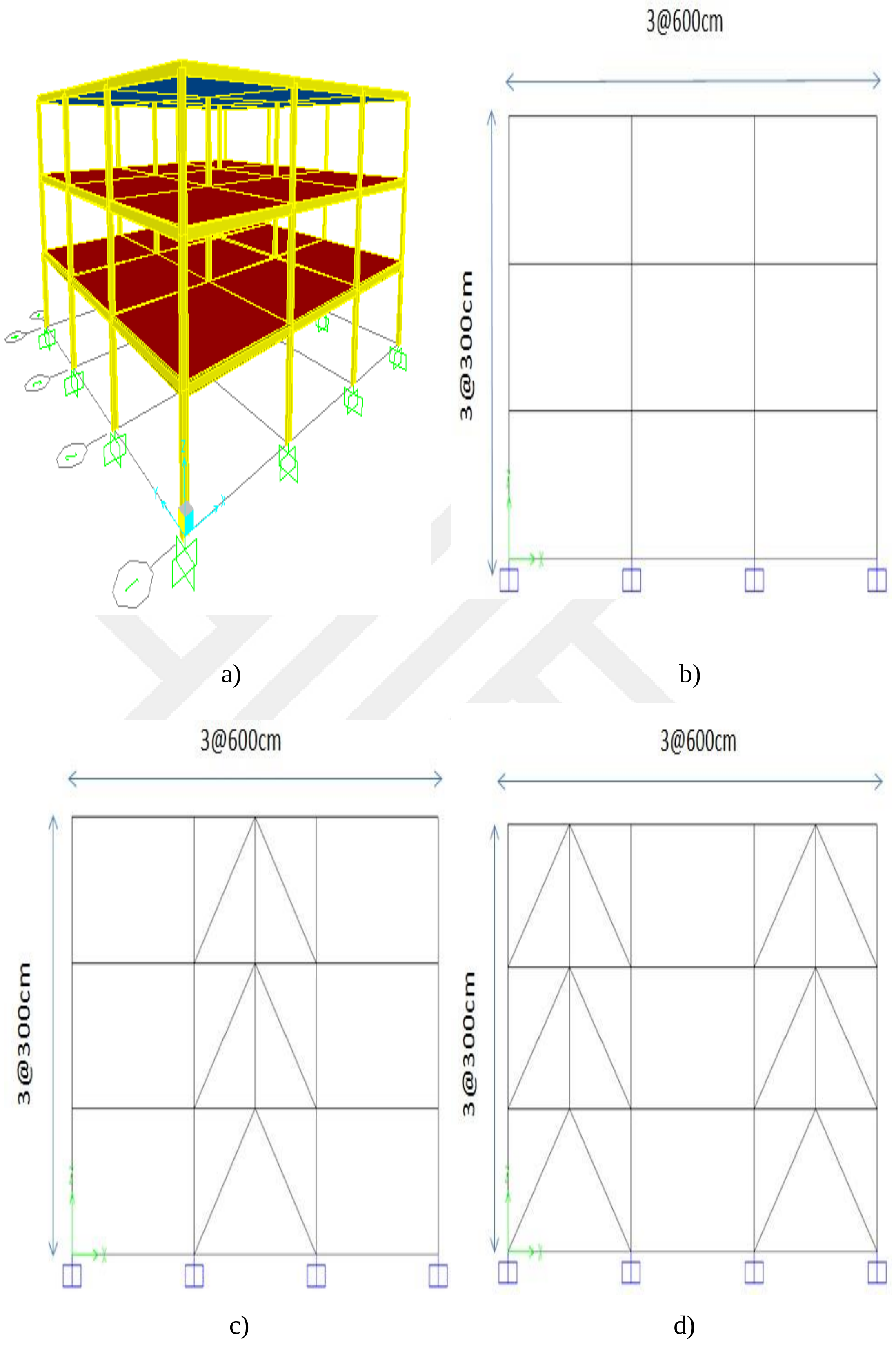


Figure 3.1 Three-story building: a) 3-dimensional view, b) elevation of the frame, c) the frame modified with single zipper brace, and d) the frame modified with double zipper brace

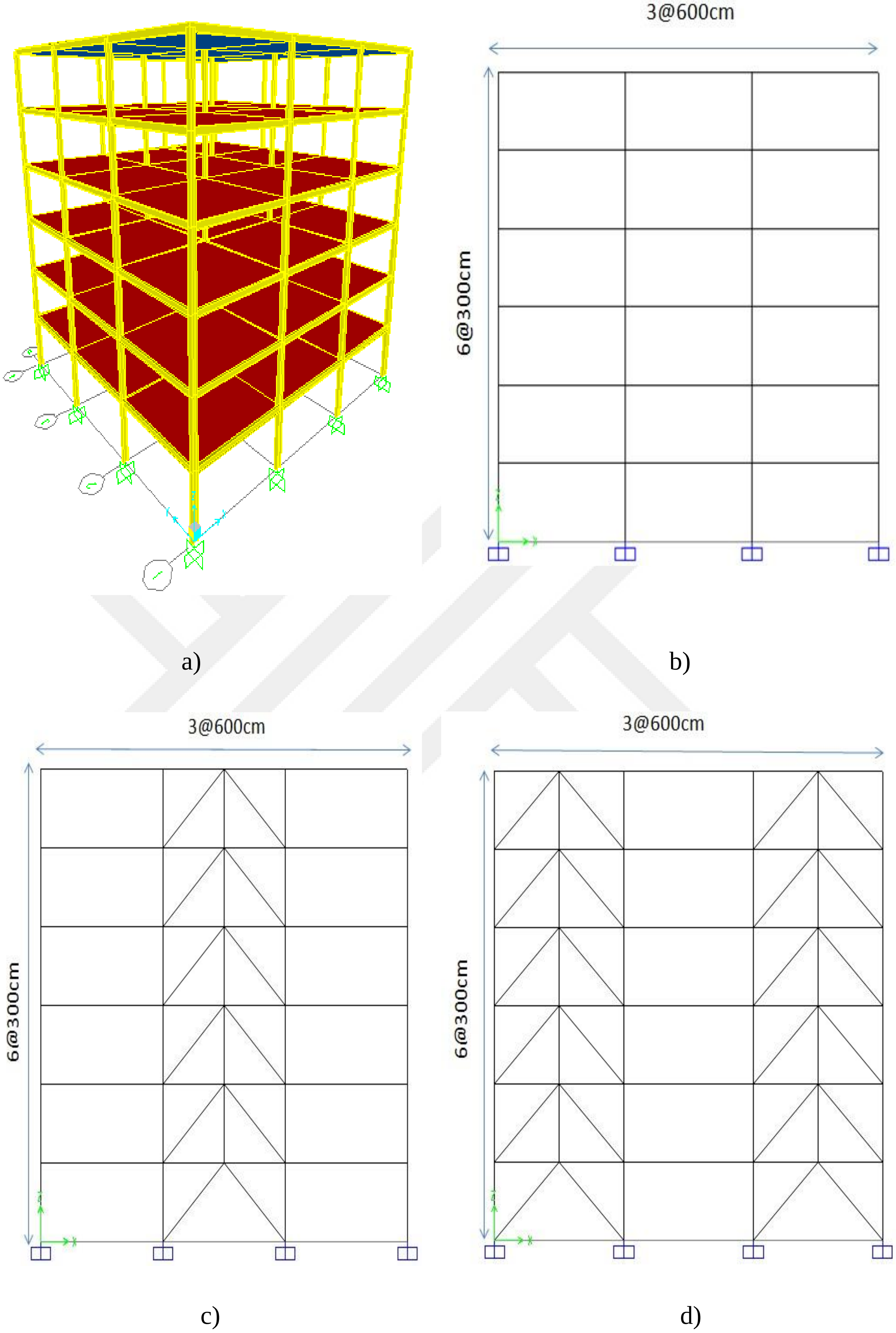


Figure 3.2 Six-story building: a) 3-dimensional view, b) elevation of the frame, c) the frame modified with single zipper brace, and d) the frame modified with double zipper brace

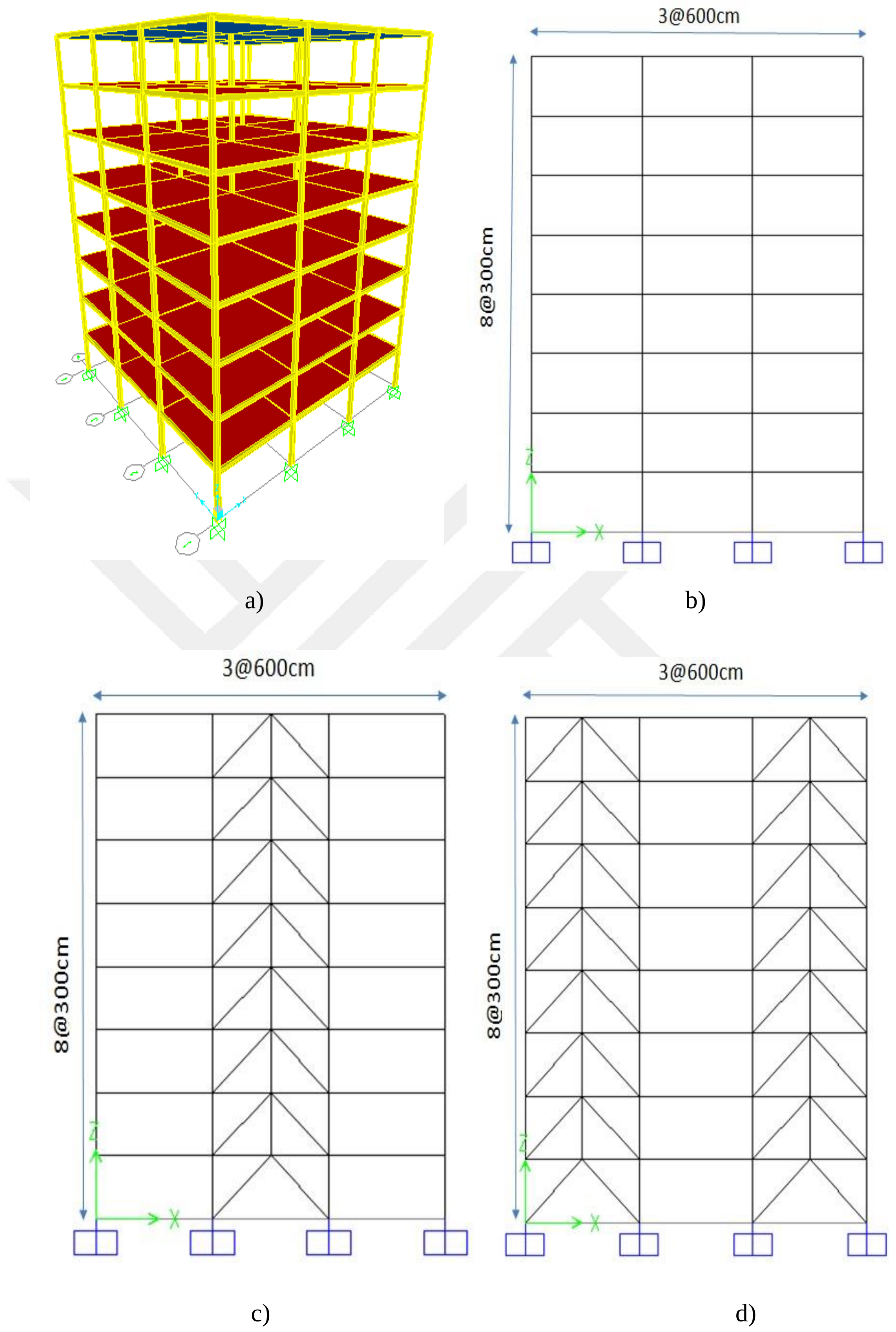


Figure 3.3 Eight-story building: a) 3-dimensional view, b) elevation of the frame, c) the frame modified with single zipper brace, and d) the frame modified with double zipper brace

3.2 Nonlinear analysis method

Although the process of changing stiffness is common to all types of nonlinear analyses, the origin of nonlinear behavior can be different, making it logical to classify nonlinear analyses based on the principal origin of the nonlinearity. Because it isn't possible to point out a single cause of nonlinear behavior in many problems, some analyses may have to account for more than one type of the nonlinearity.

3.2.1 Nonlinear static (pushover) procedure (NSP)

A pushover analysis is an incremental plastic analysis. In this analysis, the monotonically increasing lateral loads of constant relative magnitude are applied to a structure and increased until target displacement is reached. The gravity loads should be kept constant during the analysis. Thus, the structure is actually pushed over. The aim of the analysis is mainly to determine its ultimate lateral load resistance capacity and also sequence and magnitude of classifications when reaching target displacement point. That invention of pushover analysis has taken place when many engineers have achieved this procedure by running repeated linear elastic structural analyses by computer programs and modified the model of the structure for the progressive changes in each increment in the structure (Bruneau et al., 1998).

In the NSP, a detailed mathematical model of the nonlinear load-deformation characteristics of the building could be subjected to incremental lateral loads that represent inertia forces in an earthquake until a target displacement is reached. It continues to add that the target displacement represents the maximum displacement that is expected to be experienced by the structure during the design earthquake. The calculated internal forces of elements would be reasonable approximations of those expected during the design earthquake since the mathematical model takes the effects of material inelastic response into account according to FEMA 356 (2000).

Another usage of pushover analysis is the highlighting of the potentially weak areas in the structure. The NSP is applying a lateral load with a predefined pattern distributed along the building height. The lateral forces are then incrementally increased with a displacement control point at the top of the building until a specific level of deformation is reached. The drift corresponding to structural collapse may be the deformation expected in the design earthquake for assessment purposes or the roof displacement in case of designing a new structure. The NSP also demonstrates the sequence of yielding and failure on the structural elements and the structure and also the pattern of the overall response curve of the structure (Mwafy and Elnashai, 2001).

3.2.2 Nonlinear dynamic procedure (NDP)

The nonlinear dynamic procedure (NDP) is applied for the seismic analysis of the building, a mathematical model of the frame should be subjected to earthquake shaking which is represented by ground motion time histories. This model should account for the nonlinear load-deformation characteristics of individual components and elements of the building. It can be reported that time history analysis is used for the response calculations. With the NDP, the design displacements are determined directly through dynamic analysis using ground motion time histories instead of using a target displacement according to FEMA 356 (2000).

It can be stated that the NDP is a powerful tool for structural seismic response study. Accurate estimation of the anticipated seismic performance of structures can be reached by a set of carefully selected ground motion records. However, they continue to state its disadvantage as the great sensitivity of the calculated inelastic dynamic response to the characteristics of the input motions. And as a solution to this problem, they describe that the evaluation of the strength capacity in the post-elastic range can simply be done by NDP (Mwafy and Elnashai, 2001).

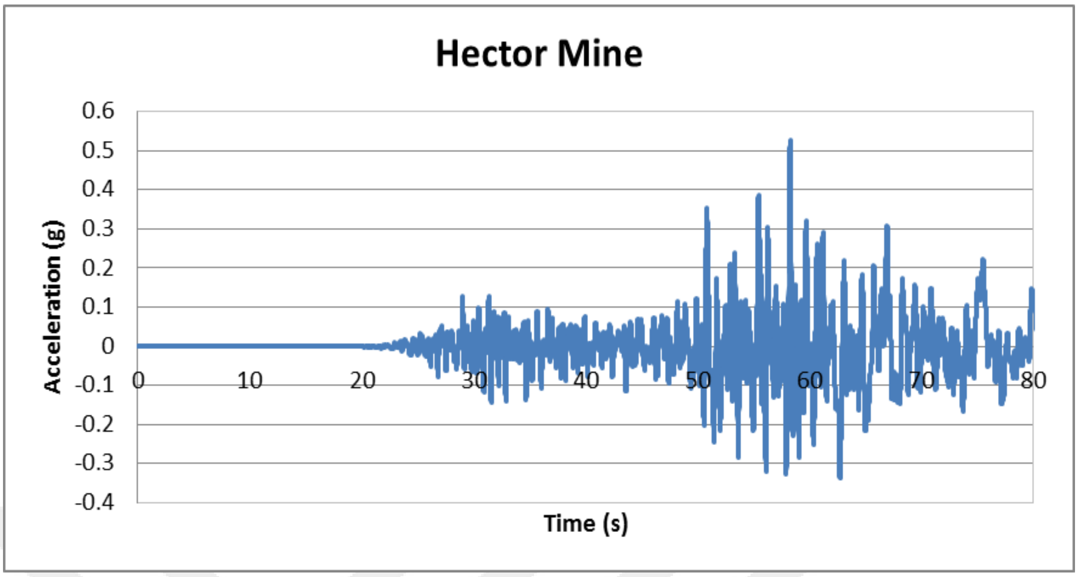
It is also noted that the NDP predicts the forces and cumulative deformation (damage) demands in every element of the structural system with sufficient reliability and is the final solution for the structural analysis. But they continue to add that the solution implementation needs the availability of a set of ground motion records for accounting the uncertainties and differences in severity, frequency characteristics, and duration because of distances and rupture characteristics of the various faults that may cause motions at the site (Krawinkler and Seneviratna, 1998).

3.3 Details of analysis in this study

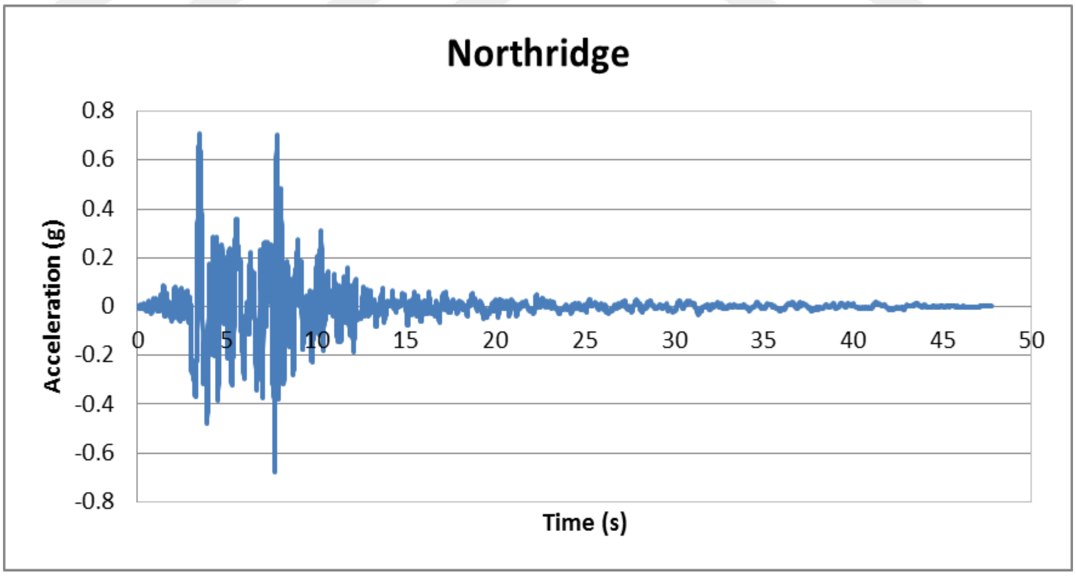
According the conducted research here, the nonlinear static and dynamic analysis were carried out by the usage of the finite element program of SAP 2000 version 14 (CSI, 2009) for the original steel frames with and without the single and double zipper braces. This was done in order to specify the actual nonlinear behavior of buildings. In this method, the buildings were subjected to real ground motions, namely, Hector Mine (1999), Northridge (1994), and Chi-Chi (1999). Hence, the inertial forces were determined from the ground motions. The seismic behavior of the original, single and double zipper braced frames was investigated under different earthquake ground accelerations. The analytical models, which had the nonlinear behavior of the structural members, were exposed 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). In Figure 3.4, the complete time history records of the earthquakes are given. Moreover, the design code spectrum and elastic spectra of the scaled natural ground accelerations are given in Figure 3.5. Furthermore, the characteristic properties of the natural ground motions such as the magnitude (Mw), 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 3.4.

Table 3.4 Characteristic properties of the scaled earthquake ground motions

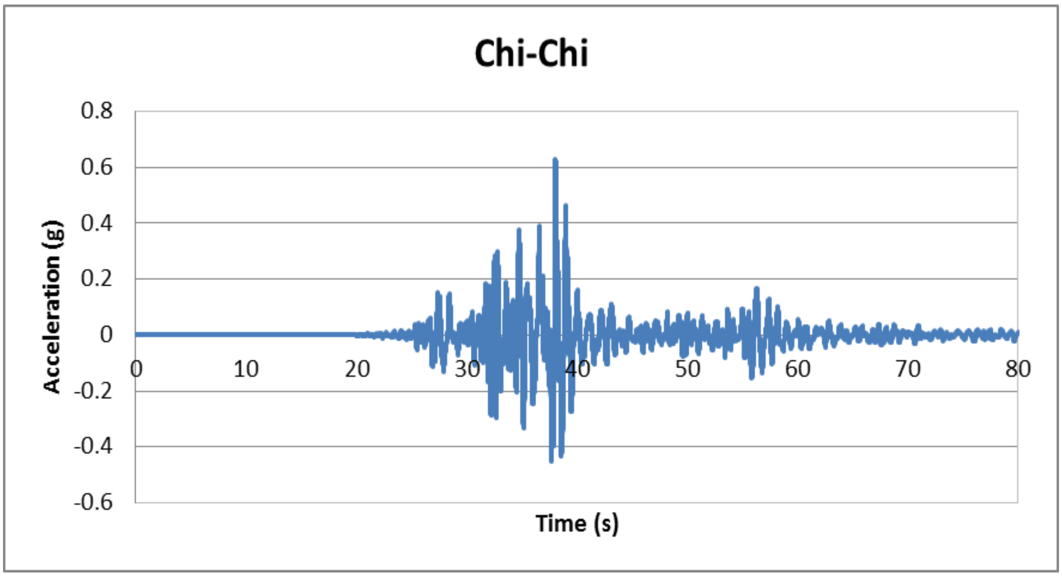
Earthquake Record	Hector Mine	Northridge	Chi-Chi
Year	1999	1994	1999
Magnitude (Mw)	7.13	6.69	7.62
Mechanism	Strike-Slip	Strike-Slip	Reverse-Oblique
Vs30(m/s)	294.2	380.1	680
PGA(g)	0.52826	0.7072	0.6303
PGV(cm/s)	103.73	69.979	74.3227
PGD(cm)	151.966	14.5451	22.3926


Hector Mine
Acceleration (g)
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
0
10
20
30
40
50
60
70
80
Time (s)

a)


Northridge
Acceleration (g)
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
0
5
10
15
20
25
30
35
40
45
50
Time (s)

b)



c)

Figure 3.4 Ground accelerations for a) Hector Mine, b) Northridge, and c) Chi-Chi earthquakes

Although the conducted research up to failed, it was possible to specify some yielding points of the system. The plastic rotating was also monitored on the frames, and the lateral inelastic forces versus displacement response for the complete structure was analytically computed. The hinge properties of the structural components, according to FEMA 356 (2000), were determined considering the component type and failure mechanism. After defining the plastic hinge properties in the model, the structures were subjected to monotonically increasing lateral forces until a specified displacement was reached. The capacity curves related to the base shear force versus roof displacement for 3, 6, and 8 story structures for the original steel frames, and those with single and double zipper braces as in the case study. The elastic design spectrum was obtained according to Turkish Earthquake Code. The first seismic zone and soil type Z4 for the seismic hazard level of 2% probability of exceedance in 50 years were considered.

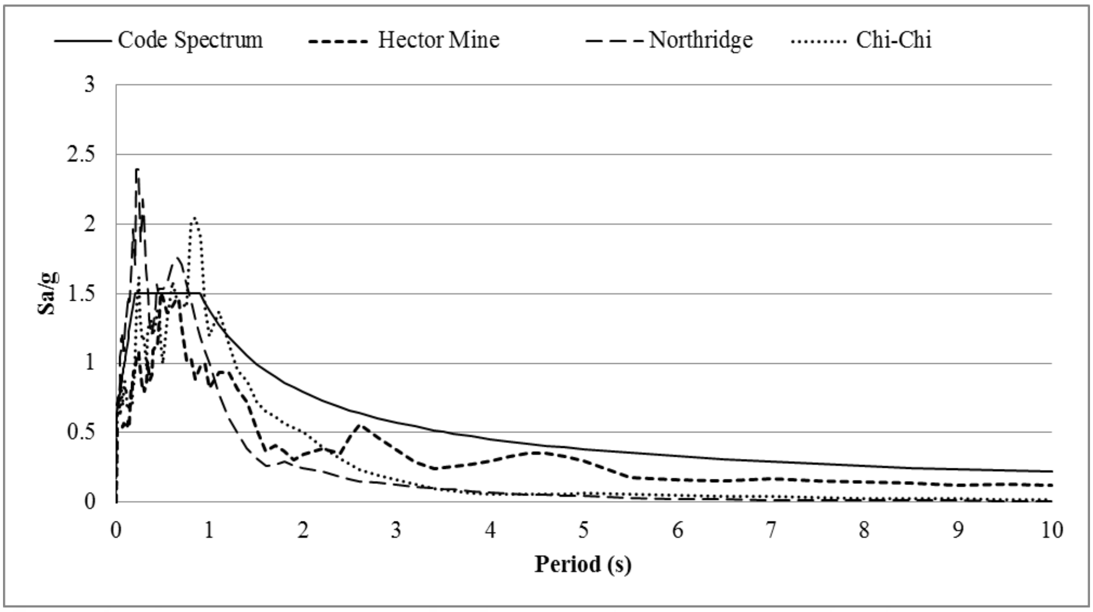


Figure 3.5 Elastic spectral accelerations of the earthquake

The elastic behavior occurred over the member of length, and the deformation beyond the elastic limit occurred afterwards 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). A series of hinges were modeled. Multiple hinges were also coinciding at the same location to capture plasticity distributed along member length. Plasticity was associated with force-displacement behaviors or moment-rotation. 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 3.5 shows the first three fundamental periods of the structures. Moreover, the hinge formations of the structures under the earthquake loading are illustrated in the Appendix part.

Table 3.5 Fundamental periods of the original, single and double zipper braced frames

Type of frame	T_1 (s)	T_2 (s)	T_3 (s)
3 story original frame	0.867	0.252	0.130
3 story single zipper braced frame	0.309	0.109	0.069
3 story double zipper braced frame	0.183	0.068	0.057
6 story original frame	1.589	0.502	0.265
6 story single zipper braced frame	0.455	0.158	0.088
6 story double zipper braced frame	0.334	0.118	0.078
8 story original frame	2.248	0.725	0.393
8 story single zipper braced frame	0.616	0.204	0.108
8 story double zipper braced frame	0.455	0.152	0.091

CHAPTER 4

RESULTS AND DISCUSSION

4.1 General

In this chapter, the analysis of results for the original frames and modified frames with single and double zipper braces were given. For this, a total of 9 different frames were considered. The original moment resisting frame, single zipper braced frame, and double zipper braced frame were denoted as OMRF, OMRF-ZB-S, and OMRF-ZB-D, respectively. Each models had different numbers of story (i.e., 3, 6, 8). The performance of the structures were evaluated by using the nonlinear pushover analysis and time history analysis. In the time history analysis, three different earthquakes records were utilized. The results were given in terms of capacity curve, displacement, roof displacement time history, and drift ratio.

4.2 Capacity curves

The capacity curves based on the pushover analysis were evaluated for different frame types. Figures 4.1 to 4.3 show the comparison of the capacity curves of the 3, 6, and 8 story original, single and double zipper braces frames. These figures were resulting from the static analysis of the original steel frames and modified frames. Many improvements were observed in the seismic performance of the buildings when the suitable modification was used**.** The figures indicated that the frames with single and double zipper braces had higher value of capacity in comparison to the original frames, irrespective of the number of story. For example, in Figure 4.1, the maximum base shear force was about 436 kN in the case of the three story original frame while that of the modified frames with single and double zipper braces were nearly 490 kN and 717 kN, respectively. This implied about 1.12 and 1.64 times higher lateral load carrying capacity for the modified cases in comparison to the original frame.

In Figure 4.2, the maximum base shear force for the six story original building was approximately 465 kN while that of the modified frames with single and double zipper braces were nearly 508 kN and 817 kN, respectively. This yielded about 1.09 and 1.75 times greater lateral load carrying capacity for the single and double zipper braced frames in comparison to the original one, respectively. Similarly, as seen in Figure 4.3 the maximum base shear force for the eight story original building was about 529 kN whereas that of the modified frames with single and double zipper braces were nearly 574 kN and 838 kN, respectively. This gave about 1.08 and 1.45 times higher load carrying capacity for the modified cases as compared with the original frame.

Thus, in this study, for all cases, the comparison between the single and double zipper braced frames showed that the latter provided higher lateral load carrying capacity than the former.

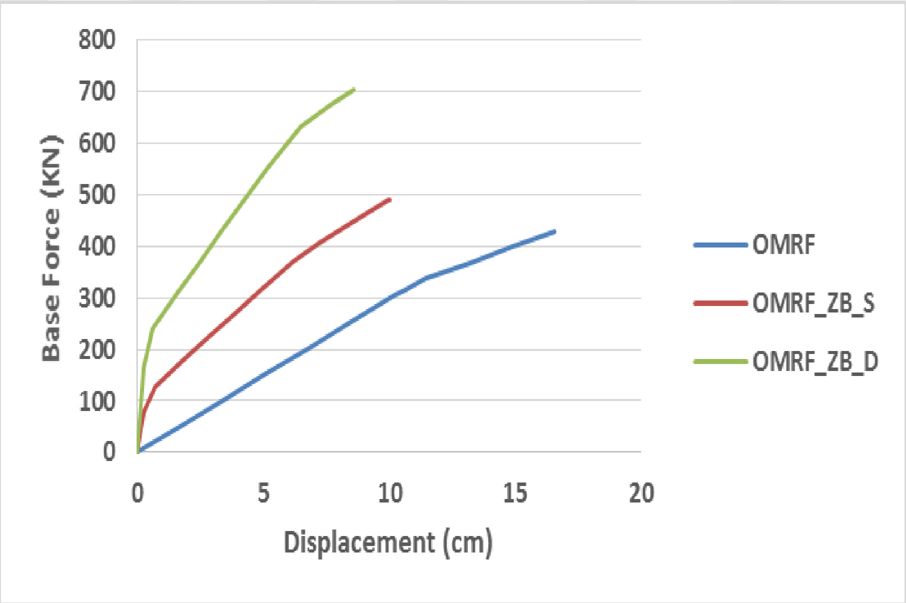


Figure 4.1 Capacity curves for 3-story ordinary moment resisting frame and that with single and double zipper braces

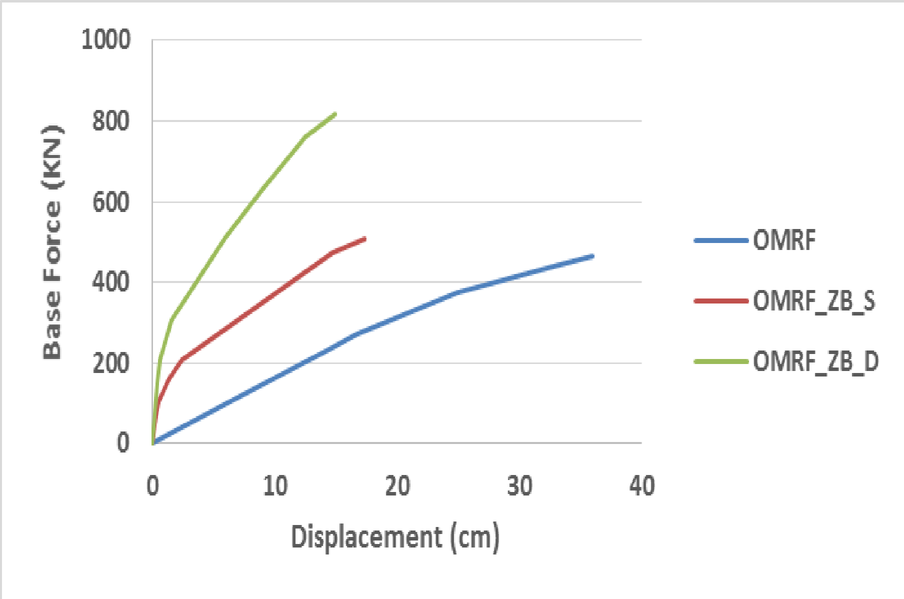


Figure 4.2 Capacity curves for 6-story ordinary moment resisting frame and that with single and double zipper braces

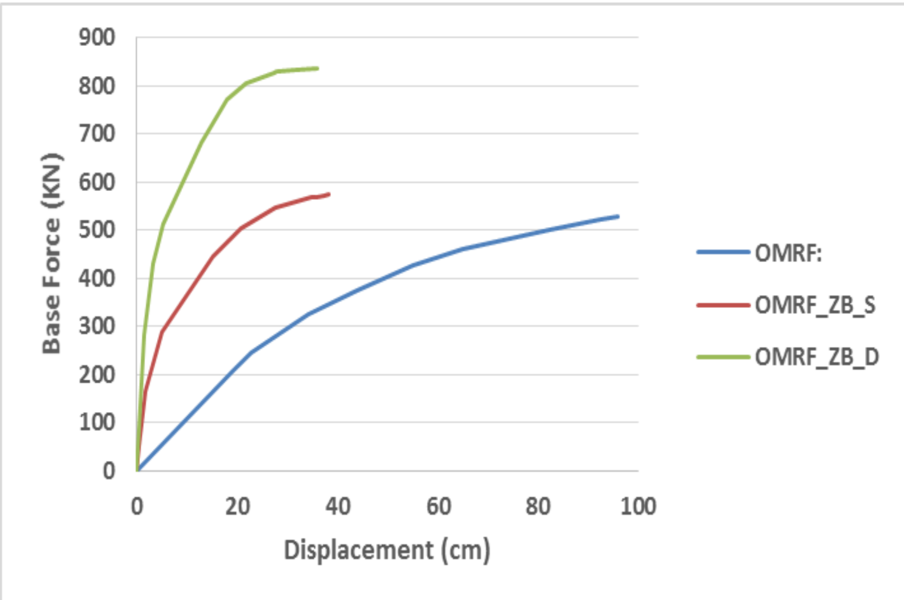
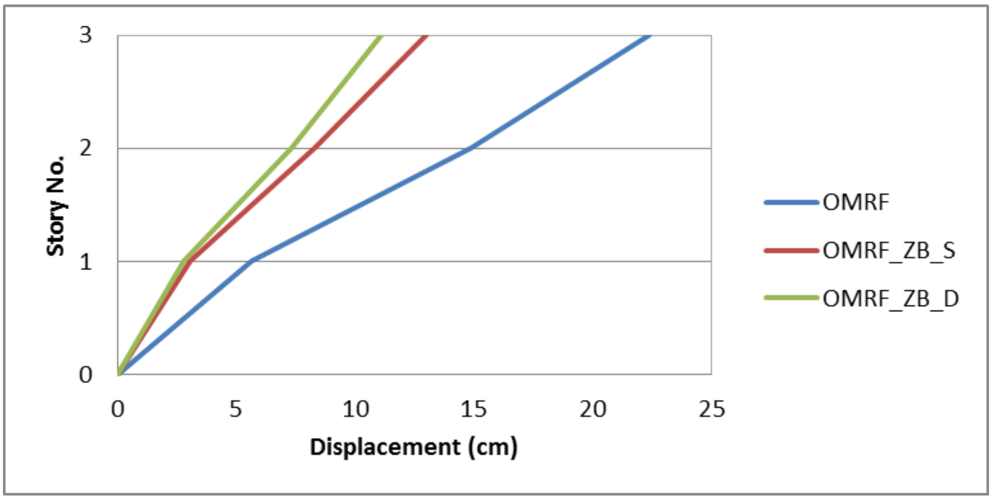


Figure 4.3 Capacity curves for 8-story ordinary moment resisting frame and that with single and double zipper braces

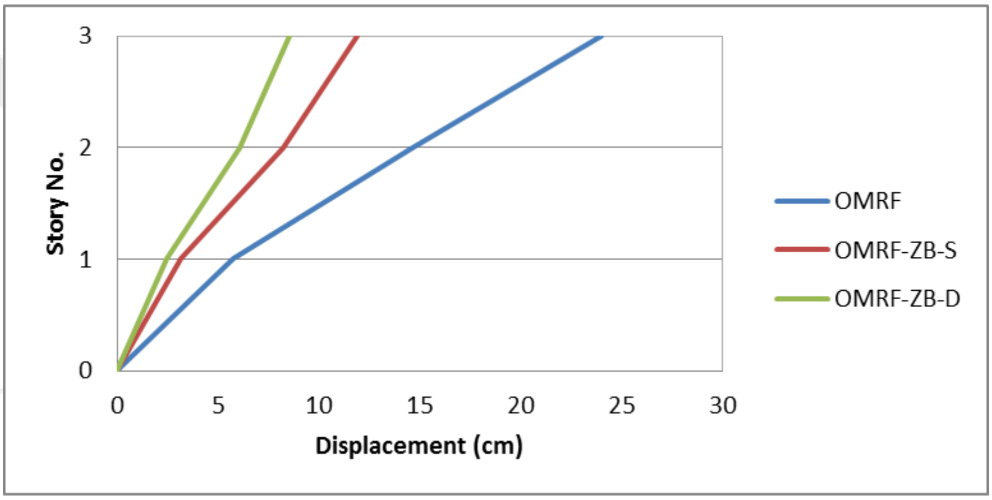
4.3 Variation of storey displacement

From the results obtained from the time history analysis, the variation of the story displacements over the story height are given in Figures 4.4 to 4.6 for 3, 6 and 8 story original and modified frames, respectively. As seen from each figure, the structures were subjected to three earthquake ground motions. The analysis results indicated that the use of single and double zipper brace system reduced significantly the value of the maximum story displacement as compared to original frames, especially in the double zipper braced frames. The maximum story displacement was also affected by the frame type and number of stories. For example, in the case of three story frames with single and double zipper brace systems, the maximum story displacement was smaller than other frames (six and eight story frames). By increasing the number of story the maximum story displacements had a tendency to increase.

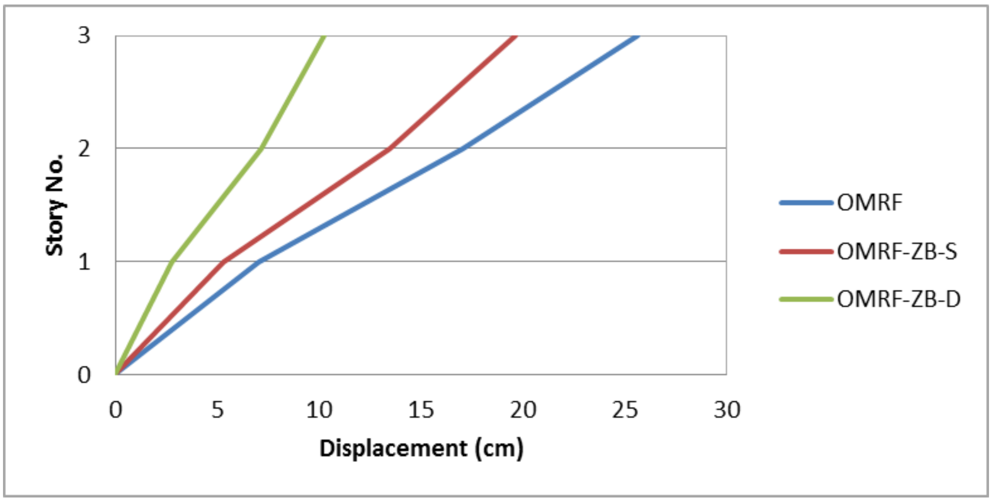
Figures 4.4 to 4.6 also exhibited that the application of the single and double zipper braces for strengthening the existing moment resisting frame were very influential for all earthquakes. For example, as seen in Figure 4.4 under Hector Mine earthquake, the maximum displacement of the three story original frame was obtained as 17 cm whereas the maximum displacement of the three story single and double zipper braced frames was lower than the original frame system such that the values were 10 cm and 9 cm, respectively. In the case of the six story building under Hector Mine earthquake (Figure 4.5), the maximum displacement of the original frame was obtained as 36 cm while the maximum displacement of the single and double zipper braced frames were 17cm and 15 cm, respectively. Moreover, for the eight story building under Hector Mine earthquake (Figure 4.6), the maximum displacement of the original frame was obtained as 96 cm whereas the maximum displacement of the single and double zipper braced frames were evaluated as 38 cm and 36 cm, respectively.



a)

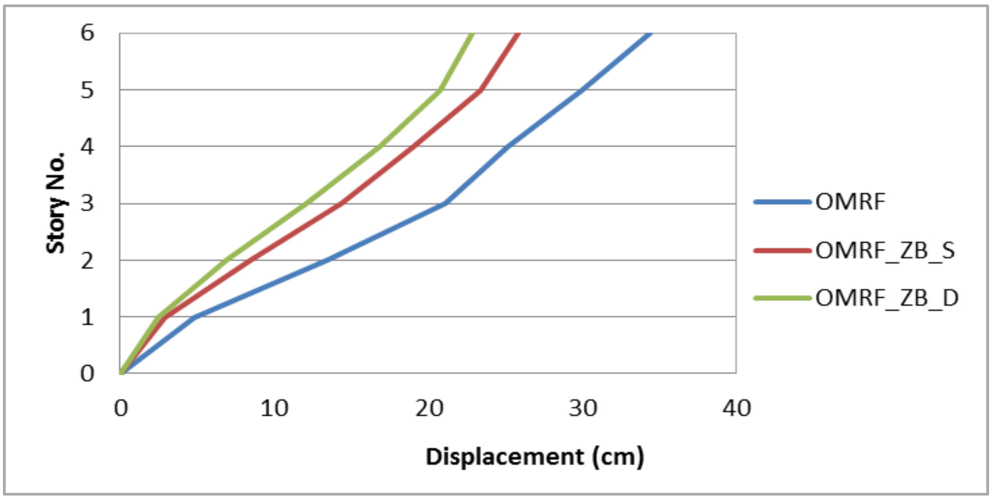


b)

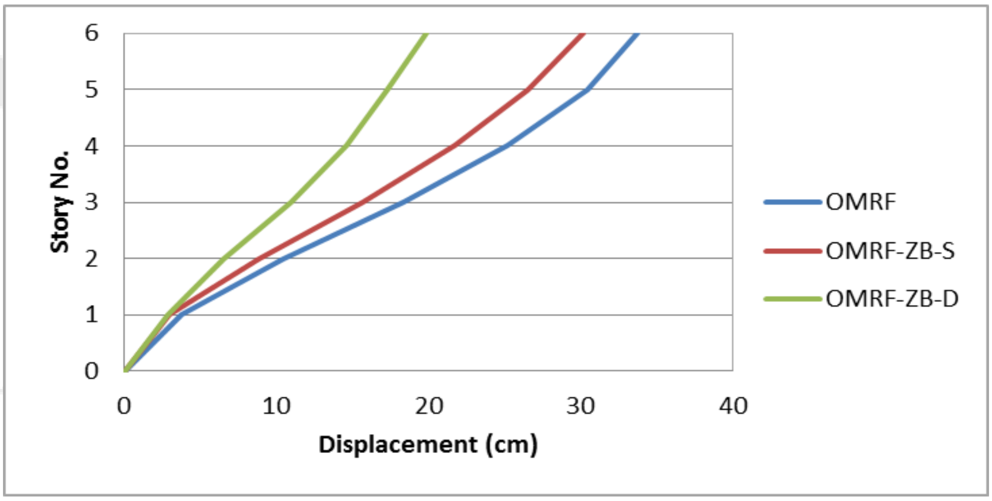


c)

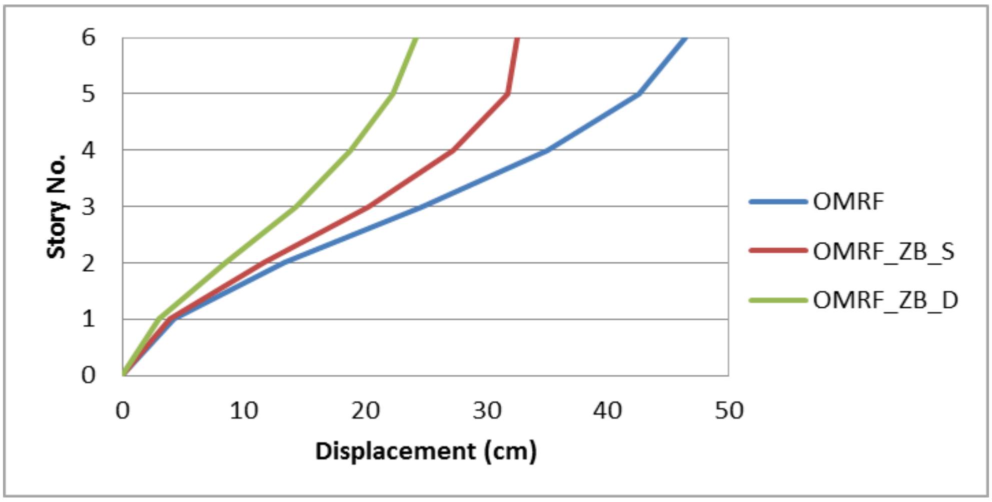
Figure 4.4 Variation of story displacement of 3-story original and retrofitted frames under a) Hector Mine, b) Northridge, and c) Chi-Chi earthquakes



a)

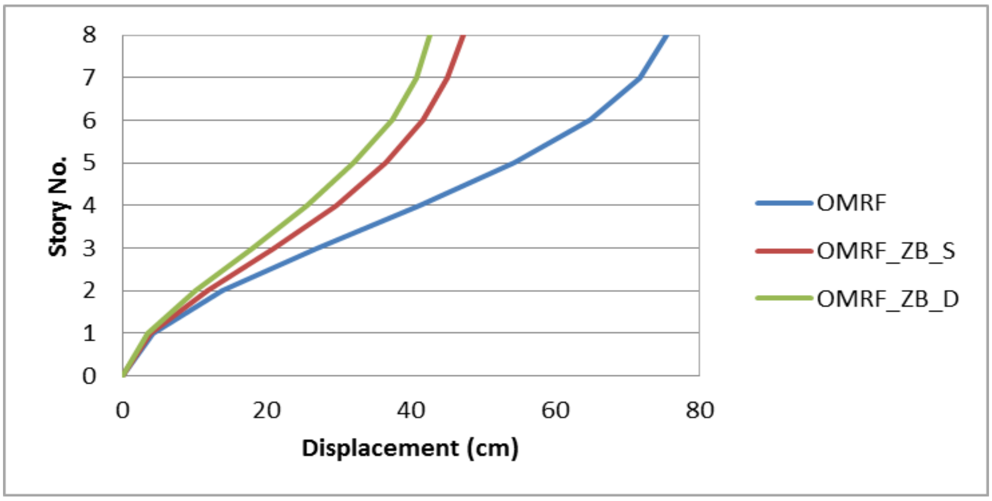


b)

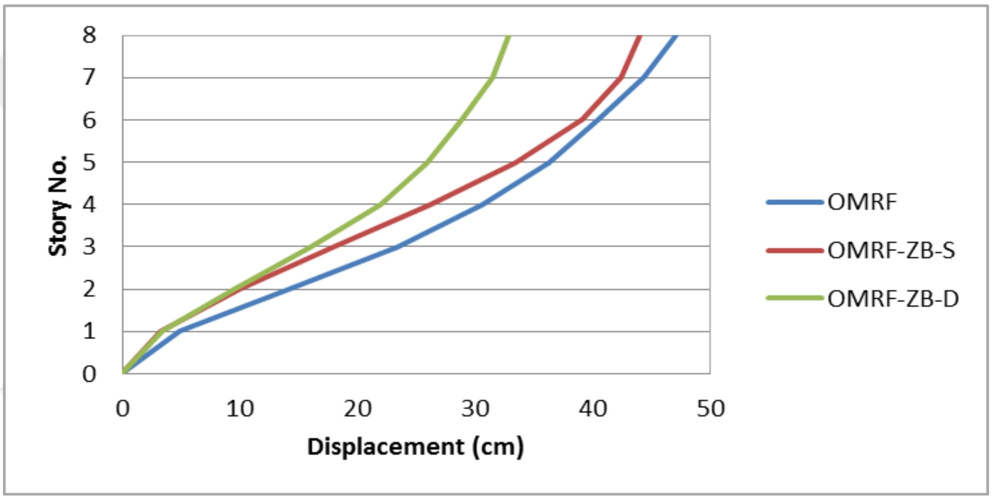


c)

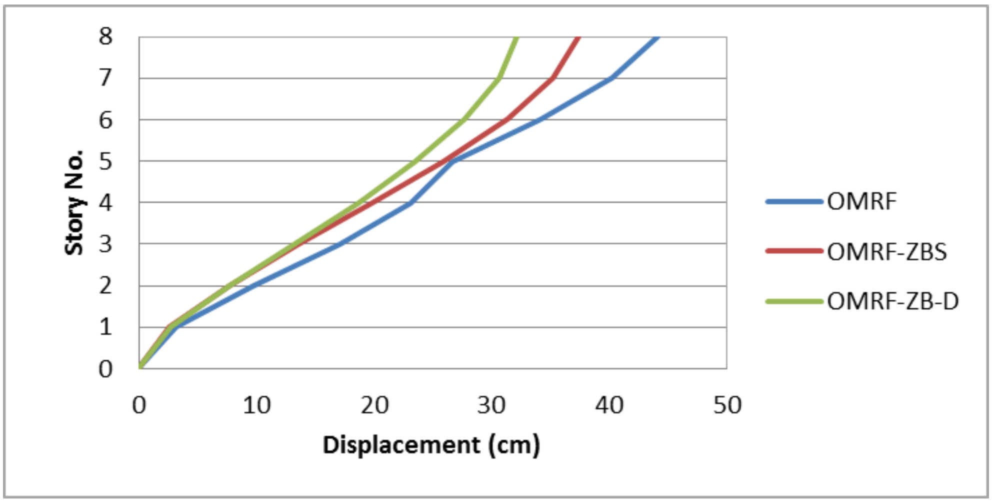
Figure 4.5 Variation of story displacement of 6-story original and retrofitted frames under a) Hector Mine, b) Northridge, and c) Chi-Chi earthquakes



a)



b)

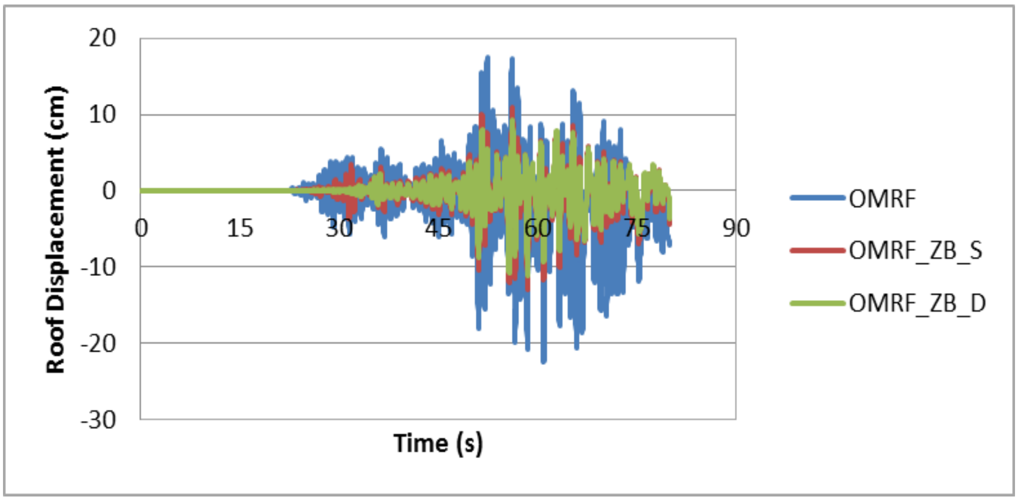


c)

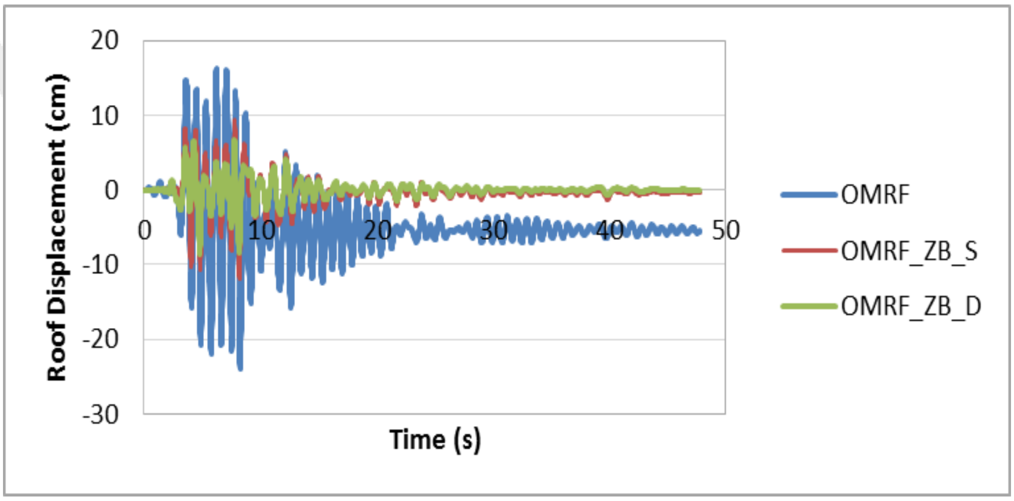
Figure 4.6 Variation of story displacement of 8-story original and retrofitted frames under a) Hector Mine, b) Northridge, and c) Chi-Chi earthquakes

4.4 Variation of roof displacement-time history

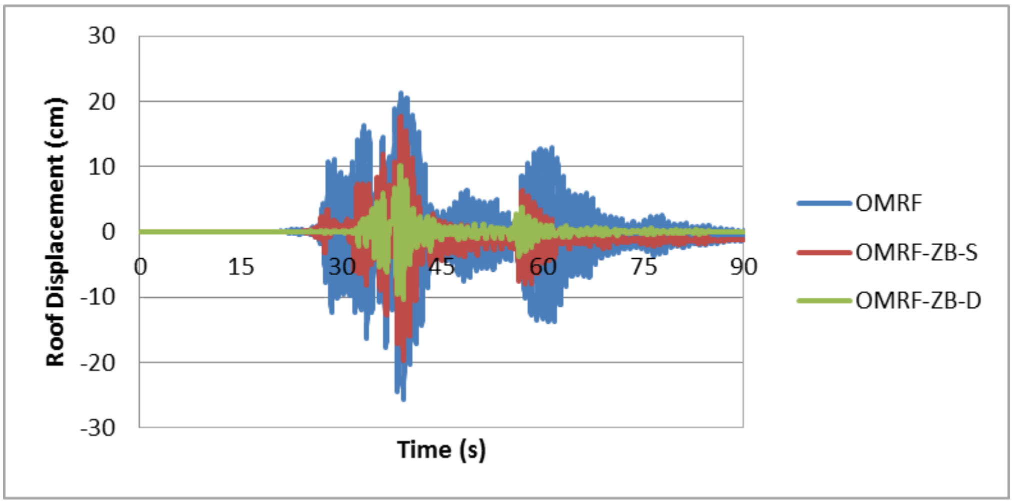
The results of the roof displacement versus time are presented in Figures 4.7 to 4.9 for the three, six, and eight story original frames and those retrofitted with single and double zipper braces under three different ground motions (Hector Mine, Northridge, and Chi-Chi earthquakes), respectively. The maximum roof displacement was also affected by the frame type and the number of story. Moreover, it was observed that the use of both the single and double zipper braces significantly reduced the values of roof displacements compared with the original frame for all earthquakes. For example, as seen in Figure 4.7, for the three story building under Hector Mine earthquake, the maximum roof displacement of the original frame was about 22 cm while the maximum roof displacement of the single and double zipper braced frames were achieved as approximately 13 cm and 11 cm, respectively. Moreover, for the six story building under Hector Mine earthquake (Figure 4.8), the maximum roof displacement of original frame was about 34 cm while the maximum roof displacement for the single and double zipper braced frames were obtained as 26 cm and 23 cm, respectively. Furthermore, for the eight story building under Hector Mine earthquake (Figure 4.9), the maximum roof displacement of original frame was about 75 cm while the maximum roof displacement of the single and double zipper braced frames was evaluated as nearly 47 cm and 43 cm, respectively. Therefore, it was pointed out that the addition of single and double zipper brace systems decreased considerably the roof displacement in the frames. The results also showed that the use of double zipper braced frames was better than that of the single zipper braced frame. When the effects of the other two earthquakes (Northridge and Chi-Chi earthquakes). For example, under Northridge earthquake, the maximum roof displacement results of the three-story buildings with the single and double zipper braced cases were 12 cm and 8.5 cm, respectively. For the six story buildings with the single and double zipper braced cases were 30 cm and 20 cm, respectively. In addition, the eight story buildings with the single and double zipper braced cases were 44 cm and 33 cm, respectively. For Chi-Chi earthquake, these values ranged from 10 to 20 cm, 24 to 40 cm, and 32 to 38 cm for the three, six, and eight story structures.



a)

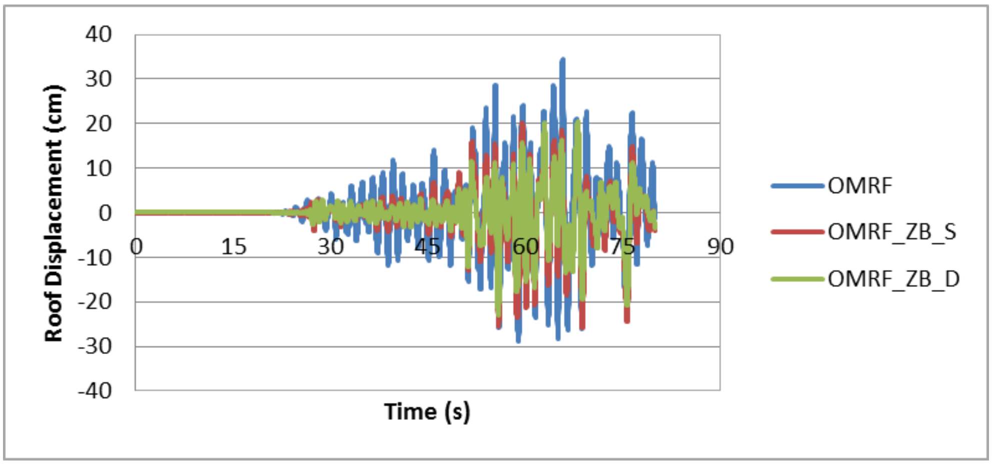


b)

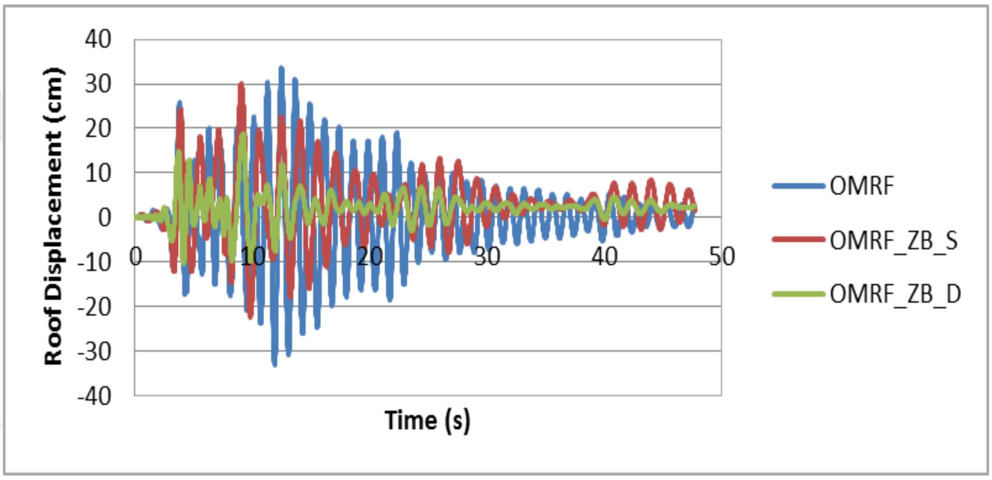


c)

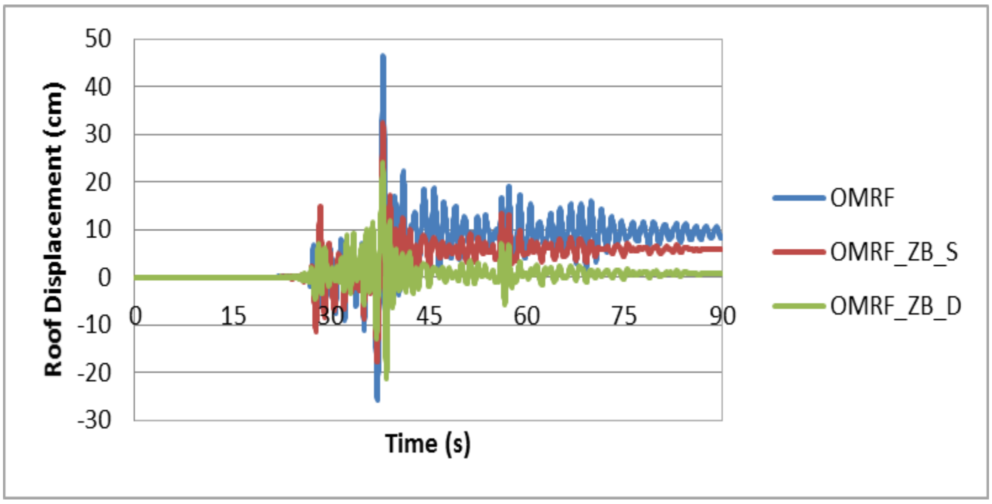
Figure 4.7 Roof displacement vs. time for the 3-story original frame and retrofitted frames: a) Hector Mine, b) Northridge, and c) Chi-Chi earthquakes



a)

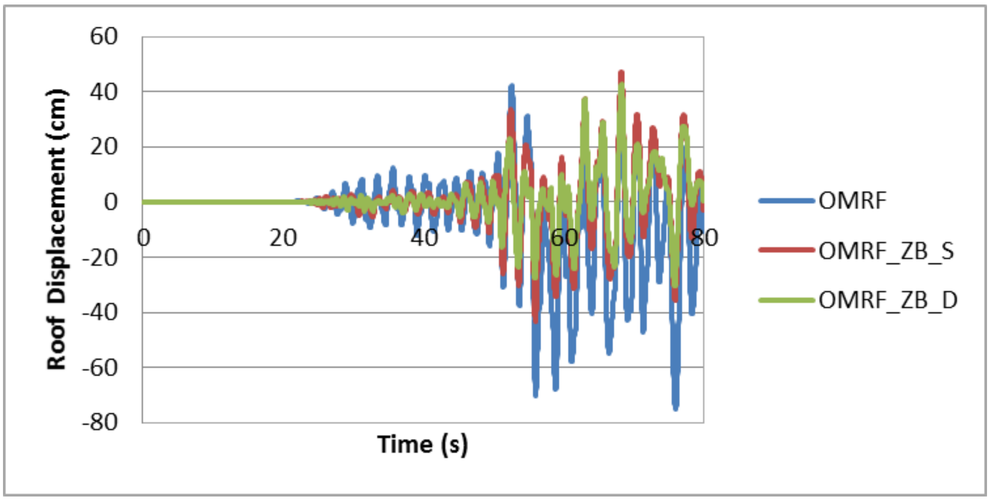


b)

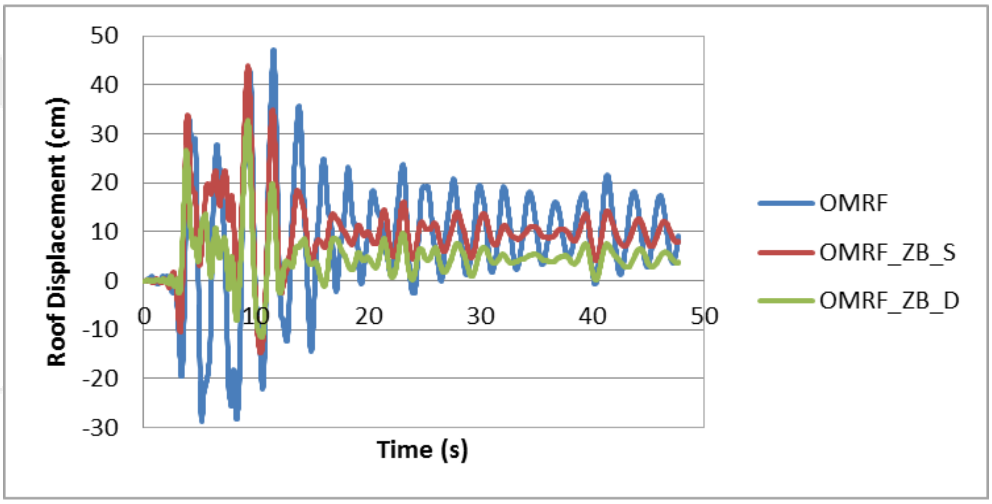


c)

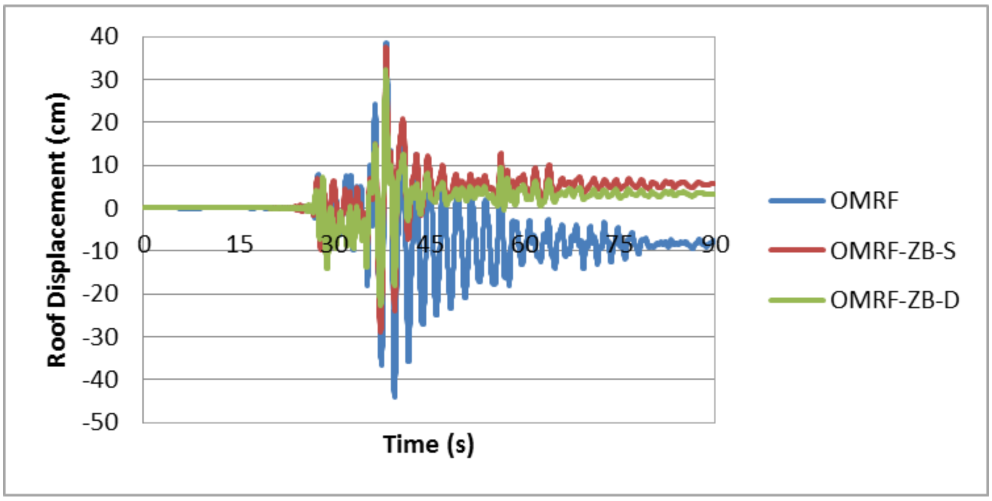
Figure 4.8 Roof displacement vs. time for the 6-story original frame and retrofitted frames: a) Hector Mine, b) Northridge, and c) Chi-Chi earthquakes



a)



b)

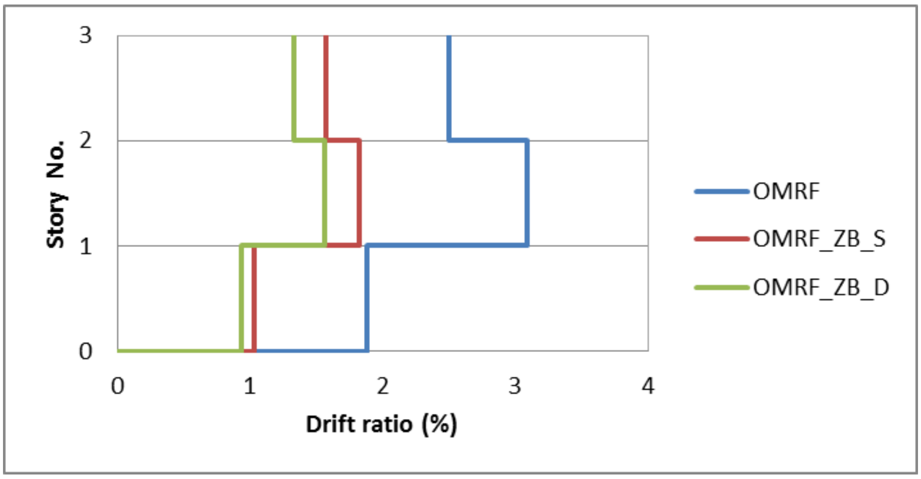


c)

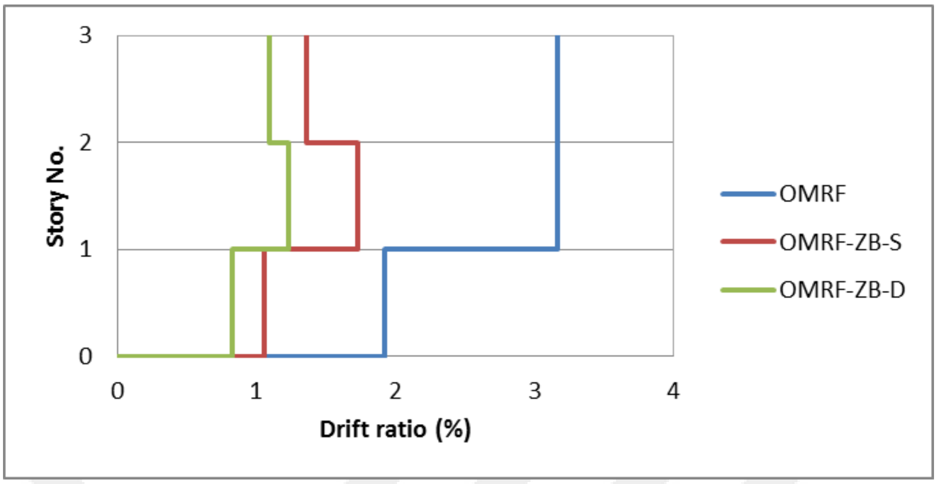
Figure 4.9 Roof displacement vs. time for the 8-story original frame and retrofitted frames: a) Hector Mine, b) Northridge, and c) Chi-Chi earthquakes

4.5 Interstory drift ratio

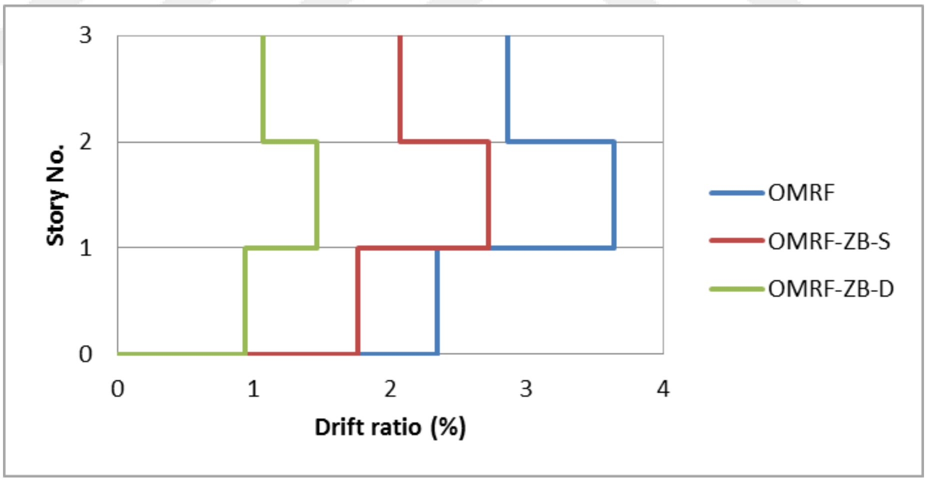
The results of the interstory drift ratio of the original, single and double zipper braced frames subjected different earthquakes are shown in Figures 4.10 - 4.12. The analysis results indicated that the utilization of the single and double zipper brace system diminished significantly the value of the interstory drift ratio as compared to original frame, especially for the double zipper brace frame. For example, as seen in Figure 4.10, the three story building under Northridge earthquake, the maximum interstory drift ratio of the original frame was obtained as 3.16% whereas the maximum interstory drift ratio of the single and double zipper braced frames were 1.36% and 1.09%, respectively. In the case of the six story building under the Chi-Chi earthquake (Figures 4.11), the maximum interstory drift ratio of the original frame was obtained as 1.52% while the maximum interstory drift ratio of the single and double zipper braced frames were computed as 0.79% and 0.76%, respectively. For the eight story building under Hector Mine earthquake (Figures 4.12), the maximum interstory drift ratio of the original, single, and double zipper braced frames achieved as 1.90%, 0.87%, and 0.65%, respectively.



a)

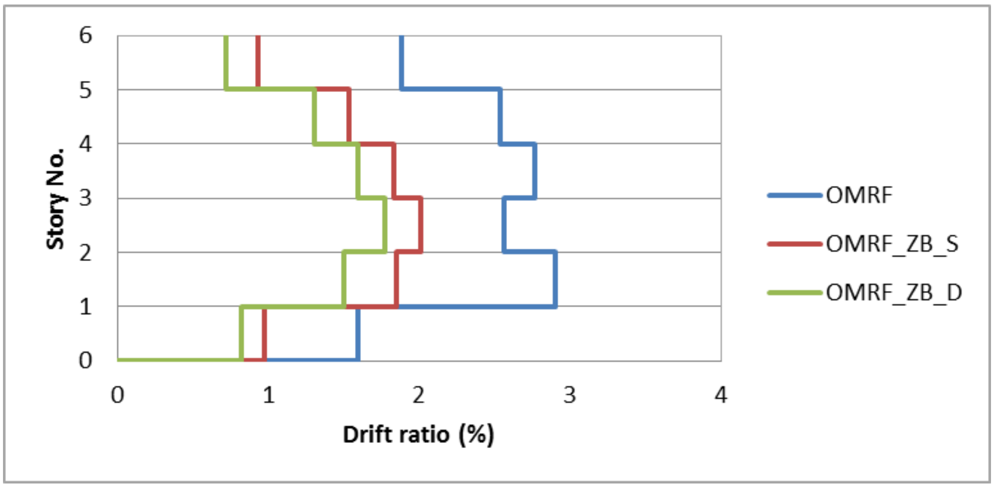


b)

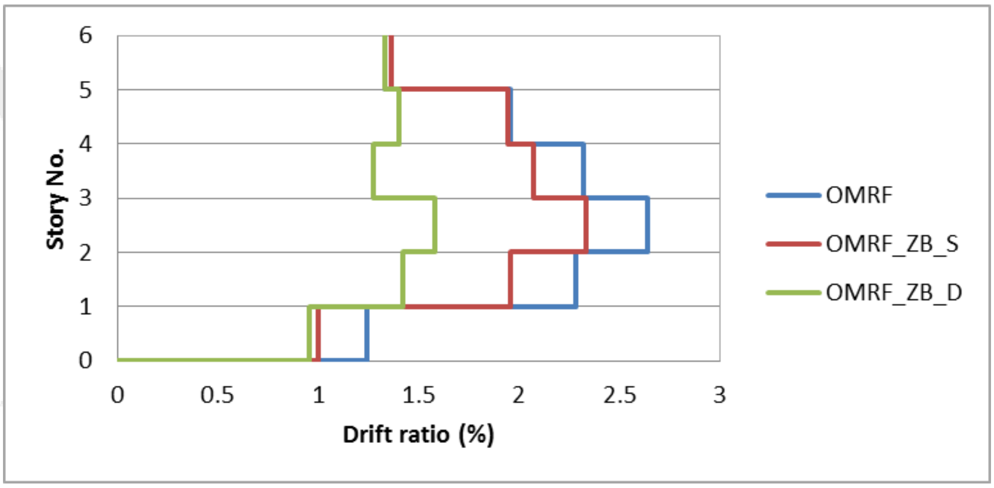


c)

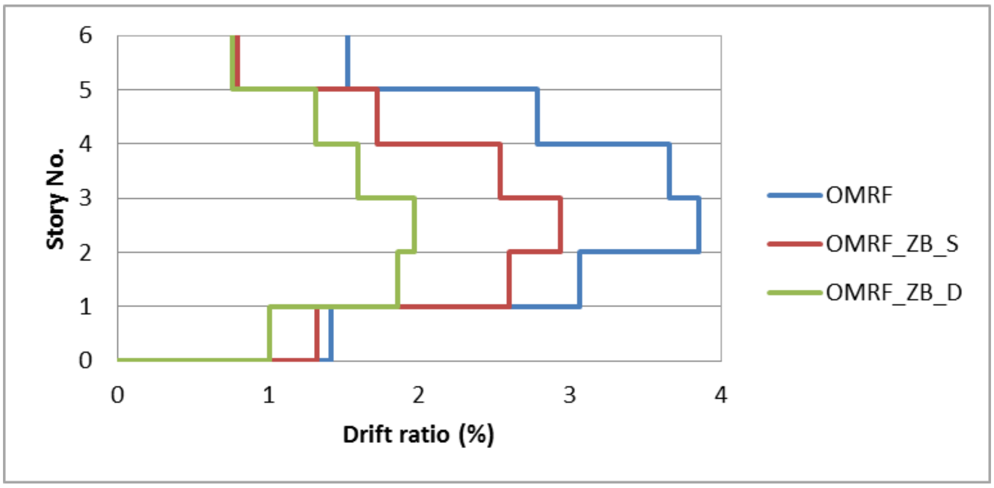
Figure 4.10 Interstory drift ratio for the 3-story original and retrofitted frames a) Hector Mine, b) Northridge, and c) Chi-Chi earthquakes



a)

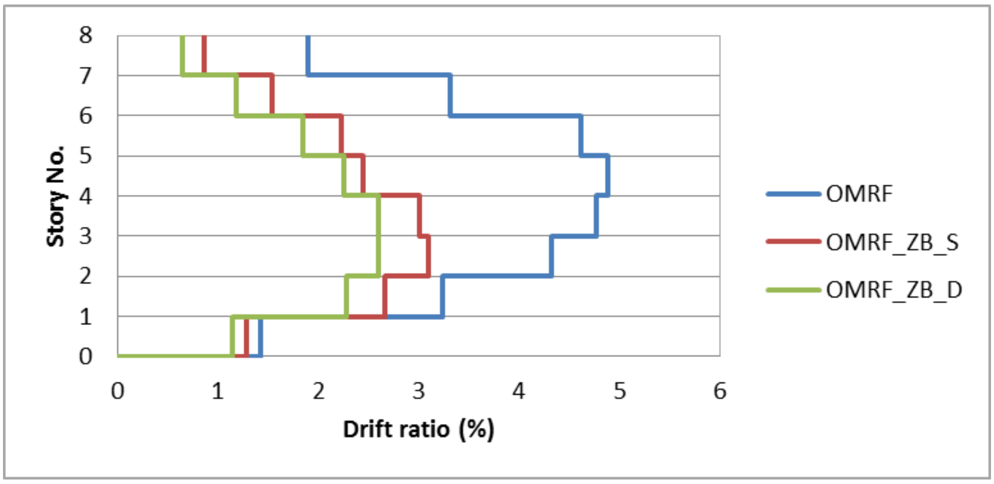


b)

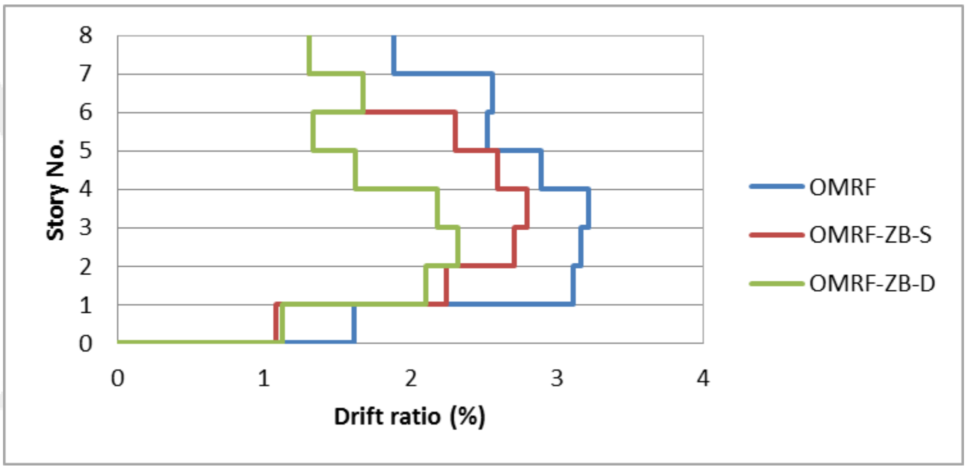


c)

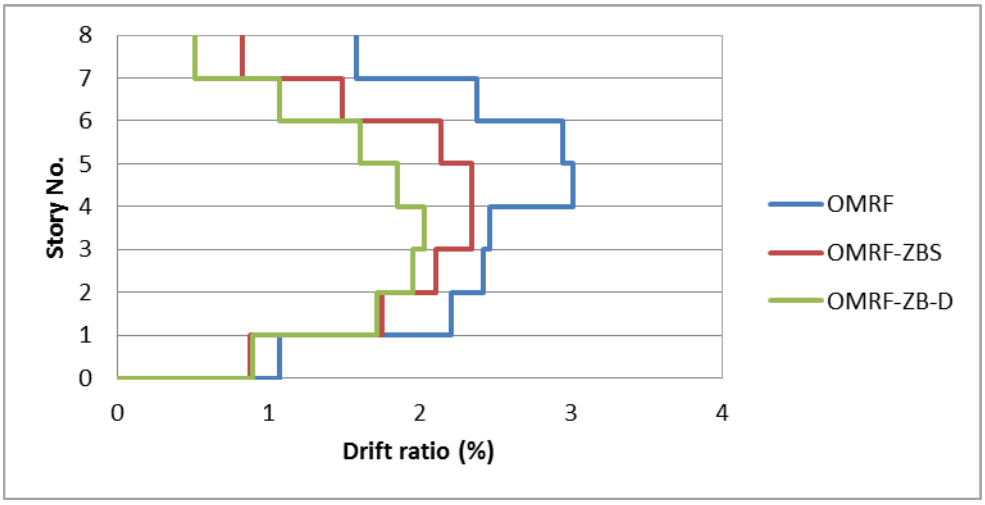
Figure2.11 Interstory drift ratio for the 6-story original and retrofitted frames a) Hector Mine, b) Northridge, and c) Chi-Chi earthquakes



a)



b)



c)

Figure 4.12 Interstory drift ratio for the 8-story original and retrofitted frames a) Hector Mine, b) Northridge, and c) Chi-Chi earthquakes

CHAPTER 5

CONCLUSIONS

Based on the nonlinear static and time history analysis conducted on the ordinary moment resisting frames and those retrofitted by the single and double zipper braces, the following conclusions were drawn:

- The analysis of the results exhibited that the frames with single and double zipper had greater load carrying capacity than the original frame, irrespective of the story number.

- According to the capacity curves, for 3-story frame, the use of the double zipper brace in retrofitting resulted in 1.64 times greater capacity than single one, which was 1.12 times higher lateral load carrying capacity in comparison to the original frame. In the 6-story frame, the single and double zipper had 1.09 and 1.75 times higher lateral load carrying capacity than the original frames, respectively. In the case of 8-story frame, these values were evaluated as 1.08 and 1.45 for the single and double zipper braced frames, respectively.

- It was observed that the use of single and double zipper brace system reduced significantly the value of the maximum story displacement. This was more pronounced for the latter. The maximum story displacement was also affected by the frame type and the number of story.

- The results of the roof displacement time history analysis showed that the roof displacement was decreased in the single and especially double zipper braced frames as compared with the original frames. It was also pointed out that the effectiveness of using double zipper bracing in retrofitting on the maximum roof displacement value became more with increasing the story number of the structure.

- According to the analysis of the results, it was observed that the retrofitted structures had relatively lower interstory drift ratio in comparison to the existing ones, depending mainly on the zipper brace distribution and ground motion used.

REFERENCES

Bruneau, M., Anagnostopoulou, M., MacRae, G., Clifton, C., & Fussell, A. (2010). Preliminary report on steel building damage from the darfield earthquake of septermber 4, 2010. Bulletin of the New Zealand Society for earthquake engineering, 43(4), 351-359.

Bruneau M., Uang, C., and Whittaker, A., (1998).Ductile Design of Steel Structures, McGraw Hill,

Chen, C. H., Lai, J. W. and Mahin, S. (2008). Seismic Performance Assessment of Concentrically Braced Steel Frame Buildings. *The 14th World Conference on Earthquake Engineering,* Beijing, China. Email: chchen@berkeley.edu.

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

Chimeh, M. N., and Homami, P., (2012) "Efficiency of bracing systems for seismic rehabilitation of steel structures". (Tarbiat Moallem) University, Tehran, Iran.

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

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

Hines, E. M. (2009) "Eccentric Braced Frame Design for Moderate Seismic Regions". *Structural Engineers*, pp. 784. Email: emhines@lemessurier.com.

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. Choi, H. (2005), Response modification factors of chevron-braced frames. *Journal of Engineering, Structure*, 27:285–300.

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. Email: ceholee@snu.ac.kr.

Krawlinkler, H., and Seneviranta, G.D.P.K. (1998). "Pros and Cons of a Pushover Analysis of Seismic Performance Evaluation," *Engineering Structures* **20(4)**: 452-464.

Koboevic, S., Rozon, J**.,** and Tremblay, R. (2012). The impact of seismic response of outer beams, braces and columns on global seismic behaviour of chevron-type eccentrically braced frames. WCEE2012_4378.

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, (pp.89-96). Taipei (Taiwan) National Center for Research on Earthquake.

Lignos, D., Ricles, J., Love, J., Okazaki, T., & Midorikawa, M. (2011). Effects of the 2011 Tohoku Japan Earthquake on Steel Structures. *Earthquake Engineering Research*, Insitute.

MacRae, G. and Clifton, C., (2013) "Low Damage Design of Steel Structures". Steel Structures New Zealand.

Mwafy, A.M., Elnashai, A.S. (2001). "Static pushover versus dynamic collapse analysis of RC buildings," *Engineering Structures*; **23 (5)**: 407-424.

Naeimi, S., Shahmari, A., and Kalehsar, H. E. (2012) "Study of the Behavior of Zipper Braced Frames". *The 15th World Conference on Earthquake Engineering, Education*, Iran.

Nouri, G. R., Imani Kalesar, H., and Ameli, Z. (2009). "The applicability of the zipper strut to seismic rehabilitation of steel structure". World Academy of Science, Engineering and Technology **Vol: 3**-10-22.

Özçelik, R., (2011) "Seismic Upgrading of Reinforced Concrete Frames With Structural Steel Elements". PhD thesis, Middle East Technical University.

Popov, E. P., and Engelhardt, M. D. (1988). "Seismic Eccentrically Braced Frames". *Journal of Constructional Steel Research*, 10, 321-354.

Pourbabaa, M., Karimia, M. R. B., Zareib, B., and Azarc, B. B., (2013) "Behavior of Zipper Braced Frame (ZBF) compared with other Concentrically Braced Frame (CBF)". *International Journal of Current Engineering and Technology*, **Vol: 3**, No.4, ISSN 2277 – 4106.

Rai, D.C., Goel, S.C., (2003) "Seismic evaluation and upgrading of chevron braced frames". *Journal of Constructional Steel Research* 59 (2003) 971–994.

Razavi, M. and Sheidaii M.R. (2012). Seismic performance of cable zipper-braced frames, *journal of constructional research* 74: 49-57.

Sabelli, R., Roeder, C. W., Hajjar, J. F., (2013) "Seismic Design of Steel Special Concentrically Braced Frame Systems". National Institute of Standards and Technology, NEHRP Seismic Design Technical Brief No. 8. NIST GCR 13-917-24.

Sarand, N. I., Jalali, A., Hosseinzadeh, Y., (2013) " Seismic Behavior of Zipper Braced Frames; A Review". *Journal of Basic and Applied Scientific Research.* **3(5)**, 415-419, 2013, ISSN 2090-4304. Email: irani9@ms.tabrizu.ac.ir.

Shayanfar, M., Rezaeian, A., and Taherkhani, S. (2008). "Assessment of the seismic behavior of eccentrically braced frame with double vertical link (DV-EBF)". *The*

14th World Conference on Earthquake Engineering, Beijing, China. Email: ceholee@snu.ac.kr.

Sheng, N., Yam, M. C. H., and Lu, V.P., (2002) “Analytical Investigation and the Design of the Compressive Strength of Steel Gusset Plate Connections*”, Journal of Constructional Steel Research,* **Vol: 58**, p 1473-1493.

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

Tirca L., Tremblay R., (2004). "Influence of building height and ground motion type on the seismic behaviour of zipper concentrically braced steel frames". *13th World Conference on Earthquake Engineering.* 2004. Paper No. 2894.

Tremblay, R., (2002). "Inelastic Seismic Response of Steel Bracing Members". *Journal of Constructional Steel Research*, 58, pp. 665-701.

Tremblay, R., Bruneau, M., and Wilson, J. (1996). Performance of Steel Bridges during the 1995 Hyogo-Ken Nanbu (Kobe, Japan) Earthquake. Canadian *Journal of Civil Engineering*, **23(3)**, 678-713.

Tremblay, R., Tirca L. (2003). "Behaviour and design of multi-story zipper concentrically braced steel frames for the mitigation of soft-story response". In: Proceedings of the conference on behaviour of steel structures in seismic areas. 2003. p. 471-7.

Yang C.S., Leon R.T., DesRoches R. (2008a). Pushover response of a braced frame with suspended zipper struts, *Journal of Structural Engineering*, **134:** 1619-1626.

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

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

APPENDIX

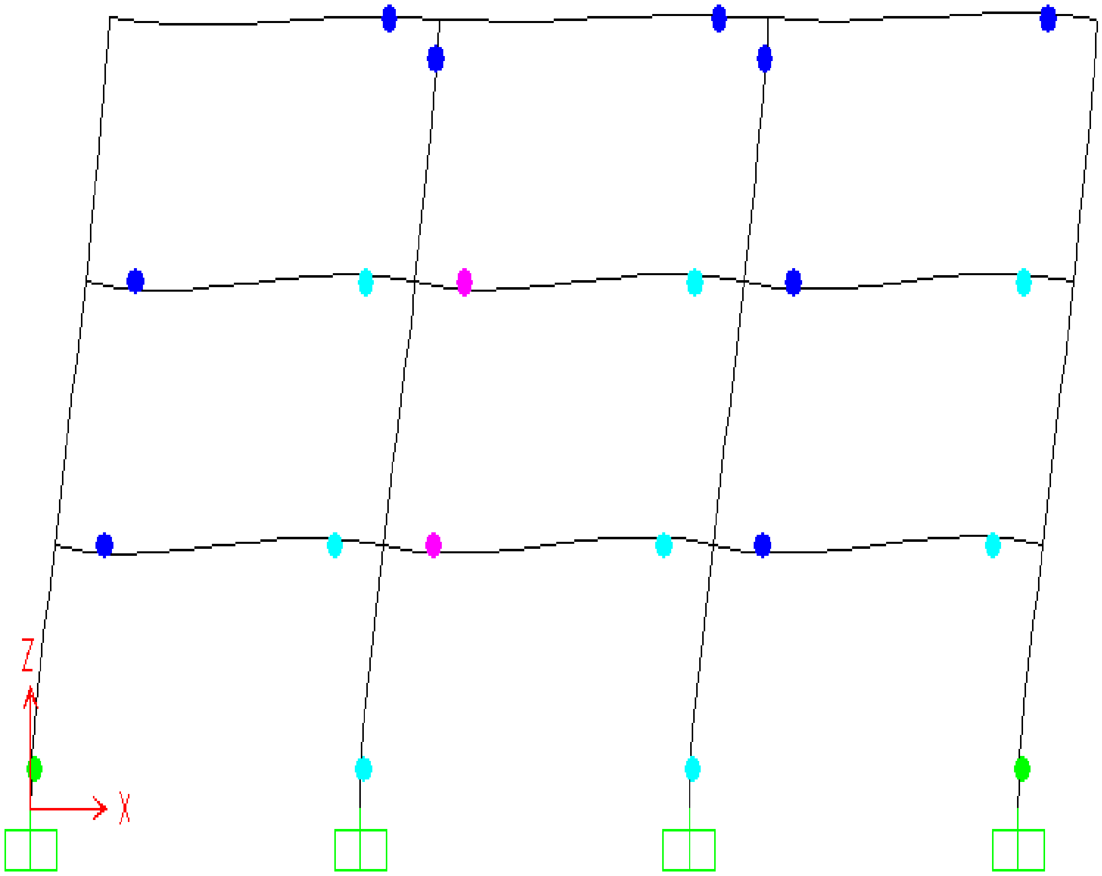


Figure A1 The hinge formation for three story original frame under Hector Mine earthquake

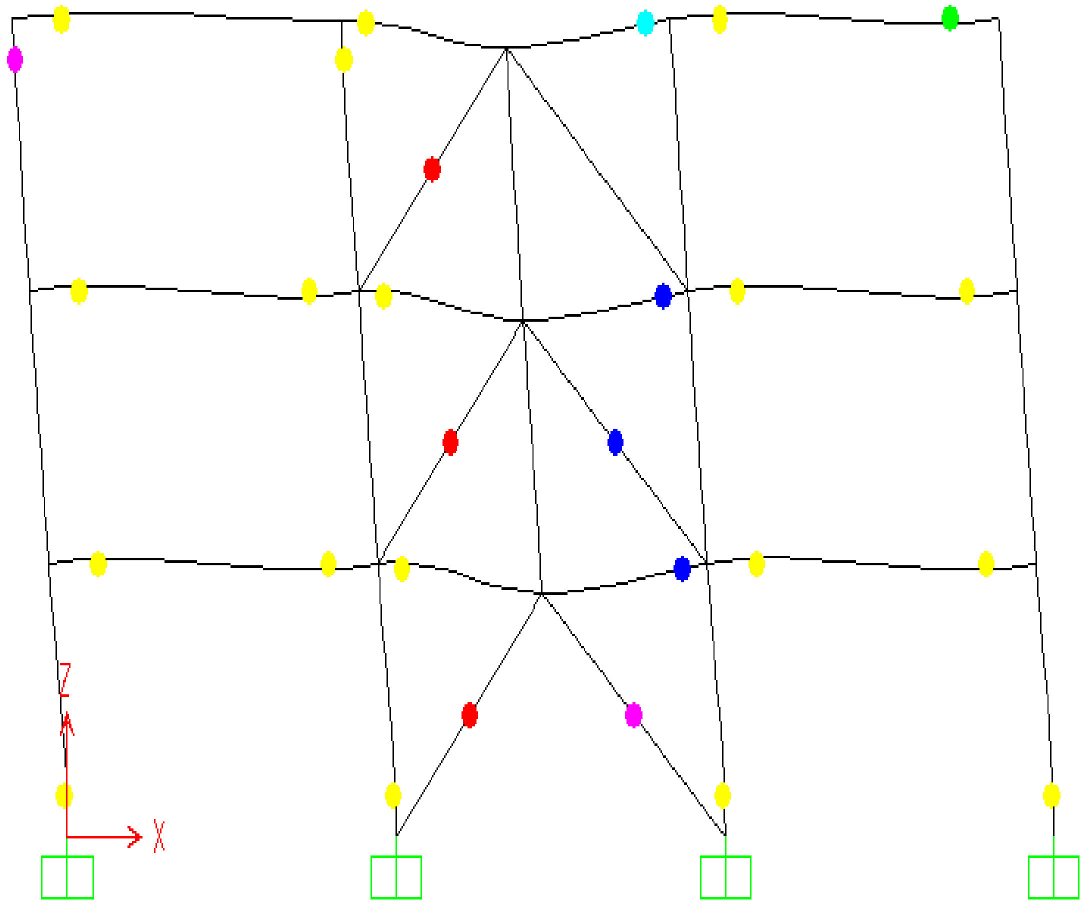


Figure A2 The hinge formation for three story single zipper braced frame under Hector Mine earthquake

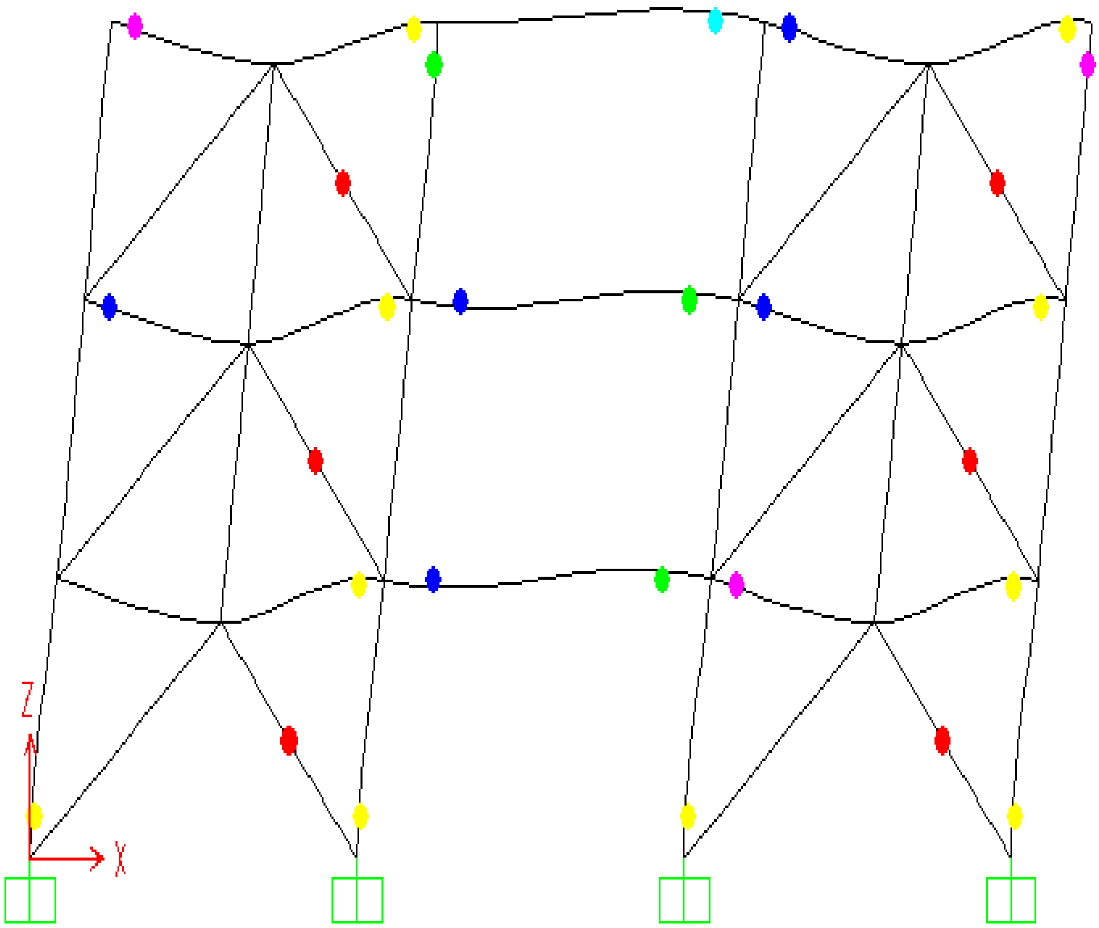


Figure A3 The hinge formation for three story double zipper braced frame under Hector Mine earthquake

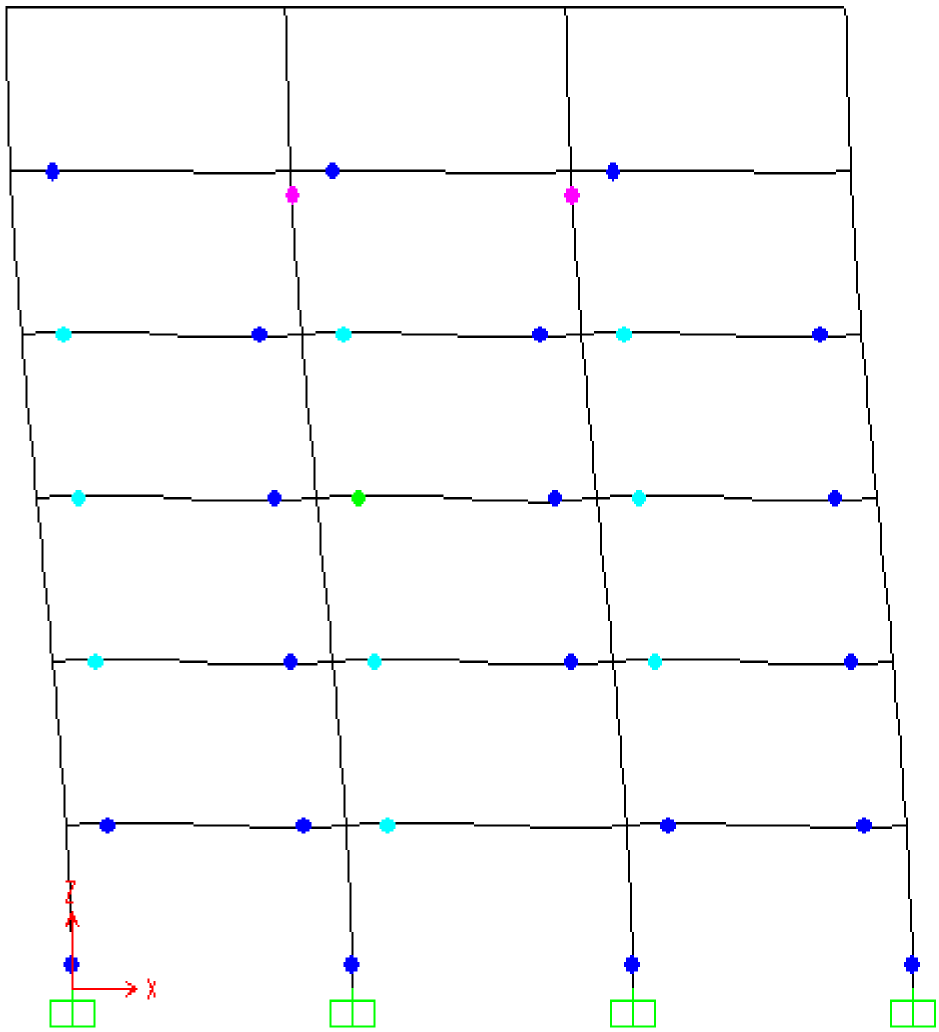


Figure A4 The hinge formation for six story original frame under Chi-Chi earthquake

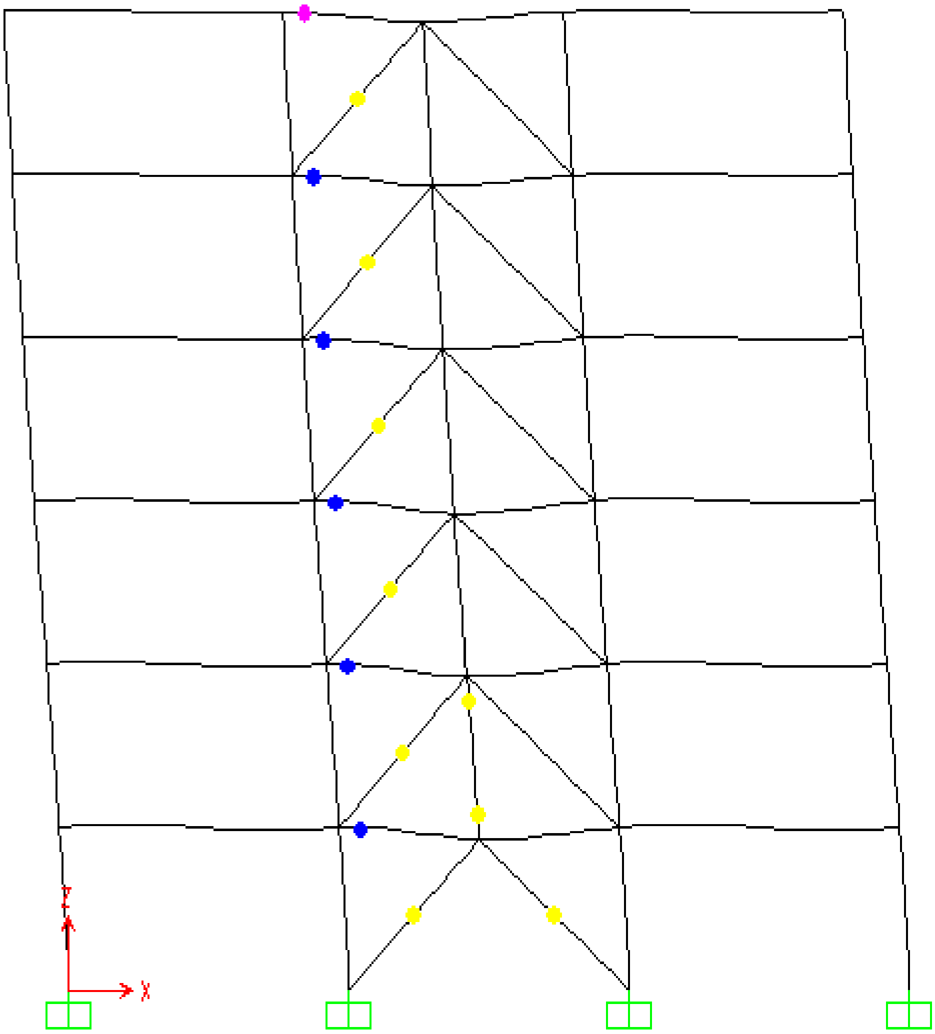


Figure A5 The hinge formation for six story single zipper braced frame under Chi-Chi earthquake

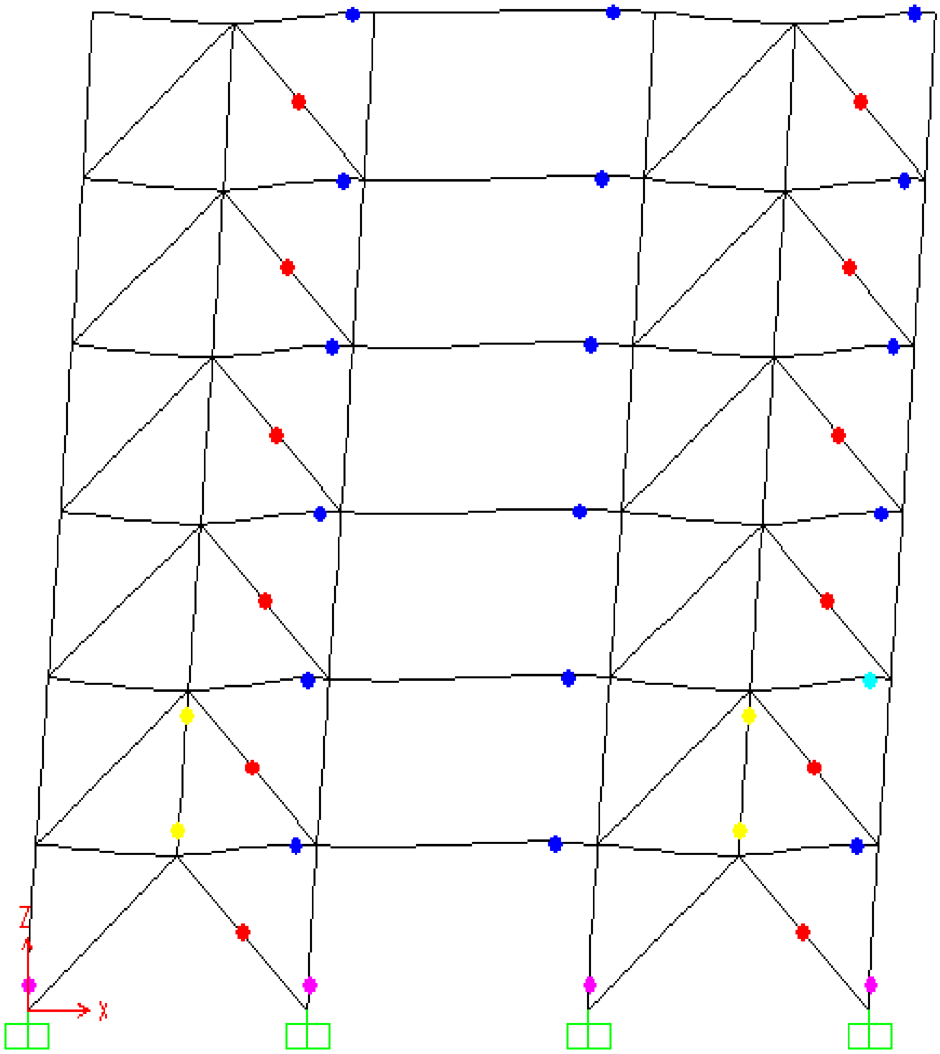


Figure A6 The hinge formation for six story double zipper braced frame under Chi-Chi earthquake

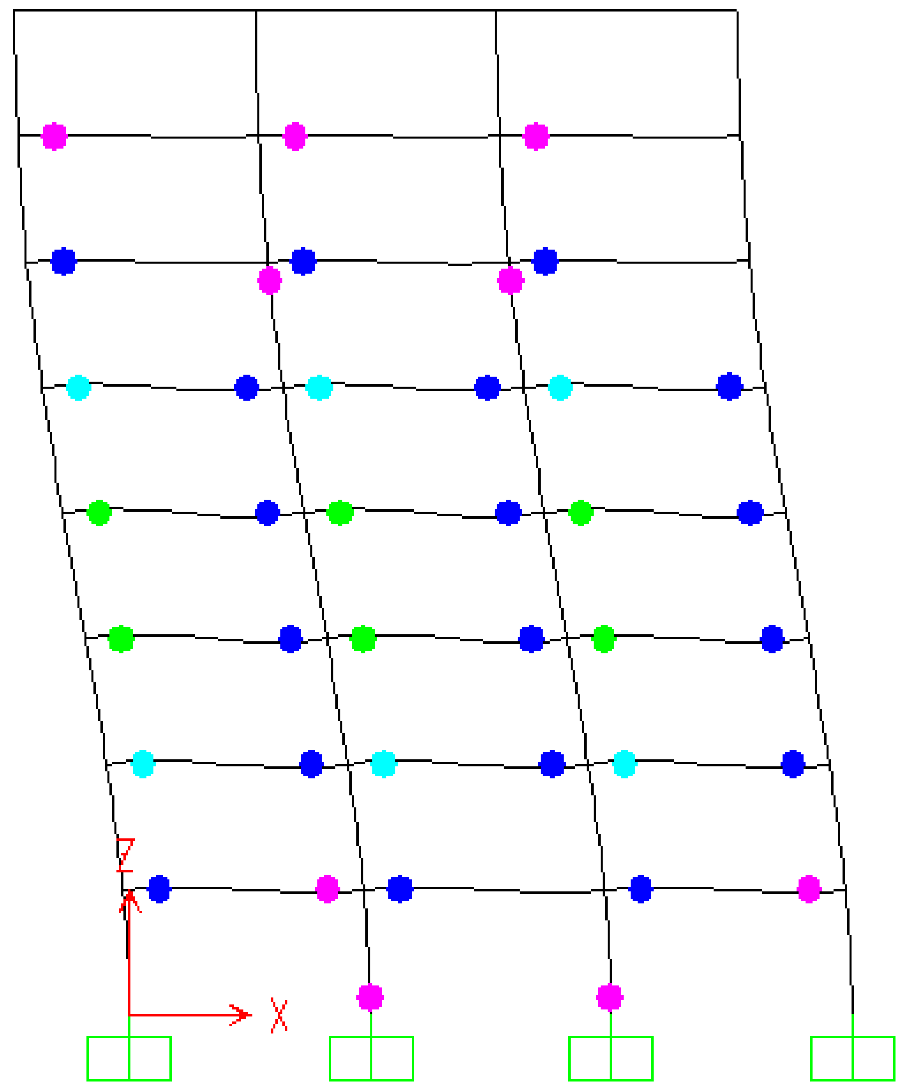


Figure A7 The hinge formation for eight story original frame under Northridge earthquake

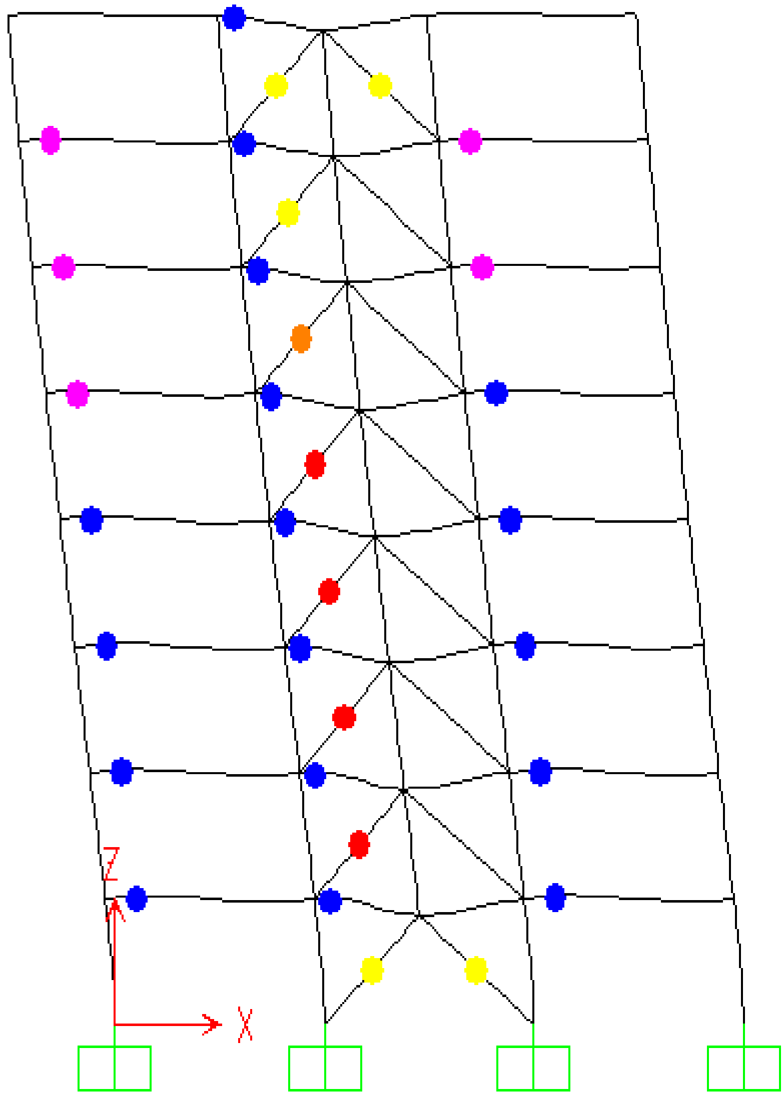


Figure A8 The hinge formation for eight story single zipper braced frame under Northridge earthquake

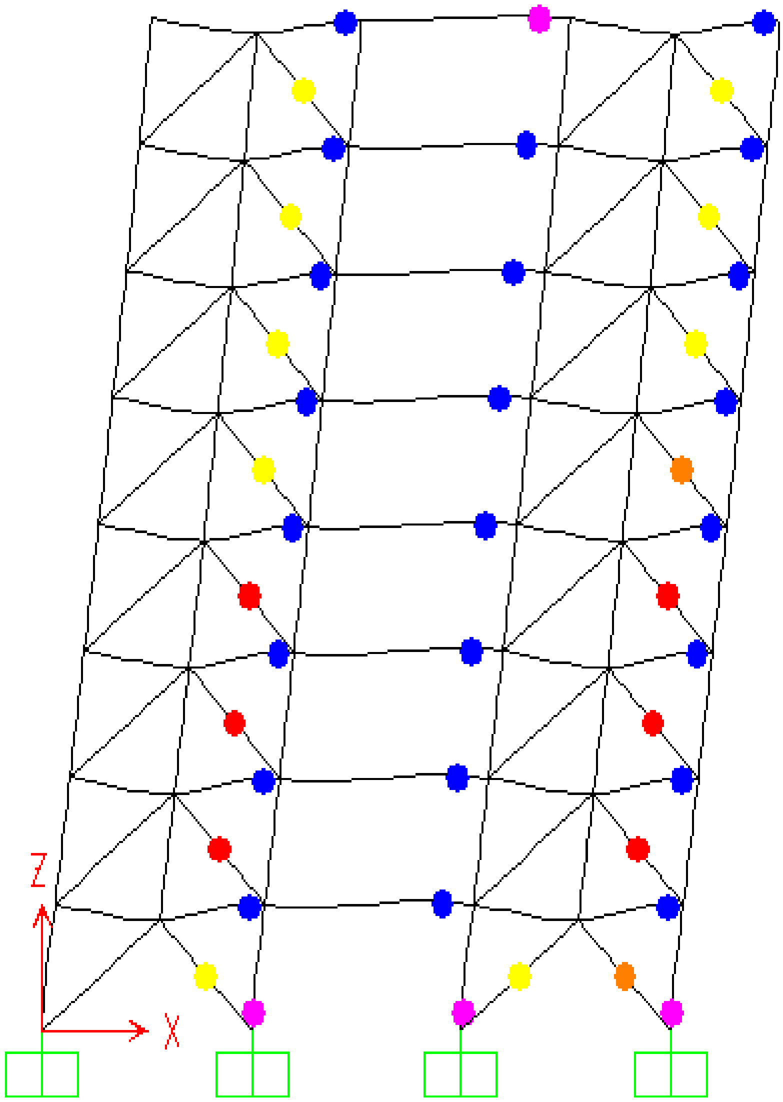


Figure A9 The hinge formation for eight story double zipper braced frame under Northridge earthquake