UNIVERSITY OF GAZIANTEP
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EVALUATING NONLINEAR BEHAVIOR OF A REINFORCED CONCRETE BUILDING WITH SHEAR WALLS AND CONCENTRIC STEEL BRACINGS

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ANWAR JABAR QADERSHEEN
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Evaluating Nonlinear Behavior of a Reinforced Concrete Building with Shear Walls and Concentric Steel Bracings

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Supervisor
Assoc. Prof. Dr. Esra METE GÜNEYİSİ

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Anwar Jabar QADERSHEEN

ABSTRACT

EVALUATING NONLINEAR BEHAVIOR OF A REINFORCED CONCRETE BUILDING WITH SHEAR WALLS AND CONCENTRIC STEEL BRACINGS

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The susceptibility of existing buildings has become an important issue since they did not satisfy modern seismic design requirements or they have been designed and constructed in accordance with the earlier codes. A great number of reinforced concrete (RC) buildings are located in the seismic zones and they have high vulnerability to damage. Therefore, either local or global strengthening strategies are required for such structures. This study aimed to assess the seismic performance of a typical residential building and its retrofitting by means of different approaches. For this purpose, an existing 9 story reinforced concrete (RC) building having the same floor plan (5x5 bays) was used as a case study. The building had a typical beam-column RC frame with no shear walls. For seismic performance upgrading of the building, RC shear wall and concentric steel bracing systems were employed. As a concentric bracing system, X-bracing, inverted-V bracing, and diagonal bracing were utilized. Analytical modelings of the existing and retrofitted RC buildings were realized by means of ETABS 2013. The performances of all structures were evaluated using the nonlinear static and dynamic analyses. From these analyses, the pushover curves, inter-story drift ratio, story displacements, and time history plots were obtained for the existing and retrofitted buildings. It was observed that the existing building had a performance level of collapse prevention and its performance improved to life safety after retrofitting.

Keywords: Reinforced concrete building; Nonlinear analysis; Retrofitting system; Shear wall; Steel bracing.

ÖZET

PERDE DUVAR VE MERKEZİ ÇELİK ÇAPRAZLI BETONARME BİNANIN DOĞRUSAL OLMAYAN DAVRANIŞININ DEĞERLENDİRİLMESİ

QADESHEEN, Anwar Jabar
İnşaat Mühendisliği, Yüksek Lisans Tezi
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Mevcut yapıların depreme karşı duyarlılığı modern sismik tasarım şartlarını sağlamadığından veya daha önceki kodlara göre tasarlanıp inşa edildiğinden önemli bir konu haline gelmiştir. Riskli deprem bölgelerinde çok sayıda betonarme yapı bulunmakta ve bu yapılar yüksek hasar görebilirlik riski taşımaktadır. Bu nedenle, bu tip yapılar için eleman veya sistem güçlendirme uygulamaları gerekmektedir. Bu çalışmada, tipik bir konut binasının ve onun farklı yaklaşımlar vasıtasıyla güçlendirilmiş durumlarının sismik performanslarının değerlendirilmesi amaçlanmıştır. Bu amaçla, aynı kat planına sahip (5x5 açıklıklı) 9 katlı betonarme bir bina örnek çalışma olarak kullanılmıştır. Yapı perde duvarsız tipik kolon-kiriş betonarme çerçevelerden oluşmaktadır. Yapının sismik performansını iyileştirmek için betonarme perde duvar ve merkezi çelik çapraz sistemler uygulanmıştır. Merkezi çapraz sistemi olarak, X-çapraz, ters-V çapraz ve diyagonal çapraz bağlantılar uygulanmıştır. Mevcut ve güçlendirilmiş betonarme binaların analitik modellenmesi ETABS 2013 programı kullanılarak gerçekleştirilmiştir. Tüm yapıların performansları doğrusal olmayan statik ve dinamik analizler kullanılarak değerlendirilmiştir. Bu analizlerden, kapasite eğrileri, göreli kat ötelenmeleri, kat yer değiştirmeleri ve zamana bağlı grafikler mevcut ve güçlendirilmiş yapılar için elde edilmiştir. Mevcut yapı göçme öncesi performans seviyesindeyken, güçlendirildikten sonra performansının can güvenliği seviyesine kadar iyileştiği gözlenmiştir.

Anahtar Kelimeler: Betonarme yapı; Doğrusal olmayan analiz; Güçlendirme sistemi; Perde duvar; Çelik çapraz

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LIST OF SYMBOLS / ABBREVIATIONS

AISC	American institute of steel construction
ASCE	American Society of Civil Engineers
ATC	Applied Technology Council
BRBF	Buckling Restrained Brace Frame
C_o	Modification factor for SDOF (MDOF)
C_1	Modification Factor to relate expected maximum inelastic displacements to displacements calculated for liner elastic response
C_2	Modification factor to represent the effect of hysteresis Shape on the maximum displacement response
C_3	Modification Factor to represent increased displacements due to dynamic P- Δ effects.
CFRP	Carbon Fiber Reinforced Polymer
CP	Collapse Prevention
EB	Existing Building
FE	Finite Element
FEMA	Federal Emergency Management Agency
FRP	Fiber Reinforced Polymer
g	Acceleration of gravity
IBC	International Building Code

IO	Immediate Occupancy
LS	Life Safety
MBF	Mega Brace Frame
MFDC	Mexico Federal District Code
M_w	The magnitude of ground motion
NDA	Nonlinear Dynamic Analysis
NDP	Nonlinear Dynamic Procedure
NSA	Nonlinear Static Analysis
NSP	Nonlinear Static Procedure
PGA	Peak ground acceleration
PGD	Peak ground displacement
PGV	Peak ground velocity
RC	Reinforced Concrete
RB-DB	Retrofitted Building with Diagonal Bracing
RB-IVB	Retrofitted Building with Inverted V-Bracing
RB-SW	Retrofitted Building with Shear Wall
RB-XB	Retrofitted Building with X-Bracing
S_a	Response spectrum acceleration
SCBF	Special Concentric Brace Frame
SFRC	Steel Fiber Reinforced Concrete
SRC	Steel Reinforced Concrete
T_e	Effective fundamental period of the building
Vs30	Shear wave velocities in 30 m

δ_t	Target displacement
Δu	Ultimate displacement
Δy	Yield displacement
μ	Ductility factor

CHAPTER 1

INTRODUCTION

Thousands of reinforced concrete (RC) buildings are placed on the hazardous seismic zones and they have high susceptibility to damage. The common structural destruction and various partial or global collapses measured in the out-coming of recent disturbing worldwide earthquakes have stressed the urgent needs to evaluate the seismic vulnerability of the existing RC buildings and occupy either local or global strengthening strategies, as suitable. Many of existing non-ductile RC building frameworks were, actually, built in urban areas before seismic codes become more detailed, therefore; the preparation of consistent and cost- effective seismic retrofitting systems is of paramount importance to enhance the structural system and public elasticity within the framework of the complete approach of seismic risk. Meanwhile unyielding behavior is proposed in most structures subjected to uncommon earthquake loading, the use of nonlinear analyses is important to evaluate behavior of structures under seismic special effects. It is commonly agreed that RC frames show highly nonlinear load deformation response because of principally to the nonlinear stress-strain behavior of the basic materials (Vecchio and Balopoulou, 1990).

Past earthquakes have stressed the experience of existing structures which did not fulfill recent seismic design requests and modern engineering standards; however, they may have been appropriately designed and constructed according to earlier codes. Many existing buildings may be insufficient strength to severe seismic. To moderate the seismic hazard of existing buildings, retrofitting system should be performed. The retrofitting system measures to improve the capacity of these structures can be executed. The assessment of the seismic capacity of existing buildings is important for the design. Additionally new buildings should be designed according to modern codes and applying retrofitting technique to enhance them against earthquake. The aim of the evaluation and retrofitting systems is either for collapse prevention and identify damage of structural and nonstructural components to reduce the risk of injury and to remain necessary circulation routs available (Farghaly and Abdallah, 2014).

Nowadays, steel bracings or RC shear walls are generally used as a main carrying load system in high-rise buildings. The appreciation of the dual structural system seismic behavior can be accommodating to structural engineers in choosing a suitable structure that are being considered, for different purposes such as increase of energy dissipation of the buildings and its capacity to withstand lateral displacements of the buildings that have moment-resisting frames. Reviewing RC building behavior as exposed to serious earthquake ground motions provide that this type of structures can show adequate strength, because of the nonlinearity of materials and sufficient amount of deformations of the structures. The applied energy will be absorbed by these type of structures and will be dissipated it through bearing great displacements under nonlinear seismic behavior (Esmaeili et al., 2013).

Since the nonlinear behavior is proposed in many structures subjected to uncommon earthquakes, the use of nonlinear analyses is important to occupy the behavior of structures during seismic special properties. To perform inelastic properties of RC frames it has proposed to use the nonlinear static procedure (NSP) or pushover analysis, because of its simplicities as defined in ATC-40 and FEMA-356. It is generally believed that, when pushover analysis is used wisely, it supplies valuable evidence that cannot be found by linear static or dynamic analysis methods (Inel and Ozmen, 2006).

1.1. Objectives of the Thesis

The main objective of this study is to investigate the nonlinear response of an existing 9-story reinforced concrete (RC) frame building retrofitted with shear wall or concentric steel bracing. As a concentric bracing system, X-bracing, inverted V-bracing, and diagonal bracing were used. Analytical modeling of the existing RC building (EB) and retrofitted building with shear wall (RB-SW), X-bracing (RB-XB), inverted V-bracing (RB-IVB), and diagonal bracing (RB-DB) were carried out by using nonlinear static analysis and nonlinear dynamic analysis. The three dimensional (3D) model of the structures were performed by using finite element analysis software ETABS. The results of the analyses were obtained in terms of capacity curve, displacement, interstory drift ratio, displacement time history, base shear time history, etc. The performances of the existing and retrofitted buildings were discussed comparatively.

1.2. Outline of the Thesis

Chapter 1-Introduction: The aim and scope of the thesis were introduced.

Chapter 2-Literature review: The previous studies based on the scope of the study was reviewed and obtained. For this, firstly, the research and development on the nonlinear behavior of the reinforced concrete buildings in the literature were given. Secondly, the retrofitting systems, especially reinforced concrete shear walls and steel bracings were outlined.

Chapter 3-Case study: This chapter provided the explanation of analytical modeling of the existing reinforced concrete building and its retrofitting cases. Furthermore, the methodology used in the analysis and design of the building structures was summarized and details for every step were given in this chapter.

Chapter 4- Results and discussion: Results obtained from the nonlinear static analysis and nonlinear dynamic analysis for the existing RC building and that with shear walls or steel bracings were given. Discussion on the results of the analysis was described in this chapter.

Chapter 5-Conclusions: The conclusions built on the results of these comparative investigations were provided in this section.

CHAPTER 2

LITERATURE REVIEW

2.1 Reinforced Concrete Buildings

Concrete buildings are one of the most common buildings in many countries because of availability of raw materials of reinforced concrete (cement, sand, gravel, steel and water), and easy in construction. Reinforced concrete is a composite material consists of concrete and steel. Concrete is the most popular materials used to construct buildings that can be formed in different shapes and desired cross-sections. Cement concrete has been used from the time when the Portland cement developed. The main advantages of using reinforced concrete for building construction (Nilson, 1997)

i. It has superior resistance to fire than steel or wood,

ii. It has high compressive strength, and

iii. It has low maintenance cost.

Figure 2.1 Residential Reinforced Concrete building in Soran, Erbeel

Concrete has high compressive strength but has limited tensile strength about ten percent of its compressive strength and zero strength after cracks develop as shown in Figure 2.2. Therefore; to improve concrete so as to increase tensile strength steel bars used with concrete to product reinforced concrete which has both compressive and tensile strength. On the other hand using of steel bar with concrete is more economical. There are many codes to design reinforced concrete structures, but to day seismic qualifications of the building has become highly important, thus nonlinear properties of each component of the structure should be determined to design reinforced concrete structures for resisting earthquake (Nilson, 1997).

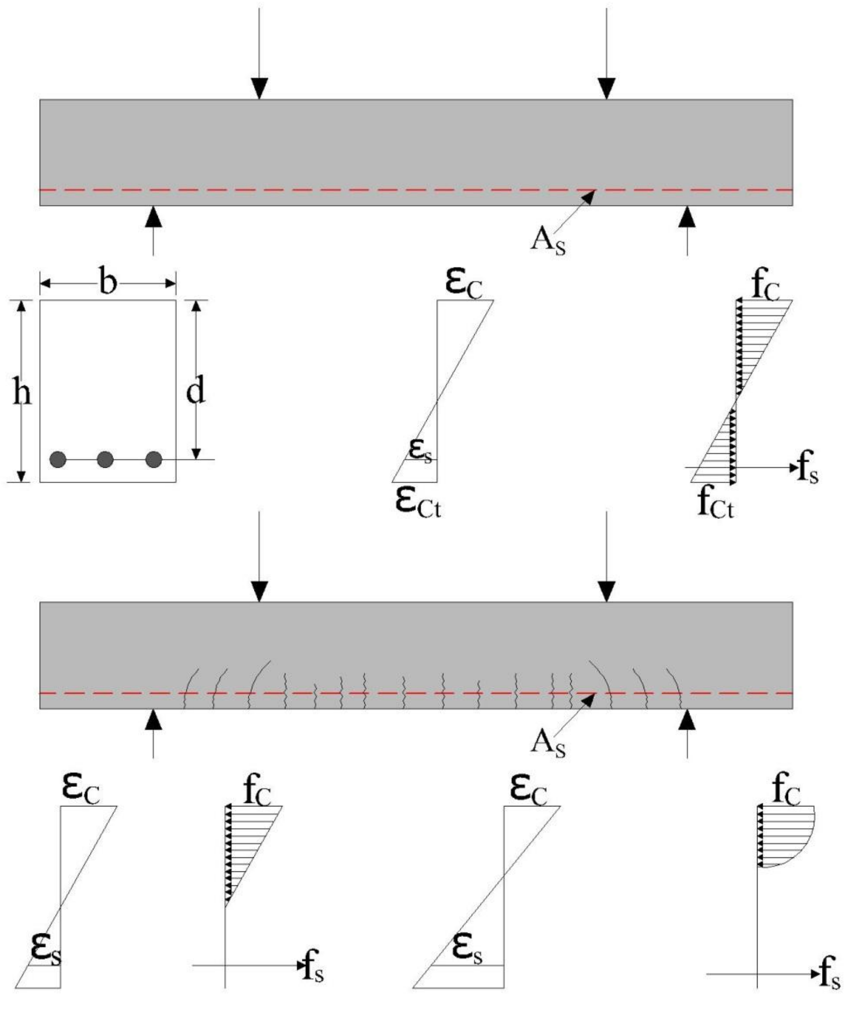


Figure 2.2 Behavior of reinforced concrete beam under loading (Nilson, 1997)

2.1.1 Nonlinear Behavior of Reinforced Concrete Frame Building

Elastic analysis is commonly used in designing buildings for seismic resistance, at the same time it will experience significant inelastic deformations under severe earthquakes. The purpose of the realistic behavior of the building structures recommended in recent performance based design procedures. It is paramount important to have the knowledge of building structure response beyond elastic range to obtain many vital features which control the seismic performance of building structures in large earthquakes. During the nonlinear analysis, strength, structure capacity, performance levels, hinge properties, and failure mechanism are established. The nonlinear behavior depends on the nonlinearities of materials used in the structures, nonlinear stress-strain relationships are a general cause of nonlinear structure behavior (Vecchio and Balopoulou, 1990). Also, large deflection can cause the structure to response nonlinearity as given in the Figure 2.3.

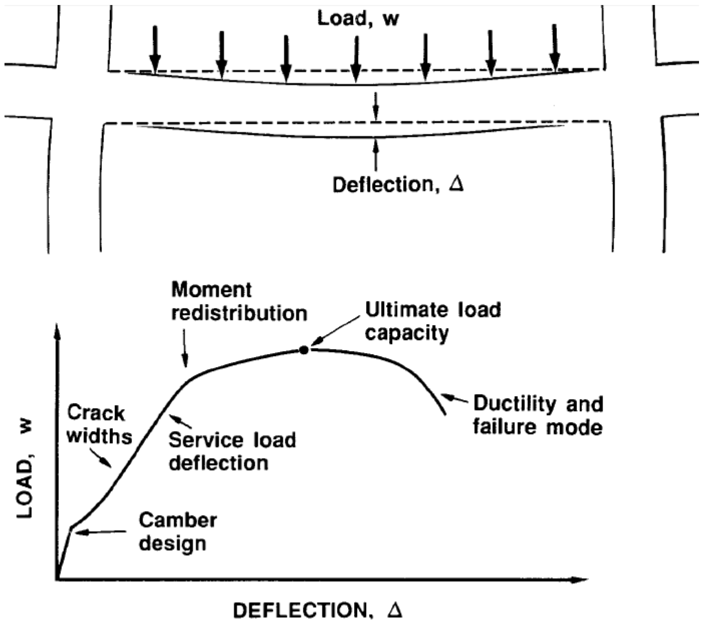


Figure 2.3 Typical frame behavior influenced by nonlinear response (Vecchio and Balopoulou, 1990)

Vecchio and Balopoulou (1990) studied experimentally by using large-scale reinforced concrete frames to determine which factors were effecting the nonlinearity of reinforced concrete frame under short-term loading condition. The test results showed that frame behavior greatly influenced by second-order such as geometric nonlinearities, material nonlinearities, shear deformation, membrane action, concrete shrinkage and torsion stiffening effects. Also for further improvement, it was necessary to consider another aspect of behavior such as failure mechanism, deflection response and ultimate capacity. In their study a large-scale one-span, two-story plane frame with span 3500 mm center to center, 2000 mm height of the story, the frames members were 300 mm wide and 400 mm deep with heavy reinforced concrete base. The material properties (concrete and longitudinal reinforcement) in their study experimentally determined as shown in Figures 2.4 and 2.5 standard cylinder tests of 150 mm x 300 mm dimensions were taken, from the above stress–strain relation curve, the compressive strength of the concrete was 29 MPa tested by the test machine using a stroke rate of 6.67 x 10-3 mm/s after curing 25 days for the concrete. Reinforcement rebar of 20 mm diameter used as longitudinal reinforcement for all members with yield stress of 418 MPa, ultimate stress of 596 MPa and have modulus of elasticity of 192600 MPa. The test observations showed that frame nonlinear behavior depended on the material nonlinearities and geometric nonlinearities.

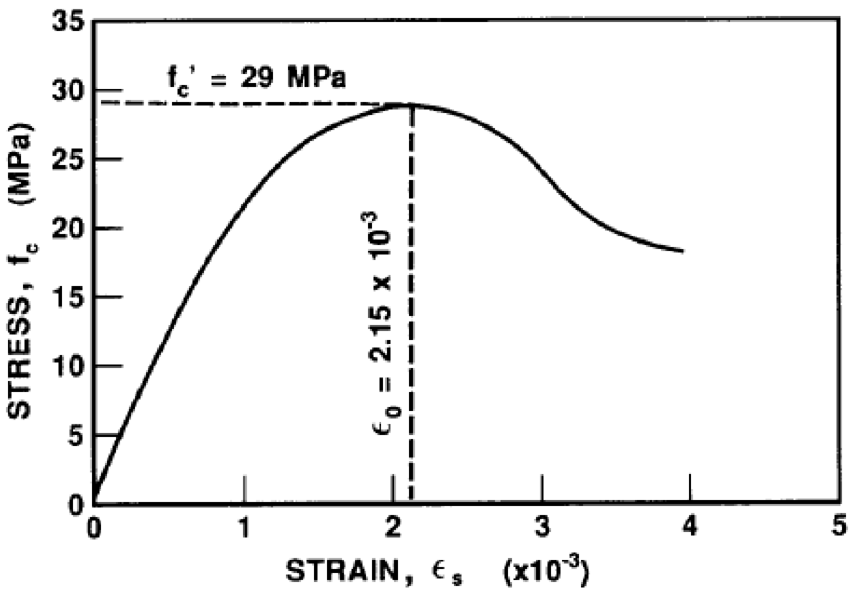


Figure 2.4 Stress-strain curve of the concrete (Vecchio and Balopoulou, 1990)

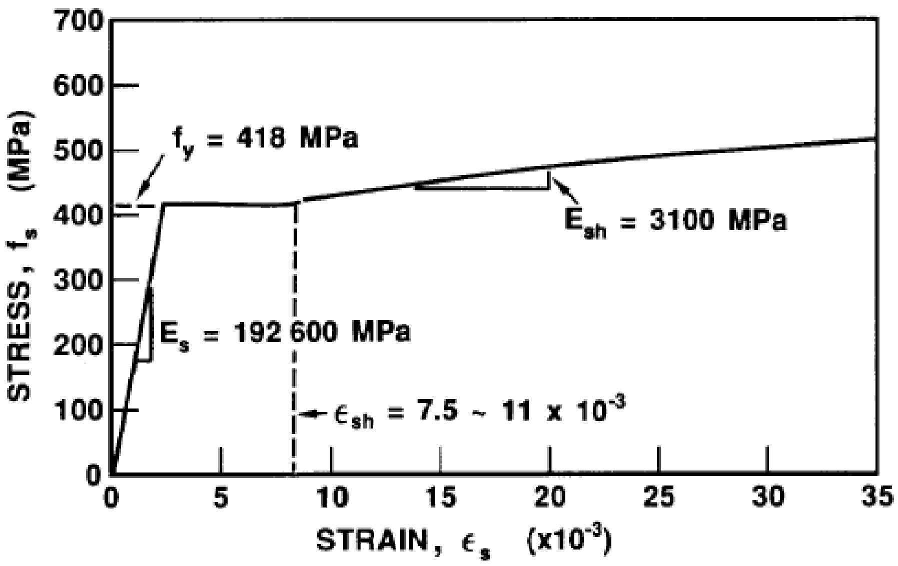


Figure 2.5 Stress-strain curve of the reinforcement (Vecchio and Balopoulou, 1990)

Fahjan et al. (2010) studied reinforced concrete shear walls designed with different methods either using combination of frame elements or shell elements. Plastic hinge applied for the structure elements located on the plastic zones at the end of the elements. Different estimations for linear and nonlinear analysis of the shear walls were investigated. SAP2000 (CSI, 2009) was used for the nonlinear analysis of the various models. To investigate the nonlinear behavior of dissimilar shear walls, nonlinear static analysis was performed. According to results of this research, the shear walls with two layers of longitudinal and transverse reinforcement bars could be formed with varied systems descripted material nonlinearity of the reinforced concrete elements.

Su and Wong (2007) tested on three reinforced concrete wall models with high concrete strength and high longitudinal steel ratio were formed under combined axial load, shear, and moment. The strength of confinement was observed to be highly reliant on the organization of the transverse reinforcement. Simply increasing the quantity of the transverse reinforcement in the sample might not produce extra degree of confinement. Axial load ratio had considerable effect on the deformability and failure manner of the samples. The maximum variation ductility decreased with increases in axial load ratio. Compression failure mode was performed when the axial load ratio was high. Moreover, an increase in the axial load ratio had a detrimental effect on strength degradation and energy dissipation of reinforced concrete walls.

Duan and Hueste (2012) investigated on a typical five-story reinforced concrete frame building conferring to the provisions of the current Chinese seismic code (GB50011-2010). Seven natural earthquake acceleration records were adapted for conformity with the accepted design spectrum. The frame structure was estimated using nonlinear dynamic time-history and analysis nonlinear static analysis. According to the results, the building frame designed by GB50011-2010 provided the post-yield behavior and response projected by the code and contented the interstory drift and maximum plastic rotation limits recommended by ASCE/SEI 41-06. However, the pushover analysis showed the potential for a soft first story mechanism under significant lateral demands.

Godínez-Domínguez and Tena-Colunga (2010) used static nonlinear analyses to show the nonlinear behavior of medium to low ductile moment-resisting concentric braced frame structures supposed that placed in soft soil circumstances in Mexico city using capacity design technique adjusted to the common requirements of the seismic, reinforced concrete and steel guidelines of Mexico Federal District Code (MFDC-04). 4 to 24 story designed buildings executed using Drain-2DX. It was observed that the capacity design method used by the researchers was profitable in the design in low and medium rise ductile frames when the columns of the moment frames withstood not less than 50% of the total seismic shear force. The results showed numerical verification to stand by the strength balance recognized in MFDC-04 so as to approve ductility for this structure to perform the collapse mechanism expectation, mainly for medium to high rise buildings.

2.1.2 Ductility of Reinforced Concrete Building

In modern buildings, ductility has become a main point considering for all designing of the buildings to increase significant performance of the buildings against collapse. Ductility is paramount important specially for reinforced concrete buildings to obtain enough strength during ground motion so as to minimize the risk of injury or casualties and to keep essential circulation routes accessible. Ductility is the behavior of reinforced concrete buildings after proportional or linear behavior. The ratio of ultimate displacement (Δu) to yield displacement (Δy), called the displacement ductility factor μ as shown in Figure 2.6. The capacity of a structure to withstand plastic deformations is identified as ductility and it is principal importance for seismic design to give the best choice for the building design. Performance-based design is a new technique for the seismic design and the performance levels for the nonlinear procedures indicate further significant building performance than for the linear procedure. (Penelis and Kappos, 1997)

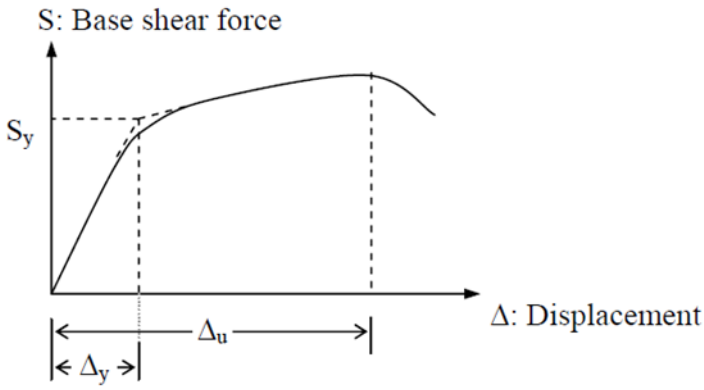


Figure 2.6 Definition of the displacement ductility factor (Penelis and Kappos, 1997)

Tawfik et al. (2013) experimented on a study for the behavior and ductility of high strength reinforced concrete frames. The structures with dissimilar cross section of the columns were investigated. The stirrups were organized in half spacing distance at connection with detail to even distance of all columns and beams. The dimension of frames was designated to represent half scale frames and confirmed under cyclic loading. All samples of the experimental program were experimented in the reinforced concrete testing laboratory. According to the results, the increase of inertia of beam for the frame developed the ultimate lateral load, energy dissipation, and stiffness by a small value while it reduced the ductility factor. The lateral load resistance decreased when the h/L ratio decreased.

In the study of Adiyanto et al. (2011), they used 3 and 18 story reinforced concrete buildings subjected to lateral loading. To predict the building capacity against lateral load, pushover analysis that provides the information regarding the strength and lateral displacement of structural system was performed. They observed that the 18 story model provided more ductility compared to the 3 story model counterpart, and also predicted that the same level of R-factor, the 3 story model would experience larger response when subjected to earthquake compared to 18 story models.

2.2 Retrofitting Strategies

Many existing and new reinforced concrete structures located in seismic zones. Most of them have insufficient lateral strength in severe earthquakes. There are some methods used in buildings for strengthening them, these techniques are called retrofitting systems. The aim of retrofitting systems to improve the existing buildings provides stiffness, strength, and energy dissipation needed to resist lateral load forced by earthquakes. Figure 2.7 shows the shear force versus displacement relations for different strengthening strategies (Ambrose and Vergun, 1995).

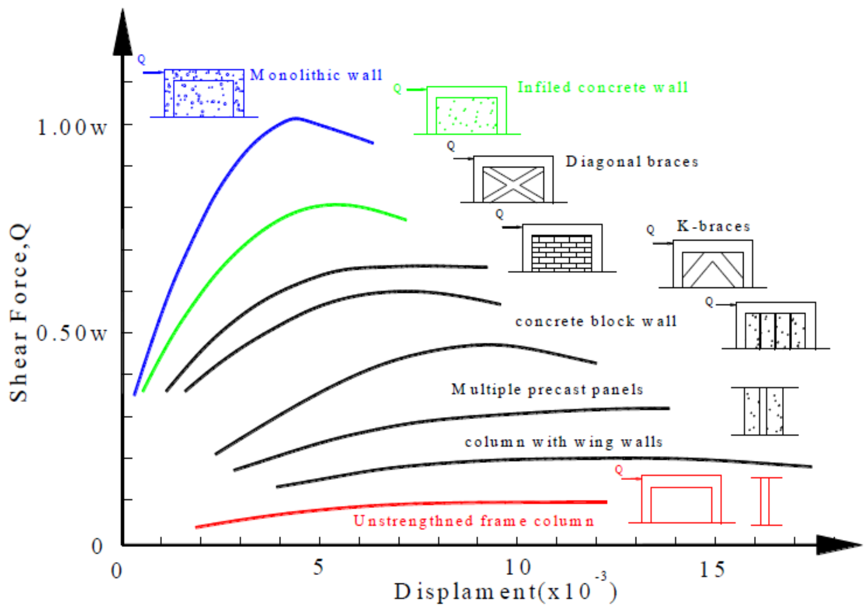


Figure 2.7 Relation between shear forces and displacement in different kinds of strengthen techniques (Ambrose and Vergun, 1995)

Shear wall and steel bracing systems are most widely used in reinforced concrete buildings because they have adequate strength and more feasible solution for seismic retrofitting of buildings. Many existing RC buildings need retrofitting to increase seismic load resistance. The Figure 2.8 shows the rehabilitation techniques used in some buildings in Japan. It was reported that the most used technique was adding shear wall to the existing buildings (Gajanan et al., 1996).

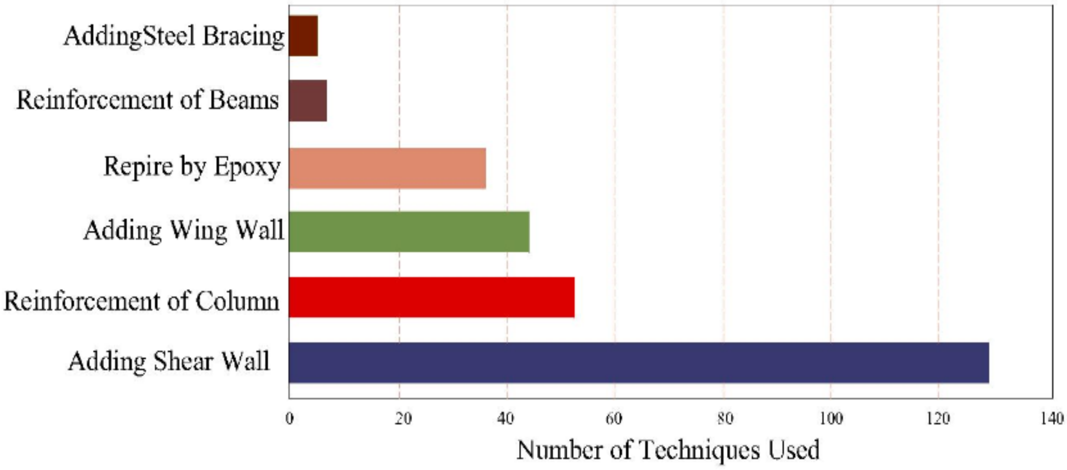


Figure 2.8 Rehabilitations techniques used in some building in Japan (Gajanan et al., 1996)

Farghaly and Abdallah (2014) used four reinforced concrete models consisting of frames, beams and columns as part of the lateral and vertical resisting system retrofitted with different techniques used in Egypt. In their study several retrofit techniques were evaluated by relevant on different kinds of existing structures to indicate the suitable one of tested structures. Four retrofit techniques, normally, reinforced concrete shear walls, steel bracing, column jacket, and column strengthen by 4 steel angles at each corner were considered. Each technique was examined on each tested structure. Examination on seismic strengthening techniques of the RC existing buildings by column jacketing, steel bracing, and RC infill walls was performed. It was established that with the correct structural system, it was possible to create a successful design for strengthening the existing structures and choice of technique type depending on type and situation of the structure. It was found that the application of shear walls to the structural system improved the capacity of the bare frame as predicted.

2.2.1 Reinforced Concrete Shear Wall

Reinforced concrete (RC) shear walls are vertical elements start at foundation level and are continuous through the whole of building height. Shear walls have the stiffness and strength to resist the horizontal forces. RC shear wall buildings are a common selection used in many reinforced concrete buildings located in earthquake zones. Nowadays applying shear walls with buildings become most popular retrofitting techniques because buildings that designed and detailed properly with shear walls have shown very good performance in past earthquakes. Figure 2.15 shows the reinforced concrete shear wall structures (Belmouden and Lestuzzi, 2007).

Esmaeili et al. (2013) used the dual structural systems in the system of steel moment-resisting frames joined with concentrically braced frames and steel moment-resisting frames joined with reinforced concrete shear walls. The evaluation of the nonlinear behavior of the 30-story structures under earthquakes was observed. The results showed that the steel moment-resisting frames joined with reinforced concrete shear walls system had higher ductility and reaction modification factor than the steel moment-resisting frames joined with concentrically braced frames.

Liao et al. (2012) carried out a series of tests consisting of reinforced concrete (RC) shear walls with steel reinforced concrete (SRC) boundary columns specimens and to simulate the composite shear walls under constant axial load and lateral loading to evaluate ductility, strength, and energy dissipation a finite element (FE) model was advanced. To show the effect of varies limits on the significant performance of reinforced concrete shear wall with SRC boundary columns, parametric studies were considered comparatively. Figure 2.9 revealed the view of SRC-RC wall model. A larger section column dimension made the wall section to be crushed firstly and the column could withstand the vertical load in the later step, therefore avoiding the building collapse and constructing the structure to be repaired easy. The test results showed that the SRC–RC walls could be considered as resisting systems for buildings situated in earthquake districts.

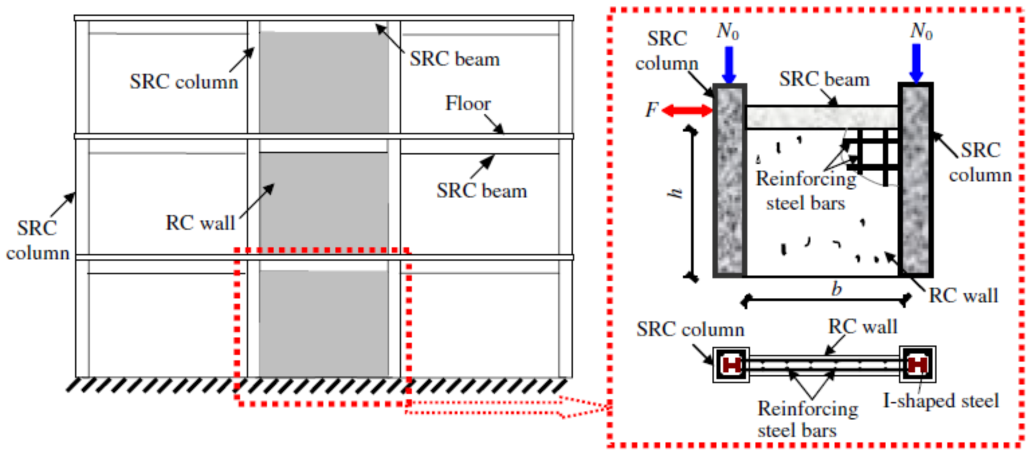


Figure 2.9 Schematic view of SRC–RC wall model (Liao et al., 2012)

Le-Nguyen et al. (2014) investigated experimentally and numerically on two lightly reinforced concrete walls with different aspect ratio. Dissimilar retrofitting strategies have been evaluated using carbon-fiber-reinforced-polymer (CFRP) materials. Pushover analysis have been used for two reformed numerical methods for the concrete examples: the first one was two dimensional plane stress approach with local concrete established on crack pattern and fracture energy while the second one investigated elastoplastic damage form utilized local approach in two dimensional and three dimensional imitations to consider the performance of the reinforcement by CFRP strips on the lightly reinforced concrete wall behavior. In both conditions, it was presented that the strengthening by CFRP improved the crack form at failure because of more performance shear properties for the reinforced concrete walls.

Pecce et al. (2014) studied the performances evaluation of a whole RC building with large lightly reinforced walls along the perimeter. Internal frames were also examined by static nonlinear and linear dynamic analysis. The typical applied in SAP2000 performed to be capable in terms of global behavior. They demonstrated that RC buildings with large lightly reinforced walls on the perimeter appear to be structural forms considered by a certain global ductility. Structural reinforced concrete (RC) walls were effectual systems for buildings that had critical withstand essential seismic actions, commonly because they admitted limiting displacements in tall buildings.

Belmouden and Lestuzzi (2007) used capacity design technique to investigate the reinforced concrete walls designed for seismic loads. The properties of the hysteretic behavior of the RC walls, counting stiffness degradation, strength degradation, pinching and slippage, bond slip effect, plastic shear deformation mechanisms and internment effects were obviously formed methodically and experimentally. The application of capacity design principles in structural walls led to an existence of plastic deformations in determining the locations and to developing the mechanisms for the energy dissipation. Commonly, according to the experimental results, they proposed a technique that reliably used for reinforced concrete structures to retrofit them for performance predictions. Figure 2.10 shows the view of test specimen of reinforced concrete wall.

Figure 2.10 View of the reinforced concrete walls (Belmouden and Lestuzzi, 2007)

Kitada et al. (2007) tested the reinforced concrete (RC) shear walls for a project to evaluate the three-dimensional (3D) RC shear wall behavior under earthquake load condition. The seismic design of nuclear power plant building was performed. In their paper, they defined an evaluation of the complete testing and reviewed the main results mined by the test project by mentioning the available papers relating the results of the 10-year test project. However, the results played a significant part in assessing seismic limits of essential structures in a nuclear power plant were performed of the project. It was established that the legality of the analytical procedure utilizing finite element technique to estimate the RC structure behavior to some degree of collapse under the multi-axes loading procedure.

Parulekar et al. (2014) investigated analytical and experimental methods to determine the behavior of reinforced concrete stiff squat shear wall subjected to reversed cyclic loading settings. Reinforced concrete squat shear walls compromise excessive possible for lateral load resistance. A squat short shear wall of width 3 m, 1.2 m height, and 0.4 m thick was tested. Two-dimensional and three-dimensional finite element programs for analytical simulations were performed. Experimental results exhibited that squat shear wall provided highly strained hysteresis curve with little energy dissipation and are seriously subjected by shear connected mechanisms.

2.2.2 Steel Bracing

Steel bracing is an important system in the structures used to resist earthquake loads in buildings. It is necessary to retrofit many existing reinforced concrete buildings so as to increase significant performance to withstand seismic loads. The applying of steel bracing as a retrofitting systems seismically insufficient resistance of reinforced concrete frames for improving them have enough strength during earthquake. Steel bracing is an extremely effective and economical technique of resisting horizontal forces in a frame structure. Figure 2.11 illustrates the use of exterior steel braced frames as a retrofitting method (Viswanath and Desai, 2008).

Figure 2.11 Exterior steel braced as a retrofit solution (Viswanath and Desai, 2008)

Maheri and Sahebi (1997) used steel bracing in concrete frame structures with different diagonal bracing arrangements through a series of tests. The aim of using bracing was to upgrade in-plane shear strength of concrete frames and the effect of each form of bracing on the concrete frames. For this purpose, the diagonal X-bracing system was selected and four model frames were investigated: (1) a concrete frame braced with X-bracing; (2) a concrete frame braced with a diagonal compression brace; (3) a concrete frame braced with a diagonal tension frame; (4) a concrete frame without bracing. The results indicated outsized increase in-plane shear strength of concrete frame because of using of diagonal bracing in one side acting either in tension or compression. Concrete frames braced with tension or compression diagonal bracing increased the strength about 2.5 times with concrete frames without steel bracing and when X-braced frame model used the in-plane shear strength increases about 4 times that of the un-braced system. The failure starts with the tensile failure of the bracing after compression brace buckling failure.

Youssef et al. (2007) studied on the use of concentric internal steel bracing in concrete frames applying a for-story reinforced concrete building of 12 m by 12 m dimensions. It was appropriated the building which is placed in a highly seismic area categorized as category C in the International Building Code (IBC). Two lateral loading systems were studied, reinforced concrete moment frames and braced reinforced concrete frames. The efficiency of applying braced frame was experimentally estimated. The results of this research presented that the braced frame sustained higher lateral load and produced sufficient ductility than the moment frame.

Massumi and Absalan (2013) conducted an investigation on steel bracings to advance the performance of the reinforced concrete frames. The results of two experimental models of the reinforced concrete frames were analyzed. Also, ANSYS software was performed to compare the results numerically and experimentally. Both models of frames were designed according to old traditional codes. For this case, a specimen was used, the connection of bracings to frame was observed by the use of angle which was located previously in the corner of the column and the beam connection before concrete pouring. So, there were anchorages were welded on the angles connected to the reinforcement of the frames after that concrete frame was casted and this was the possible method to use steel bracing system. Figure 2.12 shows the testing condition of the reinforced concrete frame with steel bracing. The results showed that adding bracing to the reinforced concrete moment resisting frame increased the strength, energy absorption, and stiffness. On the hand, the results revealed the significant performance between the reinforced concrete and the bracing system improved the behavior dual system. The result of the numerical analyses demonstrated a development of 18.34% in the ultimate strength of the dual system because of the inter-action between two systems.

Figure 2.12 Increasing the stiffness of beam–column connection due to steel angle and gusset plate (Massumi and Absalan, 2013)

Maheri and Hadjipour (2003) used an unit model frame designed for investigational study were 1:3 scaled models of a standard 3 m x 3 m unit ductile frame. The complete size unit frame was selected from a typical 4-story, 3-bay entry frame of the building and analyzed for the combined effects of gravity and seismic loading. The researcher showed the strength capacity and the yield capability of the ductile reinforced concrete frame increased also the total displacement decreased by directly applying either an X-bracing or knee-bracing systems which can be performed to retrofit or design for destruction level earthquake.

Safarizki et al. (2013) assessed the possible enhancement of seismic performance of an existing reinforced concrete building by the utilizing of steel bracing. Three techniques of seismic evaluation are employed for the purpose of the investigation; nonlinear static pushover displacement coefficient as defined in FEMA 356, improvement of nonlinear static pushover displacement coefficient as defined in FEMA 440 and dynamic time history analysis following the Indonesian code of seismic resistance building (SNI 03-1726-2002) criteria. The performance of this building could be classified in between life safety (LS), and collapse prevention (CP). The plastic hinges occurred in columns. Consequently, it was shown from the nonlinear pushover analysis that target displacements in both directions were decreased by 16%-55% while the steel bracings were performing.

Görgülü et al. (2012) studied on the improvement of reinforced concrete (RC) structures with steel shear walls in an external manner. The provisional program involved three-dimensional RC models. The connected tests were directed under the imposed reversed cyclic lateral sway as shown in the Figure 2.13 Consequently, performance of the proposed strengthening system was found to be sufficient for refining the seismic capacity of existing RC structures. Further the stiffness and base shear capacity of the confirmed model were considerably advanced. The capacity of the original model which calculated was 67.35 kN. Using external steel bracing increased the lateral load bearing capacity to167.24 kN. In accordance with the results, improvement was performed by 148% with respect to the origin model which was included frame model without steel bracing.

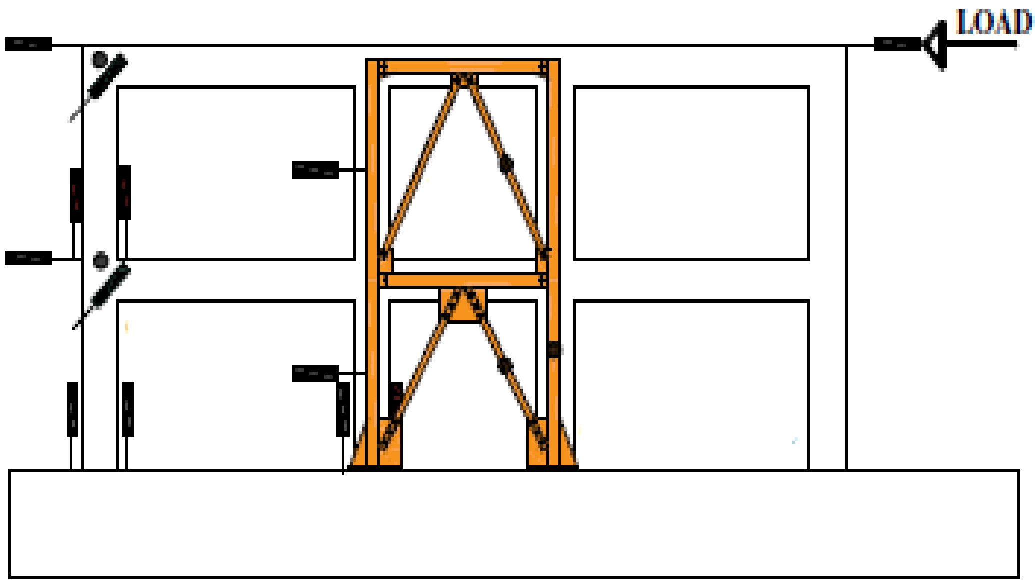


Figure 2.13 Typical frames with external steel bracing. (Görgülü et al., 2012)

Maheri et al. (2003) selected a full size unit frame having 4-storey and 3-bays. Moreover, a proto type structure for investigational study were 1:3 scaled models of a typical 3 mx3 m unit ductile frame was constructed . The seismic loading applied to the frame was evaluated using an equivalent static procedure. The purpose of retrofitting, X-or knee bracing were used to increase the lateral capacity and decrease the global displacements to the desired level. According to the results, adding knee-bracing and X-bracing systems performed to upgrade the yield capacity of a reinforced concrete frame. It was shown that in many circumstances the existing RC frames successfully achieved by retrofitting system applying direct internal bracing.

Durucan and Dicleli (2010) suggested a retrofitting system to resist seismic condition that arranged to advancement the performance of seismically susceptible reinforced concrete (RC) structures. The proposed seismic retrofitting system was collected of a steel housing rectangular frame with V-shape braces and an elastic shear link linked between the frames and the braces. The braced system was set up in the bays of an RC building frame to improve the strength, ductility, and stiffness of the structure. The research demonstrated during their study that new performance created retrofitting design method confirmed a satisfactory performance of the retrofitted buildings using the retrofitting systems. In addition, the performance of the retrofitting arrangement was evaluated in assessment to that of a conventional retrofitting system consuming to determine the more required performance of the proposal retrofitting system compared to existing systems.

Di-Sarno and Elnashai (2009) used steel moment resisting frames (MRFs) retrofitted with dissimilar bracing systems. Three structural formations were operated: mega-brace frame (MBF), special concentrically- brace frame (SCBF), and buckling-restrained brace frame (BRBF). Past yielding time-history analyses were performed to evaluate the structural performance using earthquake ground motions. A building with 9-storey steel MRF was considered with lateral stiffness inadequate to gratify code drift limits in regions with extraordinary seismic vulnerability. Their investigation presented that instant resisting frames (MRFs) with inadequate lateral stiffness could be improved using diagonal braces. The outcomes of the inelastic analyses established that MBFs were the most cost-effective. The decrease in inter-story drifts concern to the original MRF is on average equal to 70%. Determined lateral drifts in MBFs are 45%–55% lower than SCBFs; the decreases in overall distortions rested on the appearances of earthquake ground indications, particularly regularity satisfied.

Maheri and Ghaffarzadeh (2008) studied experimentally and numerically the structural behavior having 4.0 m by 3.0 m dimension. For the examinations, cyclic loading investigations were directed on scaled moment resisting frames with bracing as shown in Figure 2.14. An important concern in the design of steel-braced RC frames was the level of interface between the strength dimension abilities of the RC frame and the bracing system. It was expected that the buildings were located in extremely seismic area. Braced moment frames and moment frames were evaluated. Steel bracing was added for seismic improvement of existing RC buildings. It was reported that greater strength in a braced RC frame was because of the stiffening properties of contacts. This strength increment was designated as the capacity interaction or assembly over strength.

Figure 2.14 Steel-braced reinforced concrete frame (Maheri and Ghaffarzadeh, 2008)

Rahai and Alinia (2008) studied on two parts of composite bracings. A number of braced frames were designated to define their behaviors under cyclic loading. Additionally, the existing concrete structures of nine and three story buildings were designated. The pushover analysis was performed to find and compare the results of two types of bracings. The results indicated that all cases reinforced by compound bracings, plastic hinges were molded in comparatively smaller lateral displacements in the nonlinear range. Therefore, an advanced resistance factor was achieved.

Kadid and Yahiaoui (2011) studied on the RC buildings braced with dissimilar categories of steel braces, inverted V braced, X-braced, ZX braced, and zipper braced to evaluate the seismic behavior. Nonlinear static pushover analysis was performed to evaluate the ability of three story and six story buildings with reformed brace frame systems and dissimilar cross sections for the braces. It was established that applying braces improved the total performance of the buildings in relations of deformation, strength, and ductility related to the circumstance with no bracing, and the X and zipper bracing organizations achieved well conditional on the type and size of the cross section.

2.2.3 Jacketing

The associate level retrofit or limited retrofit of strengthening method is to improve the strength of the structures, which are seismically deficient. This method is more cost effective as compared to the structure level retrofit. The most general method of improving the individual member strength is jacketing. It contains the accumulation of concrete, steel or fiber reinforced polymer (FRP) jackets for use in limiting reinforced concrete columns, beams, joints, and foundations (Ismail, 2013).

Ismail (2013) executed a two-dimensional model of the structure with a floor of very weak concrete strength of 9MPa. The normalized base shear force was determined in accordance with Egyptian loading code (EC201-2012). Nonlinear static analysis was applied. Three kind of retrofitting techniques were studied; carbon fiber-reinforcement polymer (CFRP) composite jacket, steel jacket and reinforced concrete jacket as shown in Figure 2.15. In their study showed that all retrofitting methods enhanced the ductility properties of the reinforced concrete structure. The columns retrofitted full steel jacketing using steel plate or reinforced concrete jacketing improved structure performance with regard to strength and ductility. According to the result reinforced concrete jacketing perhaps more recommended when lateral drifts are needed to be limited. Partial steel jackets technique was not the best one for retrofitting the low strength of concrete however it would provide the ductility of the building and it did not considerably advance the total flexural of the structure elements as clearly shown in Figure 2.16.

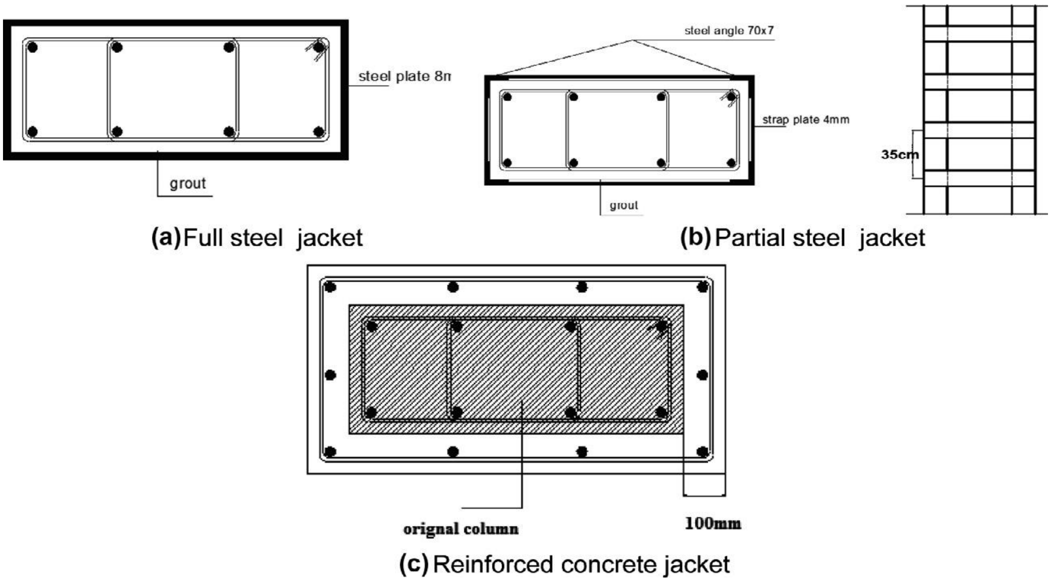


Figure 2.15 Typical jacket details for reinforced concrete columns (Ismail, 2013)

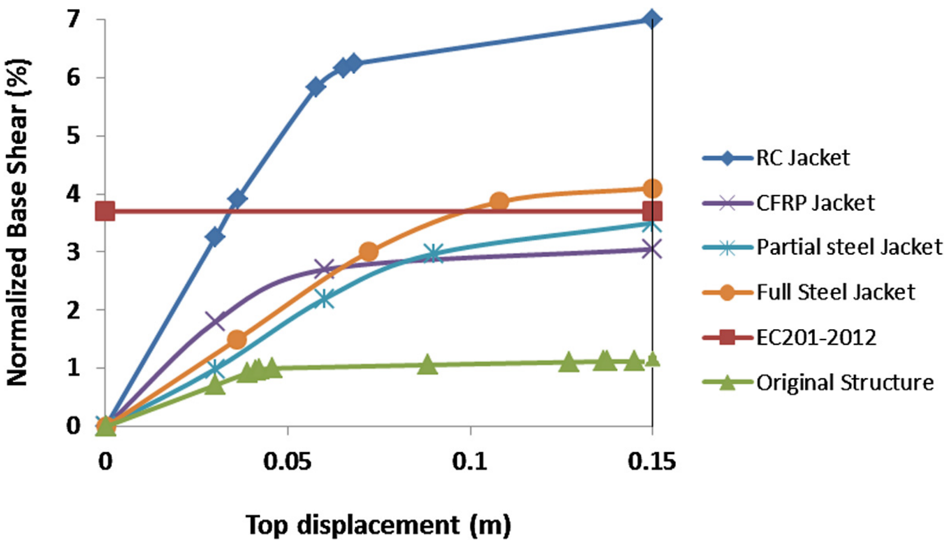


Figure 2.16 Normalized base shear-to displacements for original and retrofitted structures (Ismail, 2013)

Kim et al. (2012) investigated the reinforced concrete structures under the effects of earthquake before or after retrofitting by fiber reinforcing technique. Dynamic tests were performed to evaluate the collapse procedure. In their study, the columns were reinforced with polyester fiber belts as shown in Figure 2.17. It was found that such retrofitting technique was very operational in restraining the column and checking the developed cracks, therefore; adapting the disappointment type of the RC columns from brittle shear failure after flexural creation to flexural prevailing behavior.

Figure 2.17 Column reinforcing process (Kim et al., 2012)

Sahoo and Rai (2013) examined binary strengthening methods to enhance the seismic enactment of the existing non-ductile reinforced concrete frames using the soft story at the ground level. The principal method the column retrofitted with steel jacketing and the second system as full retrofitted applying aluminum shear links complemented energy dissipation and restrained ground story columns. The performance of existing and restrained frames was evaluated by executing nonlinear static and dynamic nonlinear analyses. Their study showed that non-ductile RC frames with soft story at the ground level had inadequate seismic performance because of their insufficient lateral strength, drift capacity, and energy dissipation impending. At the failure mold mechanism, all plastic hinges were molded specially in the ground level columns due to their inadequate shear strengths. The reinforced concrete frames with reinforced ground-story columns showed the advanced the lateral strength 3.6 times that of the RC frame without retrofitting. The reinforced concrete frame with aluminum shear links strengthened the ground story columns established the excellent seismic behavior due to increasing of the lateral strength and energy dissipation. The normal involvement of shear links to the total energy dissipation of the full retrofitted frame was estimated as 76%.

Ruano et al. (2014) performed a study on the reinforced concrete beams with high amount of longitudinal steel and minimum transverse reinforcement so that they showed shear failure. Some of the beams were reinforced with very fluid high strength steel fiber reinforced concrete (SFRC) jacketing and some of them were first established under shear to provide some damage and then they were repaired with the same method. Plain concrete and SFRC were used for the reinforcement. The experimental program presented the option of performance the retrofitting at work place. The result of this study showed that fiber reinforced concrete enhanced structural properties. Furthermore, the compatibility between the base and the retrofitting materials and the extended but thinner cracking pattern, avoided proceed of destructive agents increasing the durability of the reinforcement.

2.2.4 Concrete Jacketing

The concrete jacketing was applied to increase axial, flexural, and shear strength of existing structures, improves in stiffness and ductility were similarly succeeded. Jacketing was achieved by applying transverse and longitudinal reinforcement or a joined wire mesh welded together and adjacent the exist section and shell it with original cast in place concrete or with shotcrete. Surface roughening of exist section was achieved by sandblasting or by mechanical resources to advance uniform behavior of the elements. In common, columns are observed as the most critical structural members to be rehabilitated, since the failure of columns may lead to collapse (Moehle, 2000).

2.3 Nonlinear Analysis

Nonlinear analysis used in the design of the building structures to motivate the effects of the earthquakes for determining forces and deformations. FEMA356 specifies two analytical nonlinear procedures for a structure performance evaluating (Mwafy and Elnashai, 2001).

- Nonlinear Static Analysis (NSA)
- Nonlinear Dynamic Analysis (NDA)

2.3.1 Nonlinear Static Analysis (Pushover Analysis)

The pushover analysis of a structure is a static non-linear analysis in which significance of the lateral loads incrementally increased so as to the effects of nonlinear such as failure mechanism generated. For seismic performance assessment of new and existing structures, the pushover nonlinear static analysis has become general implement. The pushover static analysis can offer adequate information on seismic loads establish by the design ground motion on the structural system and its components. The building pushed to certain target displacement, gravity load and then pushover load cases are designated. The purpose of pushover analysis is to estimate the predictable performance of structural systems by evaluating performance of a structural system by calculating its strength and deformation demands in design earthquakes by means of static unyielding analysis. The nonlinear static pushover analysis can be stand-pointed as a technique for foreseeing seismic force and deformation levels, which consider in an estimated method for the rearrangement of internal forces that cannot be extra resist in elastic behavior. In the Figure 2.18, a capacity curve of a structure is illustrated (SERMİN OĞUZ, 2005).

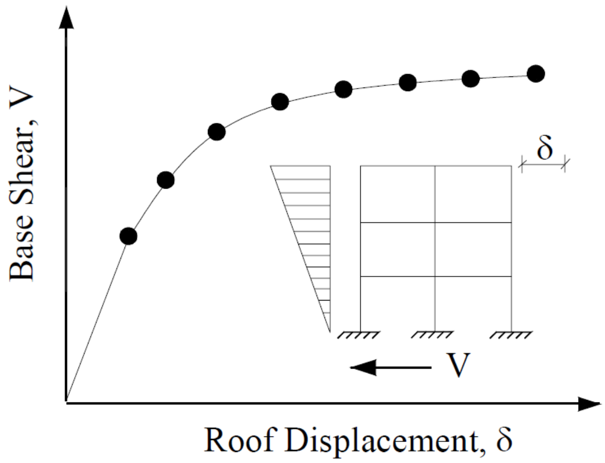


Figure 2.18 Global capacity (pushover) curve of a structure (SERMİN OĞUZ, 2005)

The basic point in the presentation of the pushover analysis is the significance of the objective displacement at which seismic performance calculation of the structure is to be observed. The target displacement satisfies as an approximate evaluation of the global displacement of the structure is predictable to experience in a design earthquake. It is the roof displacement at the center of mass of the structure. The building subjected to incrementally increasing lateral loads an earthquake until target displacement according to FEMA356 can be calculated in equation (2.1) (Krawinkler and Seneviratna, 1998).

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} \tag{2.1}$$

On the other hand, the nonlinear procedure of FEMA-356 requires the definition of the nonlinear load –deformation relation for pushover analysis, as shown in Figure

2.19. The points (A, B, C, D and E) are used to define hinge deformation behavior of the reinforced concrete elements according to FEMA-356 in which describe the state of the member and to define material nonlinearity. ASCE 41 (ASCE, 2007) and other standards commonly define three performance levels for building structure elements so as to describe states during earthquakes (Inel and Ozmen, 2006). They also explained the points on the Figure 2.19 as given below:

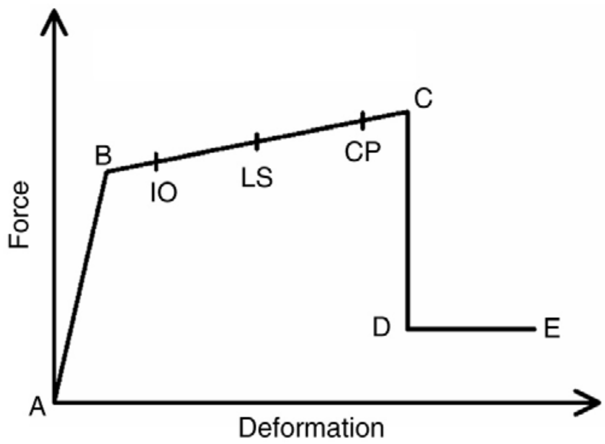


Figure 2.19 Typical Load-Deformation relations (Inel and & Ozmen, 2006)

- Point (A) represents the unloaded condition,
- Point (B) represents nominal steel yield strength,
- Point (C) represents the strength of the component or maximum force,
- Line (AB) represents the elastic state,
- Line (BC) represents strain hardening and the slope (BC) is usually taken zero to ten percentage of the elastic slope (AB),
- Line (CD) represents the initial failure of the member such as reinforcement bending failure, crushed concrete or shear failure,
- Line (DE) represents the residual strength of the member,
- Immediate Occupancy (IO) performs basically elastic behavior by specifying structure damage such as significant cracking of concrete, steel yielding and nonstructural destruction,

- Life Safety (LS) specifies destruction of structural and nonstructural elements to minimize the hazard of injury and to remain necessary circulation routs available, and

- Collapse Prevention (CP) confirms small damage of partial or complete building collapse limiting structural deformations and forces to the commencement of significant strength and stiffness deterioration.

Sharma et al. (2013) presented numerical and experimental work performed on a full-scale four story reinforced concrete structure using pushover analyses for seismic evaluation and to obtain realistic predictions. It was revealed that the basic pushover analysis considering only flexural failure might be experienced practical simulation. For this purpose an existing reinforce concrete building was selecting for examining with geometric shape and material properties as considered for testing. Structure foundation constructed uniform raft foundation for all columns to restrain possible foundation rotation of the structure. Failure mechanism at the joints identified as shown in Figure 2.20.

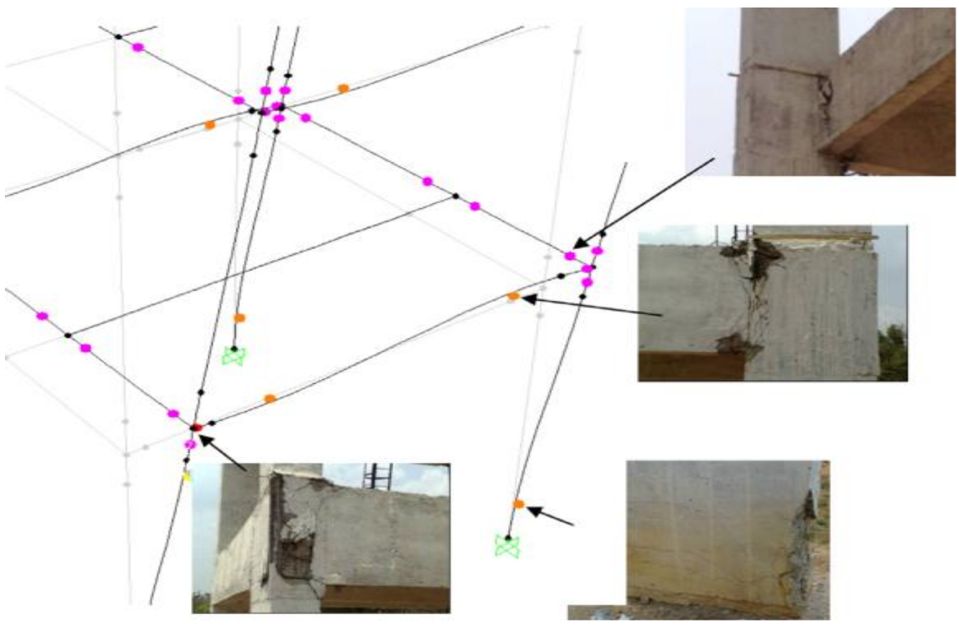


Figure 2.20 Experimental and numerical comparison of failure modes (Sharma et al., 2013)

In their study, both numerical and experimental investigations demonstrated similar results. It was prominent to estimate a predicting of the structure behavior. Applying experimental procedure to assess numerical results to consider complex circumstances such as joint shear failure so as to realistic performance executed.

Belletti et al. (2013) used a regular six-story precast reinforced concrete building by means of pushover analyses represented 9 structural walls of various cross section shapes (U shape, L shape and C shape) vertically joined with ordinary reinforcement. Plasticity model involved that entire elements of the structure persisted elastic deformation and inelastic are established in focused elements when the plastic hinge developed in case of existing shear wall the plastic hinge located base of the wall. Physical method for reinforced concrete typical for the nonlinear behavior analysis up to failure of the reinforced concrete components were exposed to plane stress. The results showed that when the seismic force acted that wall would collapse because of getting of the ultimate strain of the reinforcement bars. Consequently the direction of seismic force was paramount important to be considered.

Krawinkler and Seneviratna (1998) studied the fundamental concepts on which the pushover analysis could be established to evaluate the correctness of pushover predictions and recognize the circumstances under which the pushover would deliver acceptable information and, perchance more significantly, classified cases in which the pushover estimation would be insufficient. Their study was disturbed only with demand prediction at low performance levels, such as life safety and collapse prevention, at which it was predicted that the structure would have to experience important plastic deformations, as shown in Figure 2.21. According to the result, it was identified that performed pushover analysis would determine perception into structural conditions that controlled performance during severe earthquakes and pushover analysis could accurate predict local and global demands. This analysis would also show design vulnerability that might continue hidden in an elastic analysis. For structures that vibrate primarily in the essential mode, the pushover analysis would very expect and provide good estimations of overall, in addition to local inelastic deformation loads.

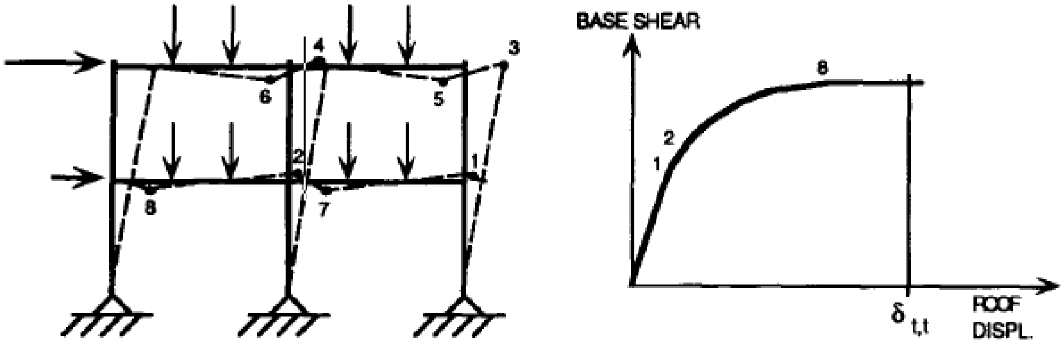


Figure 2.21 Illustration of a pushover analysis (Krawinkler and Seneviratna, 1998)

Rana et al. (2004) conducted a study on 19 story concrete shear wall building. Lateral system of the building included of concrete shear walls. The building was designed according to 1997 Uniform Building Code. The life safety performance confirmed using pushover analysis. The researchers showed in their study that the pushover analysis was an effective method of performance based seismic engineering to consider inelastic behavior of structures, however it was more complex than traditional elastic analysis, but it was need less amount of data than nonlinear response history, also pushover analysis requires inferior application. The results of this analysis on 19 story concrete building showed some modifications were made to the original code-base design so as to predict life safety performance under earthquake design.

2.3.2 Nonlinear Dynamic (Time History) Analysis

The time history method is a convenient way to calculate the building response at individual time steps using simulated time histories as base motion. The damping matrix related with the exact model shall reflect the damping in the building at deformation levels near the yield deformation. In nonlinear dynamic analysis, load and output are changing with time. Nonlinear analysis of a reinforced concrete building is difficult because inelastic deformation is not limited at critical sections, but extends throughout the building structure. The structure is defined by its stiffness and mass. The nonlinear dynamic procedure (NDP) shall be allowed for all structures. An analysis performed using the NDP shall be reconsidered and approved by an independent third-party engineer with experience in seismic design and nonlinear procedures. Earthquakes can be caused the dynamic response of a structure. Dynamic characteristics up to failure cannot be determined only through a dynamic test or a real structure since it is difficult to understand the behavior due to complex interactions of various parameters (Inel and Ozmen, 2006).

Inel and Ozmen (2006) applied the interior frames of 4story, and 7story buildings to show low and medium rise reinforced concrete building nonlinearities. Nonlinear time history and pushover analyses were performed to determine nonlinear behavior of the reinforced concrete frames. Columns and beams were formed with default and user-defined hinge properties so as to determine the difference between them in analyses which observed at displacement points equivalent to global yielding and ultimate displacement. The researchers found that default hinges and user-defined hinges for different plastic hinge length and transverse reinforcement spacing for two models had similar base shear capacity. It implied that the base shear capacity didn't depend on whether the default hinge or user-defined hinges were used. The displacement capacity of the frames influenced by the plastic hinge length, about 30% variation in displacement because of plastic hinge length, the displacement capacity depended on amount of transverse reinforcement at the possible hinge part; therefore, increasing amount of transverse reinforcement enhanced the displacement capacity. Reducing the transverse reinforcement spacing from 200 mm to 100 mm, the displacement capacity increased up to 40%, at the same time as reducing the spacing from 200 mm to 150 mm gave an increase of 12% for 4 story building frame. The study showed that the user-define hinge model was better than default-hinge model for indicating nonlinear behavior. Time-history results showed that the pushover analysis was practically successful in capturing hinging patterns for low and medium-rise buildings, except that the plastic hinge formation in the upper levels was not evaluated sufficiently by pushover analysis.

El-Sokkary and Galal (2009) studied analytically the effectiveness of different rehabilitation patterns in evaluating the seismic performance of existing non-ductile reinforced concrete frame structures using the dynamic analysis. They investigated the performance of the reinforced concrete frames in different height indicating low and high-rise buildings subjected to three variety ground motion records. The ground motion records contained low, medium, and high frequency contents. They considered three models for reinforced concrete frames; bare frame, masonry in-filled frame with soft infill, and masonry-infill frame with stiff frame. The seismic performance improvement of the studied frames estimated in terms of the maximum applied peak ground acceleration resist by the frames. Four rehabilitation patterns were studied: (1) RC shear wall, (2) steel bracing, (3) diagonal strips in case of masonry–in filled frames, (4) wrapping or partially wrapping the frame members (columns and beams). They observed that the dynamic properties of low or high-rise buildings influenced by the earthquake properties.

CHAPTER 3

CASE STUDY

3.1. General

In the study, an existing reinforced concrete (RC) building was taken into consideration. First of all, the RC building was assessed to show the nonlinear behavior and performance state of the structural system. Secondly, the existing building (EB) was retrofitted by adding shear walls or concentric steel bracings. As a concentric bracing, X, inverted-V, and diagonal bracing systems were used. After all the pushover analysis and nonlinear time history analysis were performed to evaluate the seismic behavior of the existing reinforced concrete frame building with and without shear wall and concentric steel bracings.

3.2. Description of buildings

In this investigation, the effectiveness and performance of using reinforced concrete (RC) shear wall and steel bracings in upgrading (RC) structures against seismic loads were studied. For this purpose, the 9 story RC frame system having the similar height of 3.6 m at each story with 5 bays in both X-direction and Y-direction of different spacing was studied. Five cases of the building were evaluated; case 1: RC building frames that taken as existing building (EB), case 2: existing RC building retrofitted by shear walls (RB-SW), case 3: existing building retrofitted by X-steel bracing (RB-XB), case 4: existing building retrofitted by inverted V-steel bracing (RB-IVB), and case 5: existing building retrofitted by diagonal-steel bracing (RB-DB). The plan and 3D view of the existing building are given in Figures 3.1 and 3.2, respectively while the frame elevation of the existing building is shown in Figure 3.3. For retrofitted cases, the location of the shear walls and steel bracings are revealed in Figures 3.4 and 3.5, respectively. Moreover, in Figures 3.6-3.9, the 3D views of the cases of RB-SW, RB-XB, RB-IVB, and RB-DB are given, respectively.

The sections of the frame elements such as column and beam were taken as square and rectangular sections, respectively. Pipe hollow steel sections were utilized with standard cross sectional dimensions and the details of the cross-sectional area are showed in Table 3.1. The shear walls having the same thicknesses were inserted to each floor level and the steel braces were used in the same bay of the shear walls after removing them in other case.

Table 3.1 Section properties of the frame systems

Story No.	Reinforced Concrete Members						Steel Section	
	Dimension (mm)						Dimension (mm)	
	Column		Beam		Slab	Shear wall	HSS323.9X12.7	
	Length	Width	Width	Height		Thickness	Inside Dia.	Thickness
1	550	550	300	600	150	25	324	12.7
2	550	550	300	600	150	25	324	12.7
3	550	550	300	600	150	25	324	12.7
4	500	500	300	600	150	25	324	12.7
5	500	500	300	600	150	25	324	12.7
6	500	500	300	600	150	25	324	12.7
7	450	450	300	600	150	25	324	12.7
8	450	450	300	600	150	25	324	12.7
9	450	450	300	600	150	25	324	12.7

The concrete with compressive strength of 16 MPa was used for all the RC frame structures, the modulus of elasticity and yield stress of the steel bar were 200 GPa and 420 MPa, respectively. The elastic modulus and yield stress of the bracing steel frames were 200 GPa and 350 MPa, respectively.

Along with these, all of the analytical models were evaluated according to the requirements of TS500 and Turkish seismic codes for concrete design and AISC 360-10 for steel design. During this design development, ETABS2013 was utilized which has standard usage in the design of reinforced concrete building because of its feasibility. The 3D and 2D view of the existing building is given in Figures 3.2 and 3.3, respectively. The location of the shear walls and concentric braces in plan and in the 3D view of the structures are given in the Figures 3.4 to 3.9.

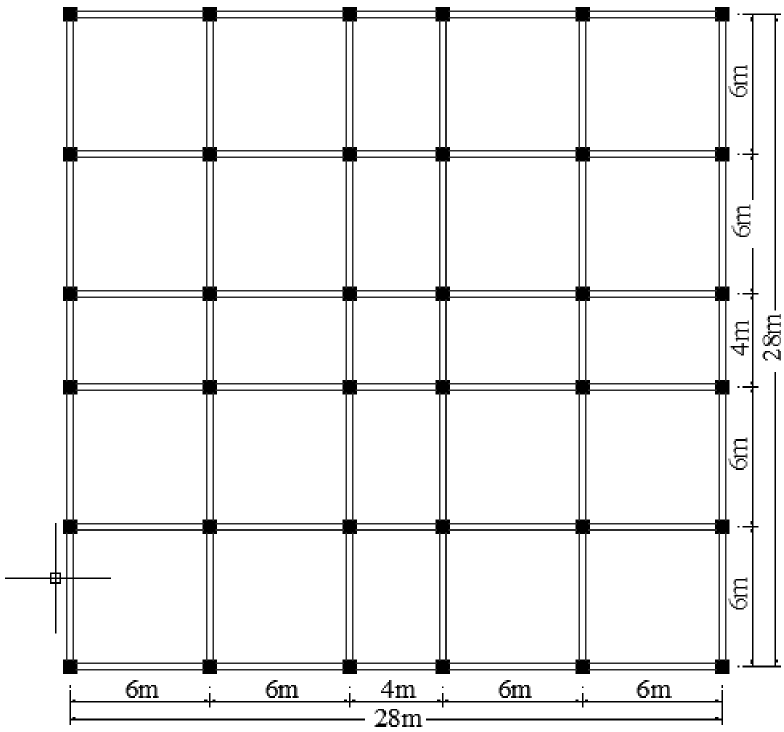


Figure 3.1 Plan view of an existing- nine story reinforced concrete building

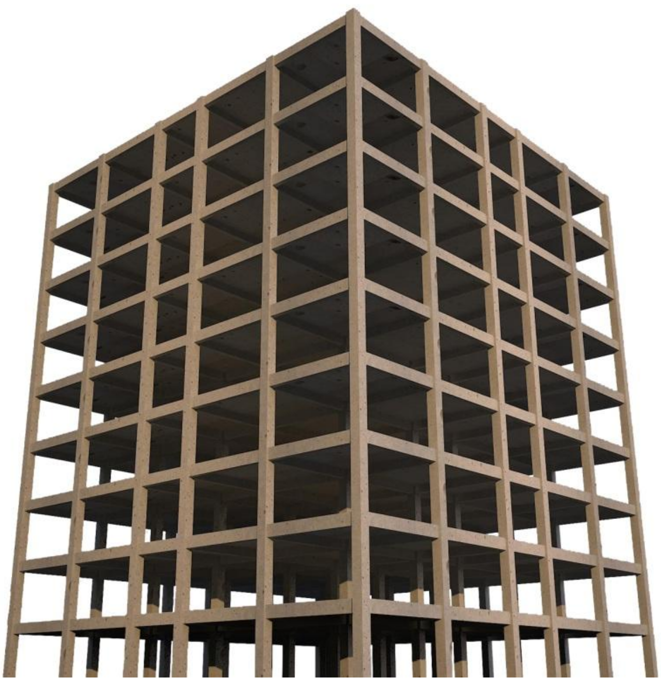


Figure 3.2 3D view of an existing- nine story reinforced concrete building frame

To examine the nonlinear behavior of the building structures, several three-dimensional analytical models were investigated in this study. This chapter demonstrated these analytical models in detail.

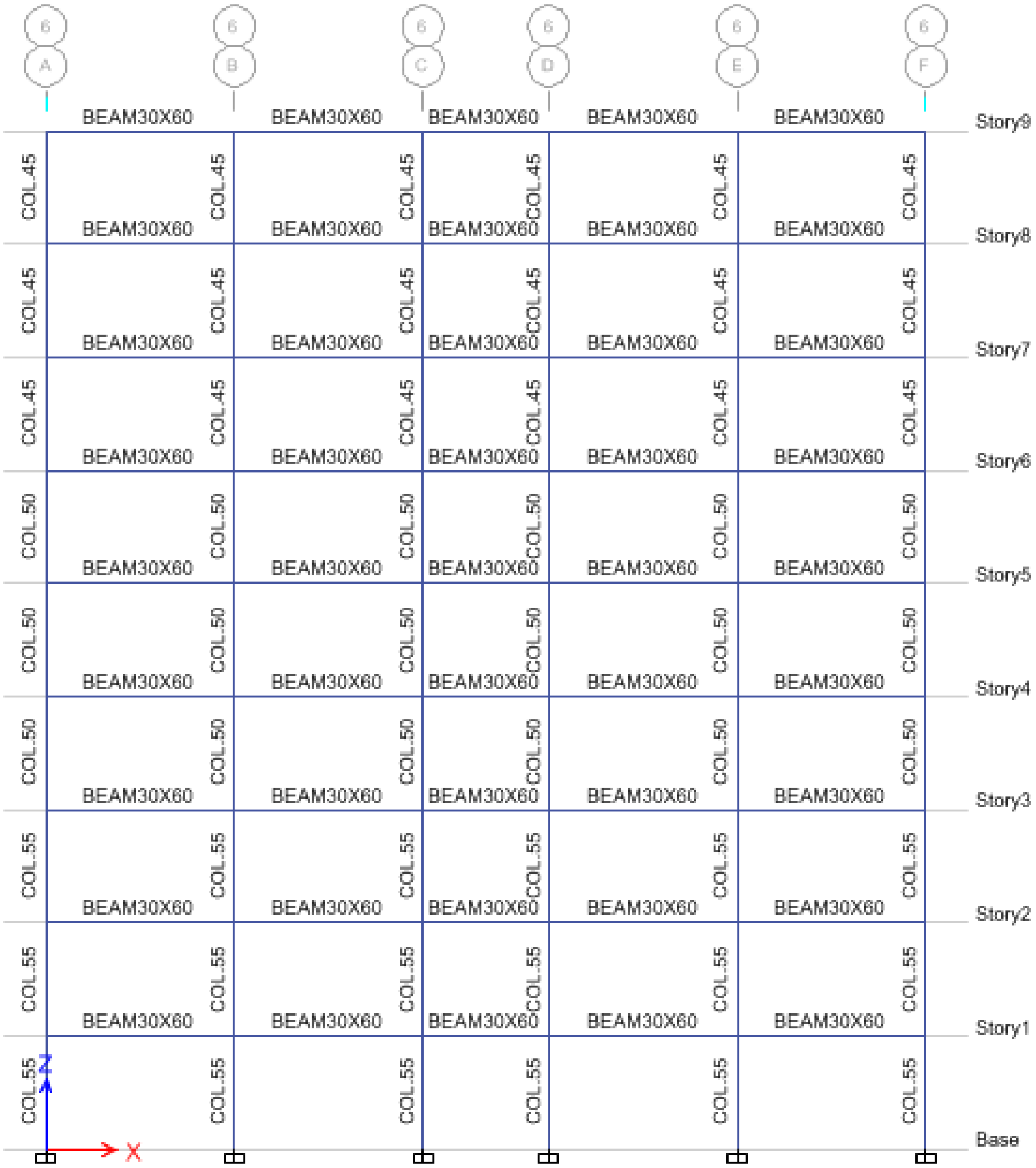


Figure 3.3 Frame elevation of an existing reinforced concrete building

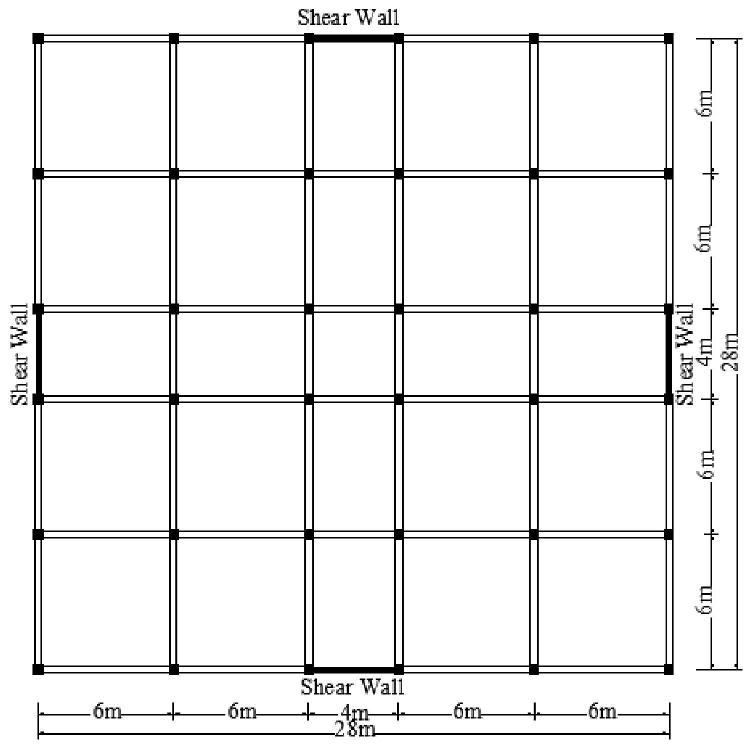


Figure 3.4 Plan view of the retrofitted RC building with shear walls

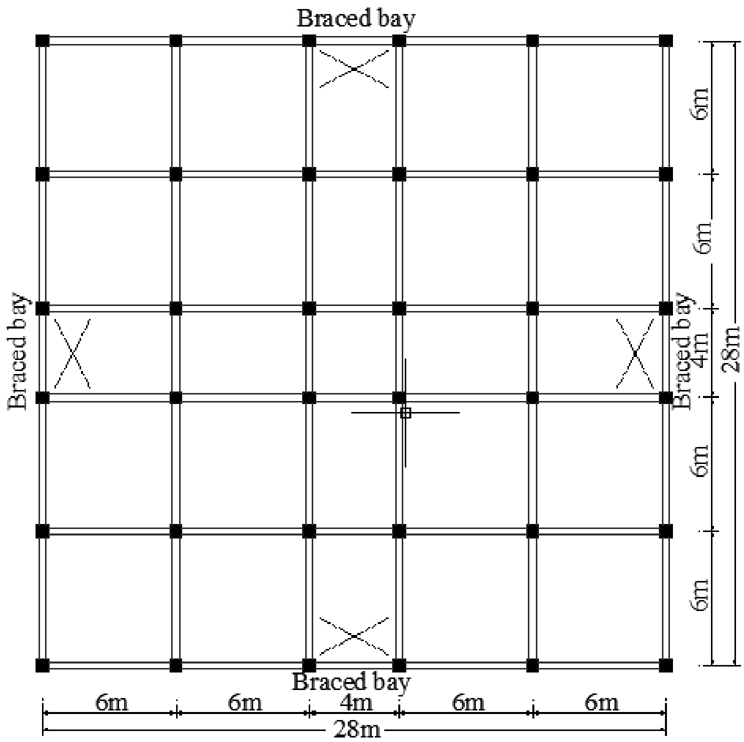


Figure 3.5 Plan view of the retrofitted RC building with steel bracing

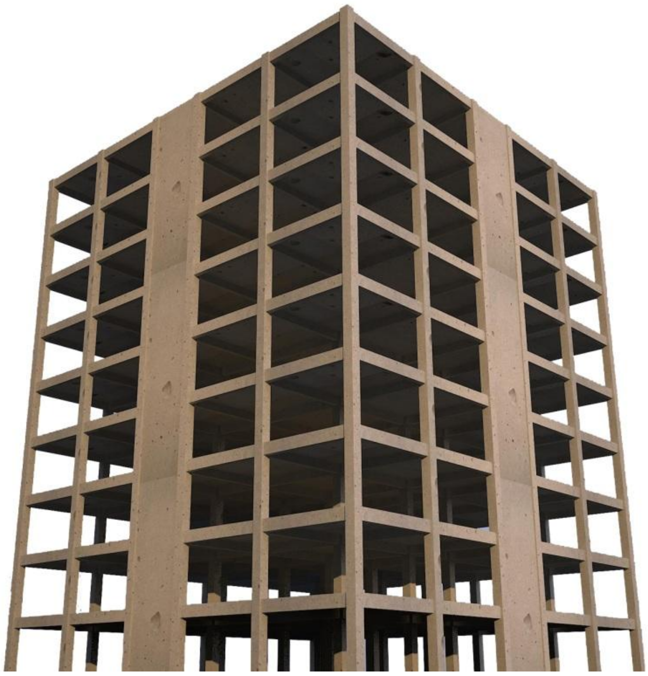


Figure 3.6 3D view of an existing-nine story building retrofitted by shear wall (RB-SW)

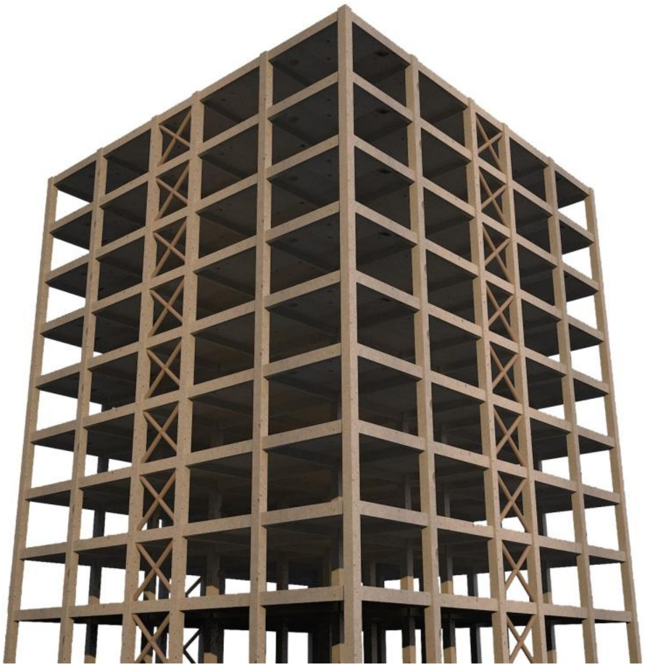


Figure 3.7 3D view of an existing-nine story building retrofitted by X-bracings (RB-XB)

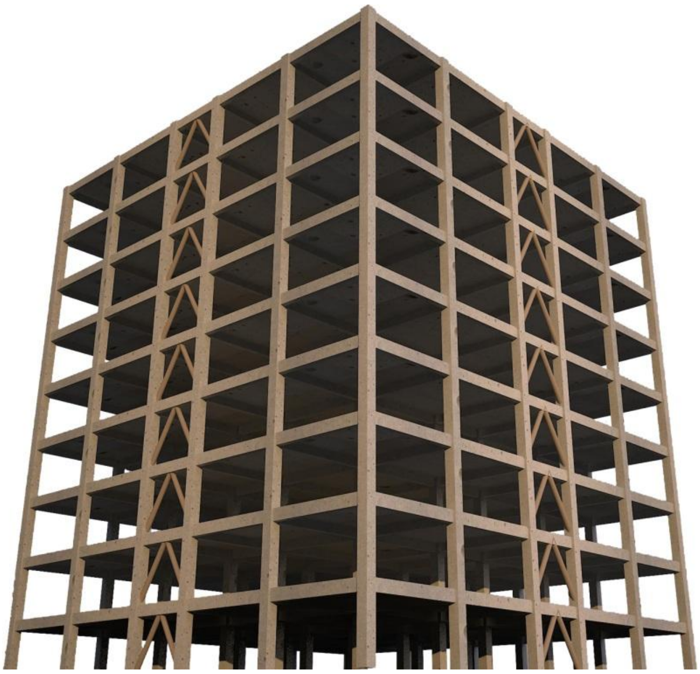


Figure 3.8 3D view of an existing-nine story building retrofitted by inverted V-bracings (RB-IVB)

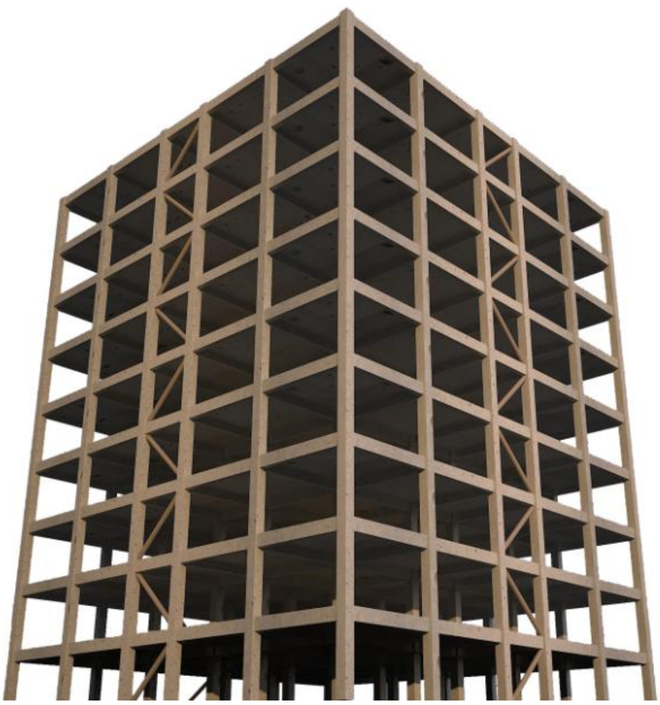


Figure 3.9 3D view of an existing-nine story building retrofitted by Diagonal-bracings (RB-DB)

3.3. Pushover and time history methodologies

Pushover analysis is a nonlinear static procedure in which the amount of the lateral force is incrementally increased, maintaining the predefined distribution pattern along the height of the building (Krawinkler and Seneviratna, 1998). Nonlinear properties were modeled and the structure was pushed until a collapse mechanism established. The base shear and the roof displacement could intend to generate the pushover curve. It provided an indication of the maximum base shear that the structure was capable of resisting at the time of the earthquake. For regular buildings, it could provide a rough idea about the global stiffness of the building. The ATC-40 and FEMA-356 documents have established modeling techniques, analysis procedures, and acceptance criteria for pushover analysis. These documents describe force-deformation criteria for hinges used in pushover analysis described the acceptance criteria depending on the plastic hinge rotations by considering different performance levels. Each plastic hinge is designed as a separate point hinge. All plastic deformation is displacement or rotation, develops within the point hinge. Five points considered A, B, C, D, and E describe the force deflection behavior of the hinge and three points termed IO, LS, and CP represented immediate occupancy, life safety, and collapse prevention, respectively. The principles for assigning values for each of these points differ liable on the form and type of member such many other parameters defined in the ATC-40 and FEMA-356 documents (Inel and Ozmen, 2006). In this study, the steps in performing the static nonlinear pushover analysis of a nine-story three-dimensional building model in ETABS2013 as tool for performing the pushover analysis procedure as follow:

- The properties of nine-story existing building were created and assigned in ETABS2013, and then properties and acceptance criteria for the pushover hinges were defined. The program included several built-in default hinge properties that were established on average values from ATC-40, FEMA-356, and ASCE 41 (ASCE 2007) for concrete and steel members.
- The pushover hinges on the model was localized by assigning two hinges to each columns and beams with deformation properties based on an assumed hinge length.

- The pushover load case was defined using gravity load and then subsequent lateral pushover load cases were specified to start from the final conditions of the gravity pushover. ETABS2013 permitted the distribution of lateral force used in the pushover to be based on a uniform acceleration in a specified direction, a specified mode shape, or a user-defined static load case.

- From this analysis, the capacity curves and performance level of each structure were established.

In addition to the pushover analysis, the nonlinear time history analysis was performed on the same analytical models with the same hinge properties so as to estimate the actual nonlinear behavior of the structure systems. In the dynamic analysis, 1999 Hector Mine was used as a ground motion (PEER, 2011). The properties of the selected earthquake acceleration record used in this study are given in Table 3.2. Figure 3.10 shows the acceleration time plot of the earthquake while Figure 3.11 shows the acceleration response spectrum of the earthquake used. From this analysis, displacement and drift variation with story level and displacement-time history plots were obtained.

Table 3.2 Properties of selected natural ground motion

Earthquake Record	Year	Magnitude (Mw)	Mechanism	Distance (km)	Vs30 (m/s)	PGA (g)	PGV (cm/s)	PGD (cm)
Hector Mine	1999	7.13	Strike-Slip	195.9	294.2	0.5104	100.222	146.827

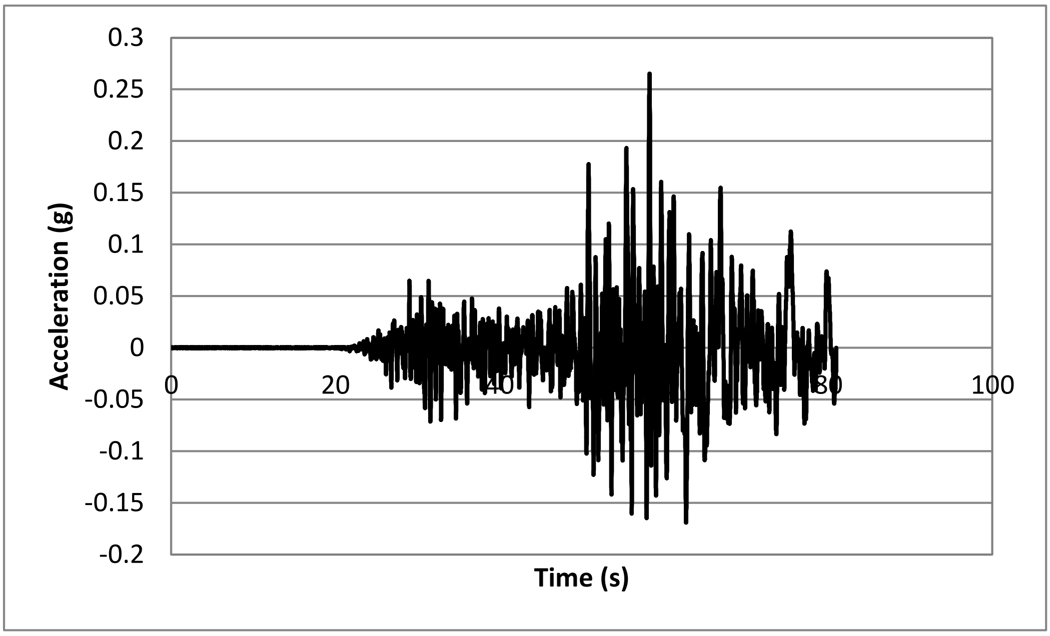


Figure 3.10 Acceleration time plot of the earthquake ground motion

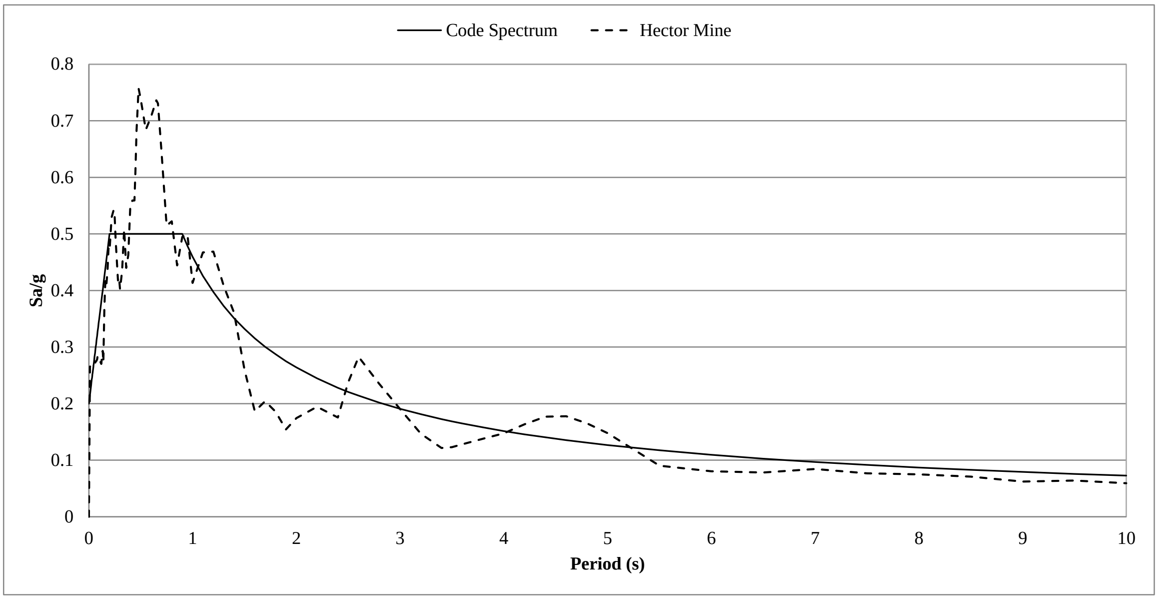


Figure 3.11 Acceleration response spectrum of selected earthquake ground motion

CHAPTER 4

RESULTS AND DISCUSSION

In this study, the nonlinear static pushover analysis and nonlinear time history analysis were carried out using ETABS2013. The existing building (EB) and retrofitted buildings with shear walls (RB-SW), X-bracing (RB-XB), inverted-V bracing (RB-IVB), and diagonal bracing (RB-DB) were analyzed. From the analysis, capacity curves, interstory drift ratio, story displacements, and time history plots were obtained for each building. The analyses of the results were discussed below.

After retrofitting the existing building (EB), the capacity curves were obtained from the pushover analysis. The performance points as calculated by ETABS2013 were found to be within the limits described by ATC-40 and FEMA-356 guidelines for the proposed design objective of life safety. Table 4.1 shows the performance levels, natural periods of each building, and the results of the global displacement and base shear of the building of different cases at the performance point. The purpose of retrofitting the existing building was to increase the lateral load capacity. According to the results the base shear of the existing building was 9646.7 kN. But when the existing building was retrofitted directly applying shear wall, X-bracing, inverted V-bracing, and diagonal bracing, the base shears increased to 22417.5 kN, 21520.8 kN, 17385.9 kN, and 15601.6 kN, respectively. In addition to the performance level of the existing building was collapse prevention (CP); they were improved to life safety (LS) after it was retrofitted. As also seen in Table 4.1, the first natural period of the existing building is 1.17 s while that of the retrofitted buildings varied between 0.90 and 1.02 s. This indicated that the period of the existing building reduced considerably after retrofitting.

Table 4.1 Performance limit states of RC buildings with and without shear walls and steel bracings

Building	Performance Points		Natural Periods			Performance Level
	Displacement (mm)	Base Shear (kN)	T_1 (s)	T_2 (s)	T_3 (s)	
Existing Building (EB)	287.8	9646.6936	1.17	1.05	0.41	CP
Retrofitted building with shear wall (RB-SW)	298.5	22417.508	0.90	0.68	0.25	LS
Retrofitted building with X-bracing (RB-XB)	347.1	21520.8175	0.98	0.76	0.31	LS
Retrofitted building with inverted V-bracing (RB-IVB)	321.4	17385.9578	0.99	0.77	0.33	LS
Retrofitted building with diagonal bracing (RB-DB)	325.4	15601.6474	1.02	0.80	0.34	LS

Figure 4.1 demonstrates the comparison of the pushover curves of existing building (EB) and retrofitted buildings with shear walls (RB-SW), X-bracing (RB-XB), inverted-V bracing (RB-IVB), and diagonal bracing (RB-DB). In accordance with the results of the analysis, the capacity of the existing building upgraded using shear wall or steel bracings. In the pushover curves, it was showed that the lateral load capacity of the existing building retrofitted with shear wall about 2.32 times greater than the existing building. The lateral load capacity of the existing building retrofitted with X-bracing was increased by 2.23 times. Moreover, in the case of the retrofitted building with inverted-V and diagonal bracing were measured about 1.80 and 1.61 times greater load carrying capacity as compared the existing building, respectively. It was also observed from Figure 4.1 that the initial stiffness of the retrofitted buildings was greater than that of the existing building. For example, the ratio between the initial stiffness of the retrofitted building (RB-SW), to that of the existing building was about 2.

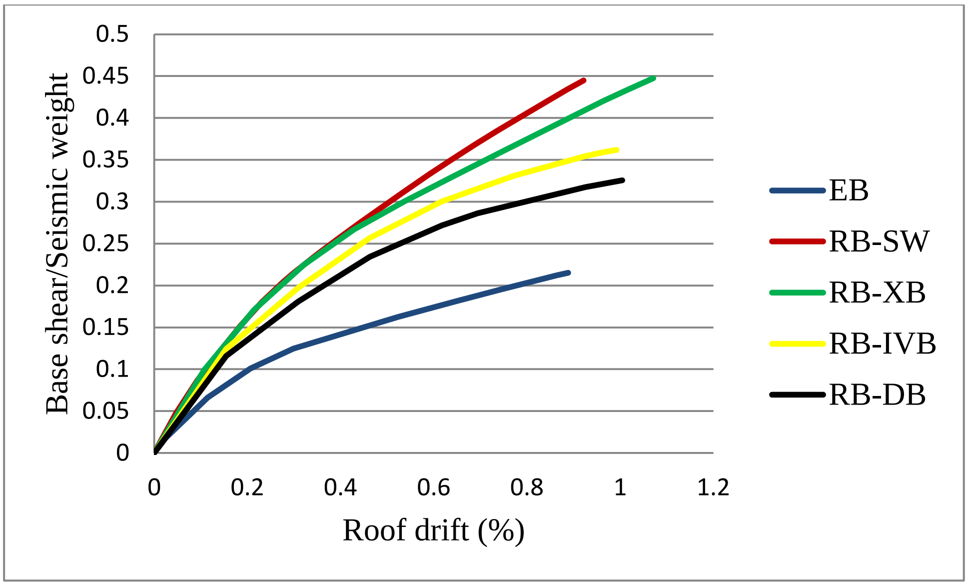


Figure 4.1 Comparison of pushover curves of existing building (EB) and retrofitted buildings with shear walls (RB-SW), X-bracing (RB-XB), inverted-V bracing (RB-IVB),and diagonal bracing (RB-DB)

Figure 4.2 shows the distribution of displacement of the existing buildings with and without of shear walls, X-bracing, inverted-V bracing, and diagonal bracing. According to the results of the plots indicated that the buildings with shear walls (RB-SW), X-bracing (RB-XB), inverted-V bracing (RB-IVB), and diagonal bracing (RB-DB) had considerably lower roof displacement compare to the existing building.

Figure 4.3 illustrates the inter-story drift ratio of the existing and retrofitted buildings. The use of retrofitting techniques had a leaning to distribute the drifts more uniformly along the height of the frames. It was found that the maximum inter-story drift ratio of the existing building (EB) was about 0.82 while this ratio varied for the retrofitted buildings with shear walls (RB-SW), X-bracing (RB-XB), inverted-V bracing (RB-IVB), and diagonal bracing (RB-DB) as about 0.42, 0.44, 0.49, and 0.57, respectively. As a result, the retrofitted buildings demonstrated significantly lower displacement than existing building. Figures 4.4 - 4.12 show the story- displacement time history variation of the existing and retrofitted buildings for different story levels. In the result of the analysis, it was indicated that the story displacement of the existing building in ninth story was 14.29 cm, but it was significantly decreased to 10.40 cm, 10.39 cm, 10.59 cm, and 12.43 cm, by using shear walls (RB-SW), X-bracing (RB-XB), inverted-V bracing (RB-IVB), and diagonal bracing (RB-DB), respectively.

Applying shear wall, X-bracing, inverted V-bracing, and diagonal bracing to the existing building achieved to increase the strength capacity of the existing building as shown in Figure 4.13. The maximum base shear observed in the existing building was 6028 kN, while those observed were 8206 kN, 8806 kN, 9149 kN, and 9042 kN, in the retrofitted buildings with shear walls (RB-SW), X-bracing (RB-XB), inverted-V bracing (RB-IVB), and diagonal bracing (RB-DB), respectively.

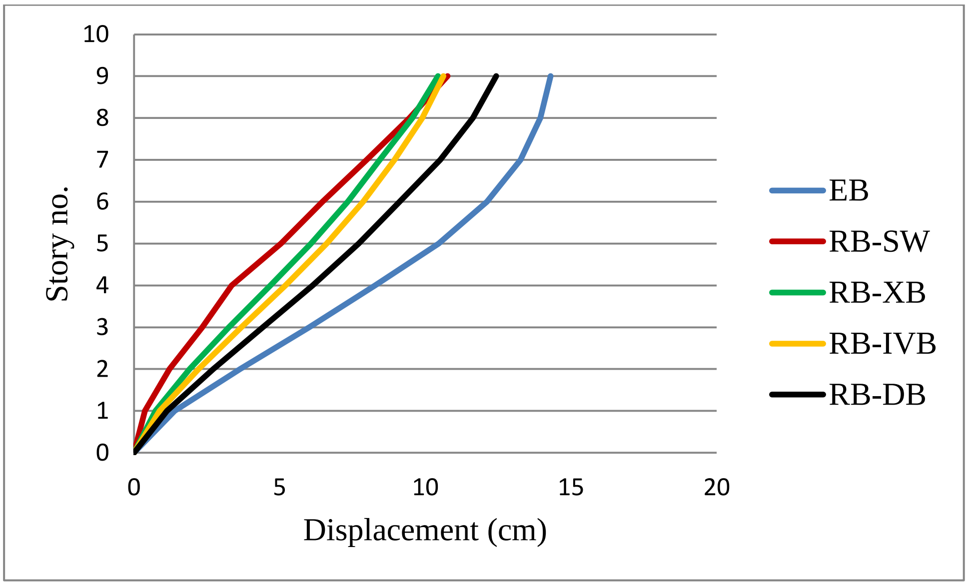


Figure 4.2 Distribution of displacement of the existing and retrofitted buildings with shear walls (RB-SW), X-bracing (RB-XB), inverted-V bracing (RB-IVB), and diagonal bracing (RB-DB)

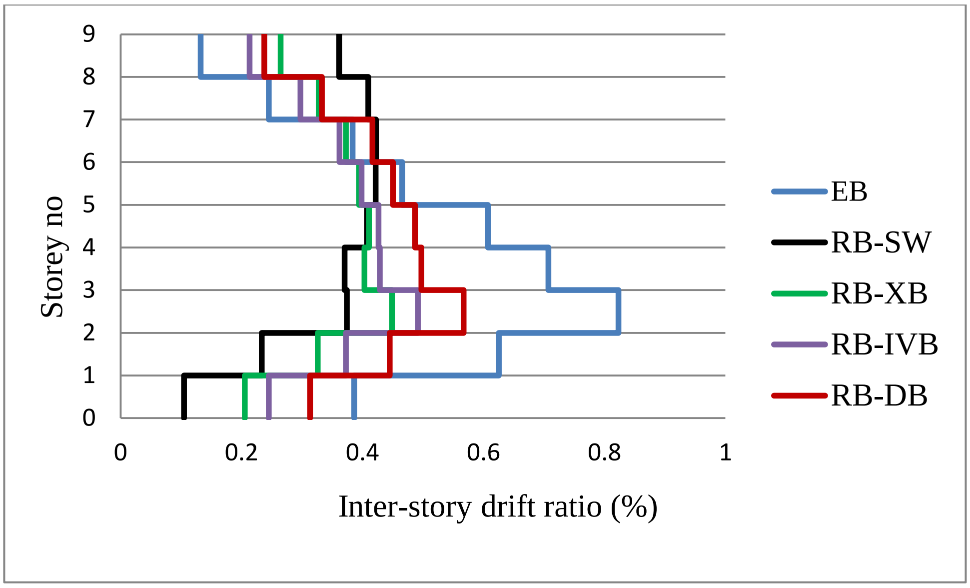
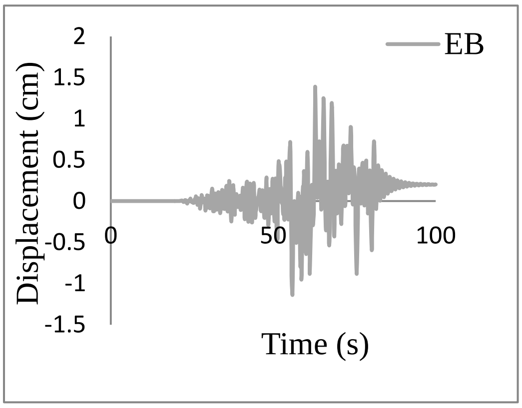


Figure 4.3 Distribution of inter-story drift ratio of the existing and retrofitted buildings with shear walls (RB-SW), X-bracing (RB-XB), inverted-V bracing (RB-IVB), and diagonal bracing (RB-DB)



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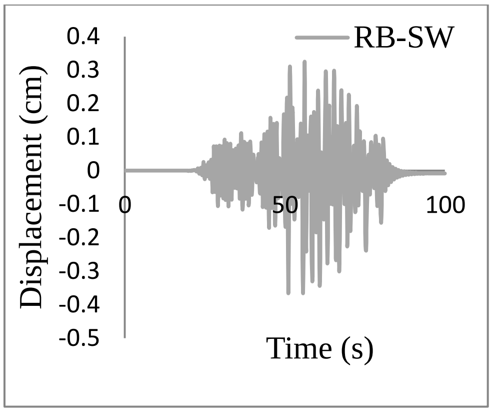

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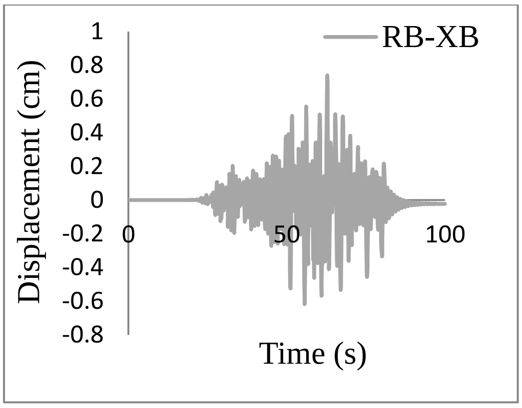

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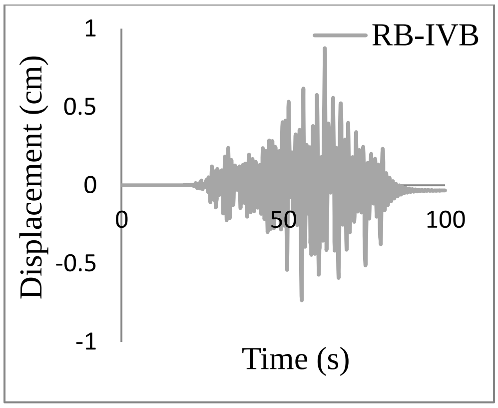

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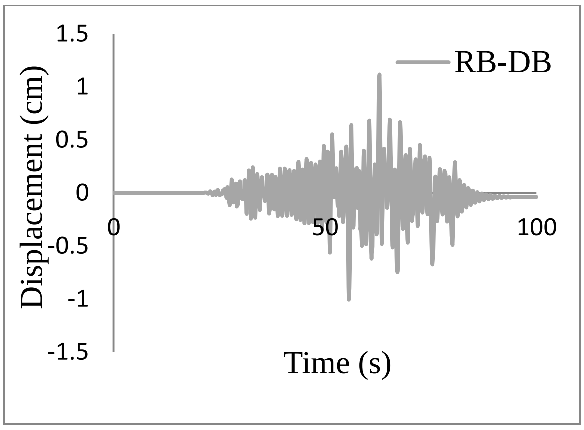

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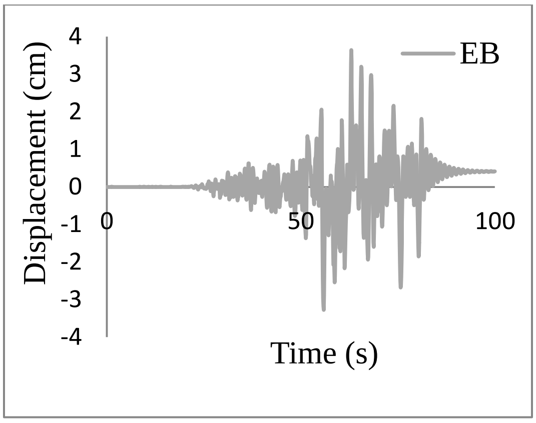
Figure 4.4 First story-displacement time history variations of the existing and retrofitted buildings



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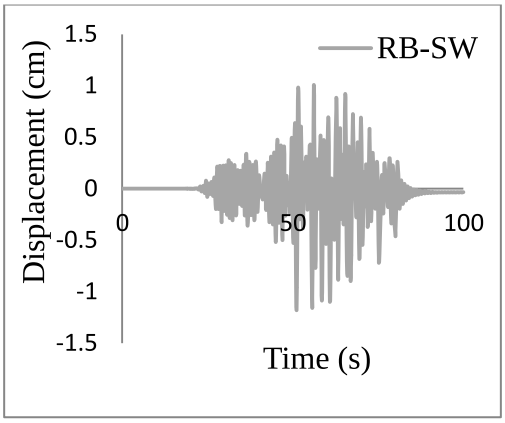

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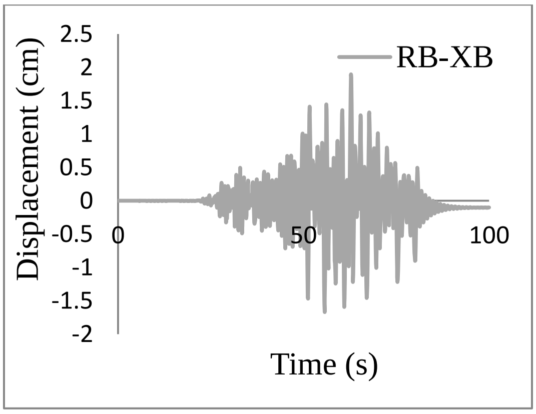

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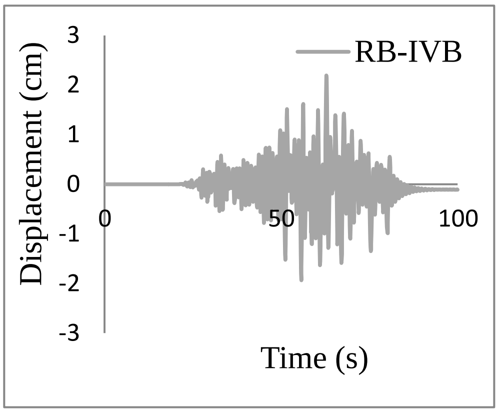

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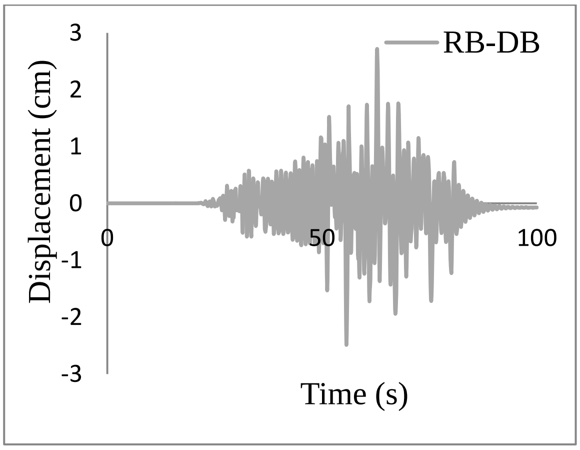

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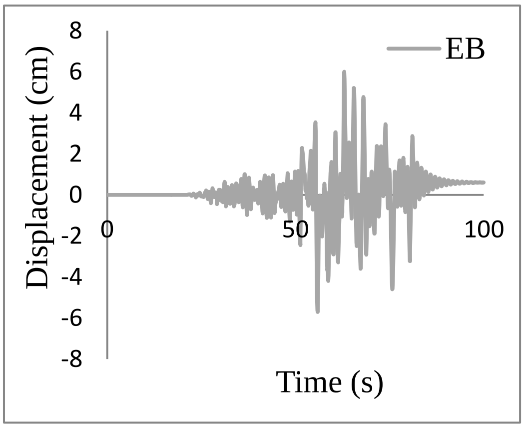
Figure 4.5 Second story-displacement time history variations of the existing and retrofitted buildings



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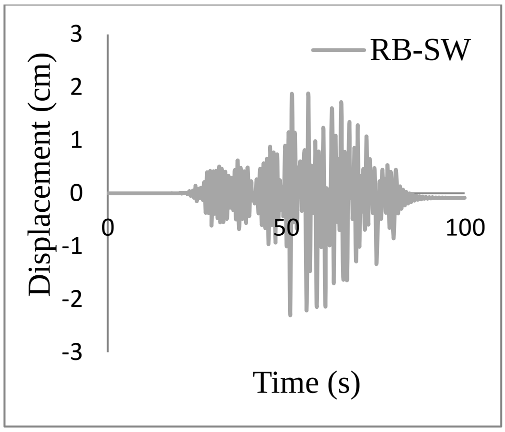

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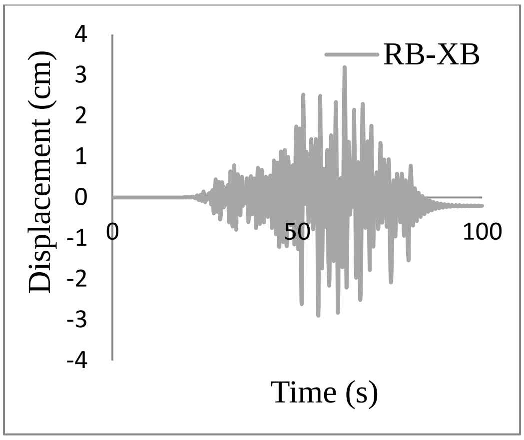

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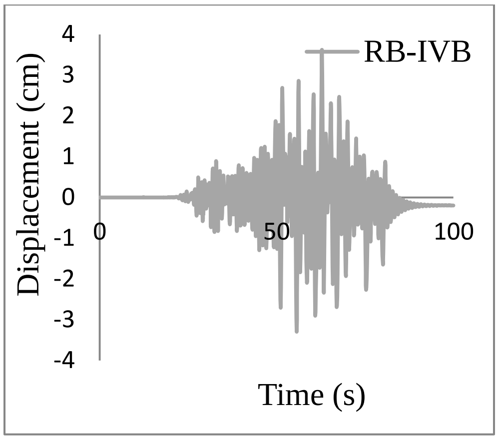

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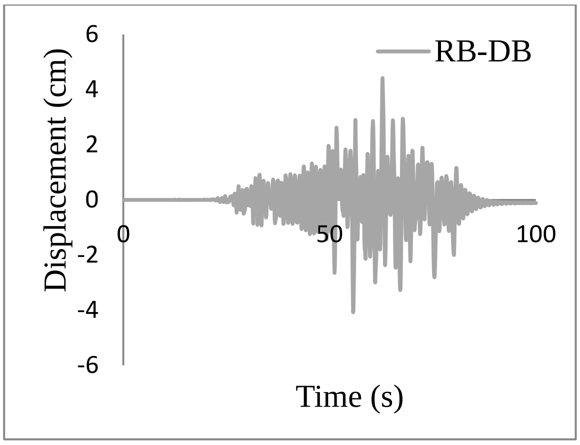

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Figure 4.6 Third story-displacement time history variations of the existing and retrofitted buildings

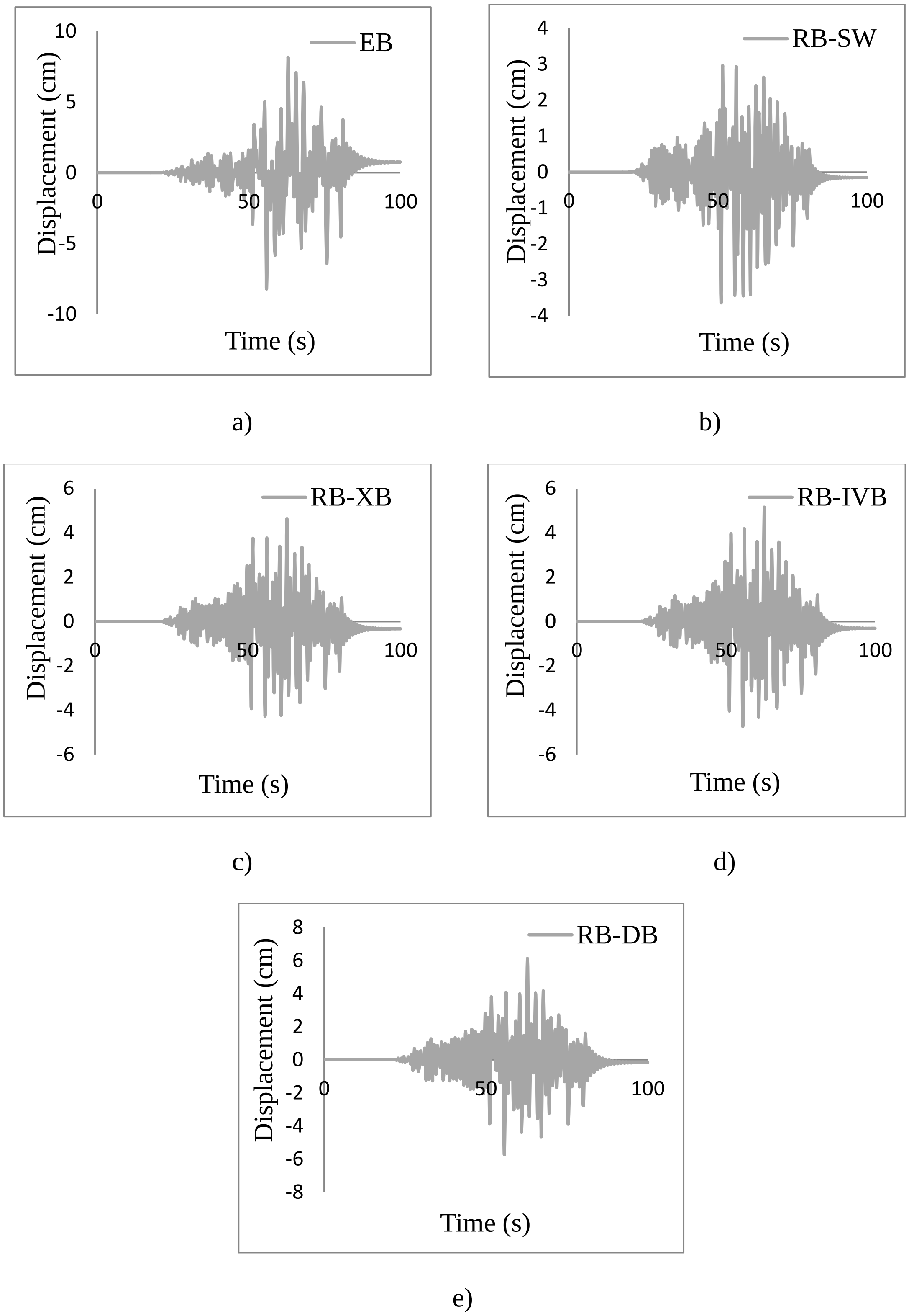
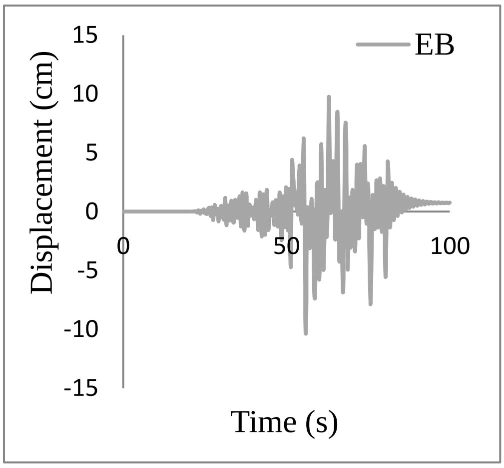


Figure 4.7 Fourth story-displacement time history variations of the existing and retrofitted buildings



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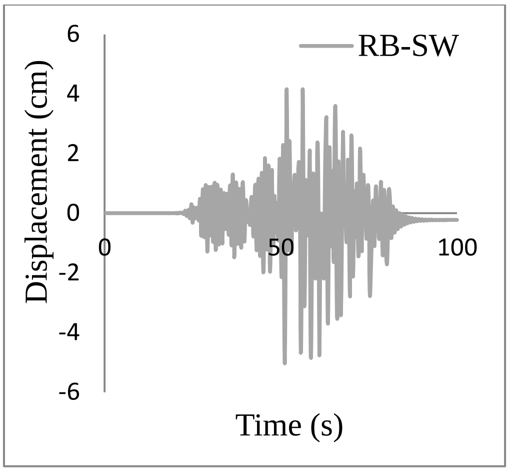

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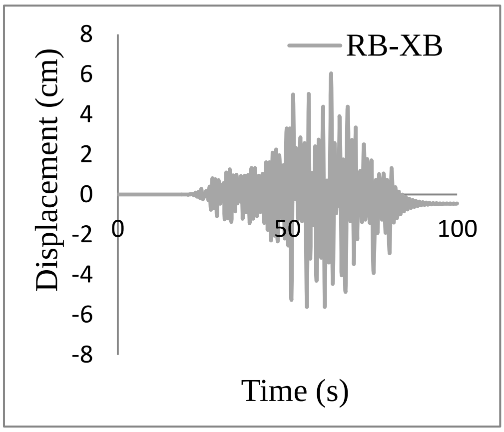

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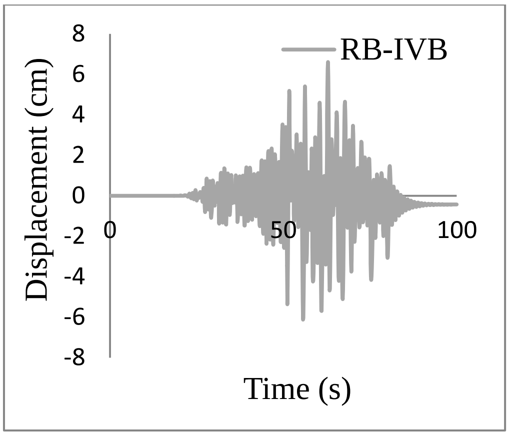

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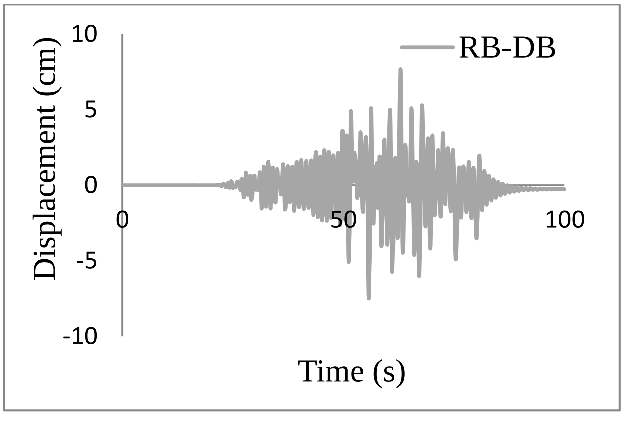

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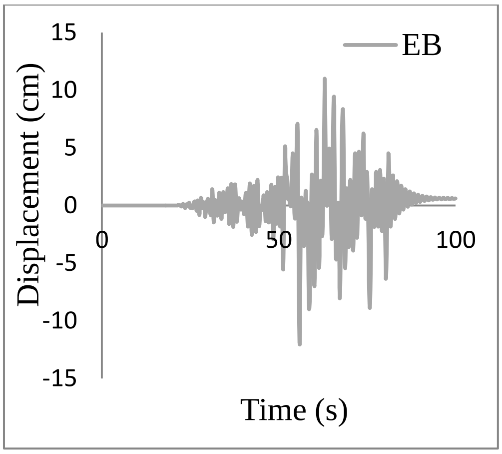
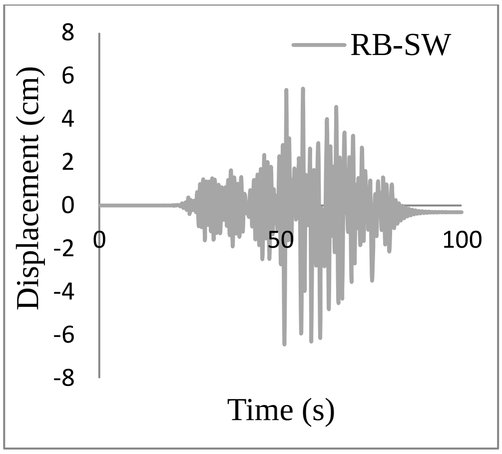
Figure 4.8 Fifth story-displacement time history variations of the existing and retrofitted buildings





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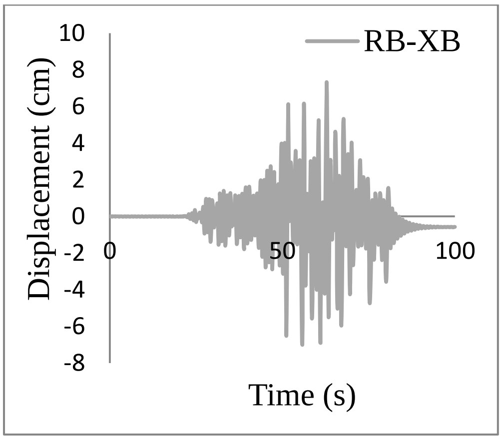

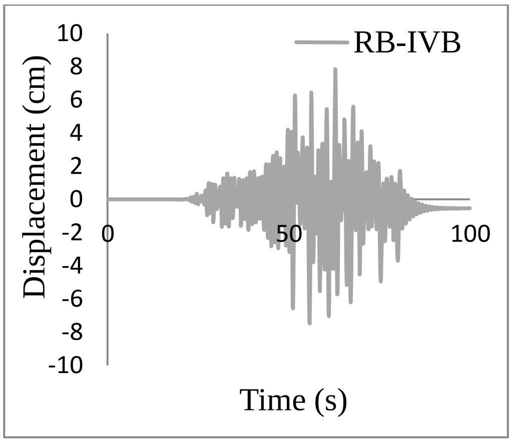

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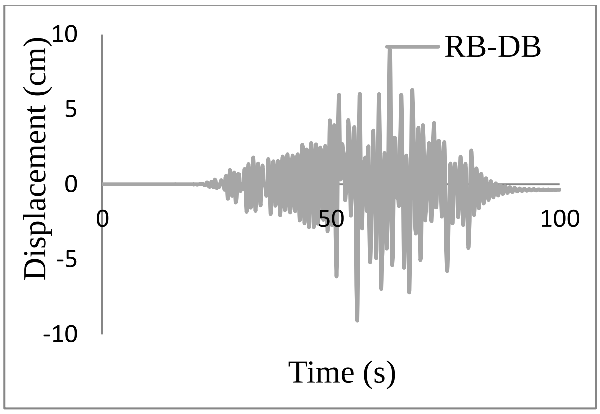

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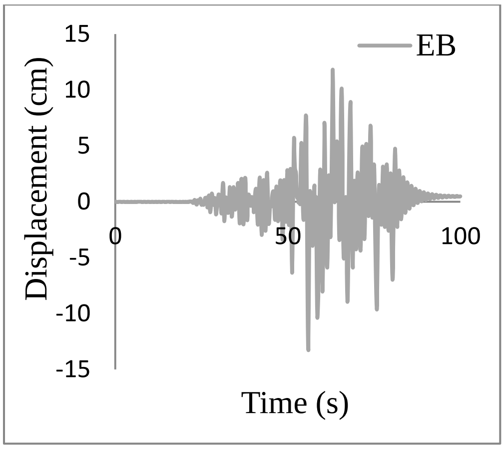
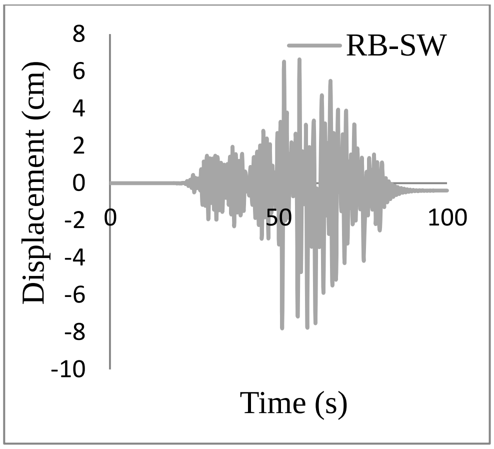
Figure 4.9 Sixth story-displacement time history variations of the existing and retrofitted buildings





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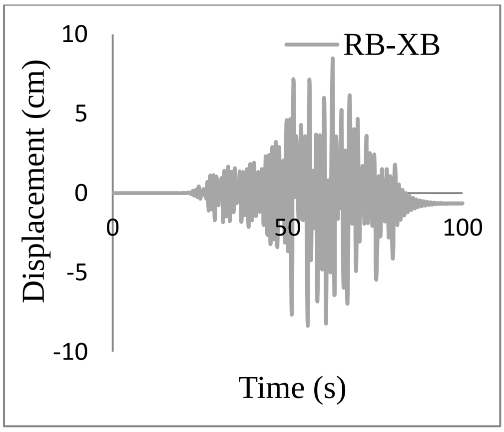

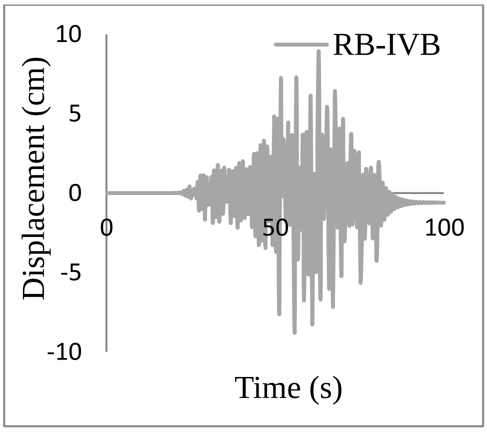

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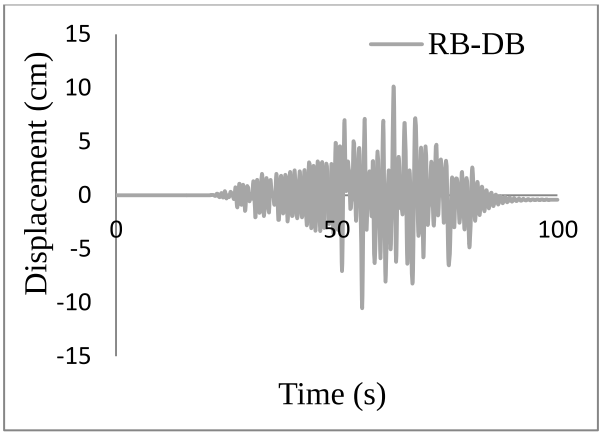

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Figure 4.10 Seventh story-displacement time history variations of the existing and retrofitted buildings

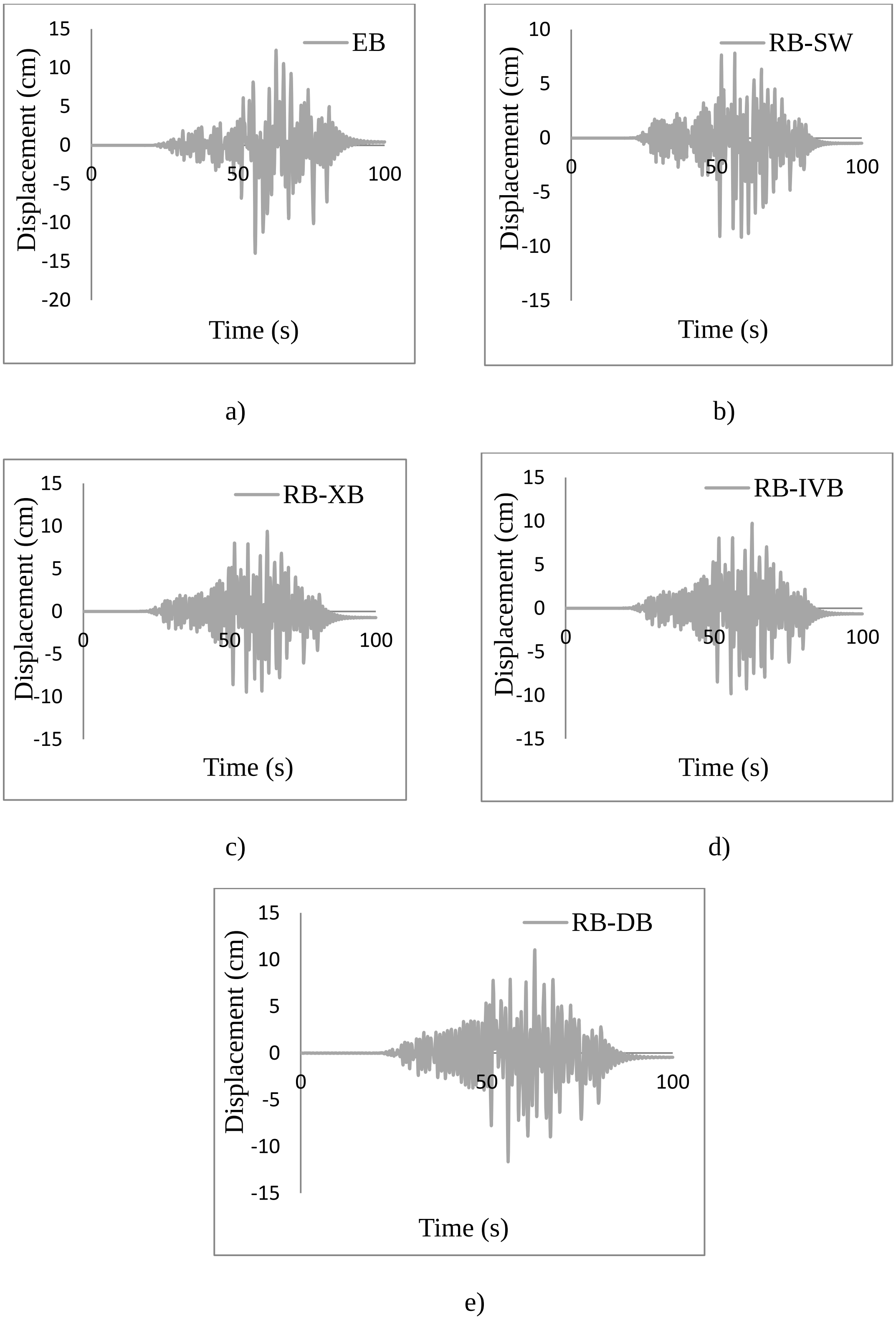
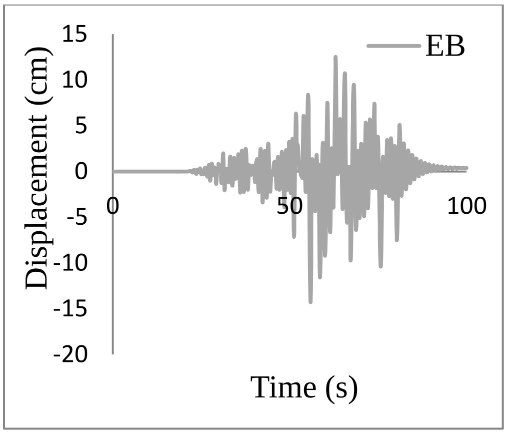


Figure 4.11 Eighth story-displacement time history variations of the existing and retrofitted buildings



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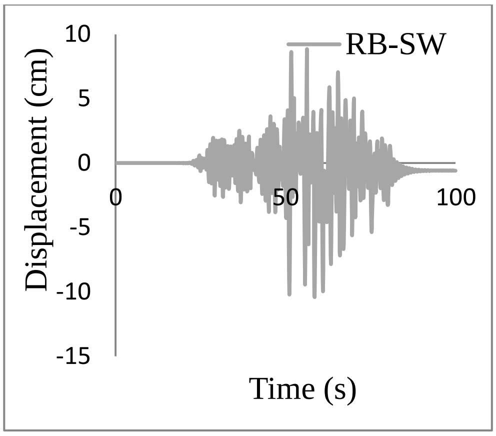

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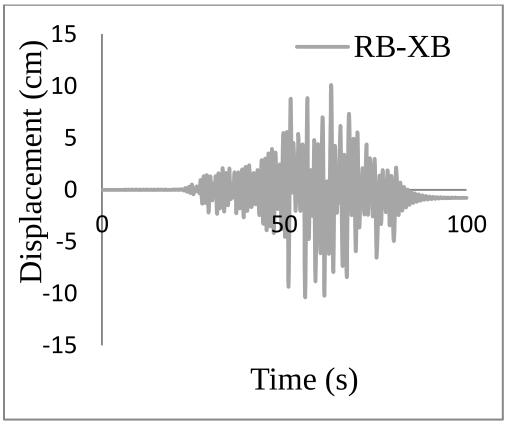

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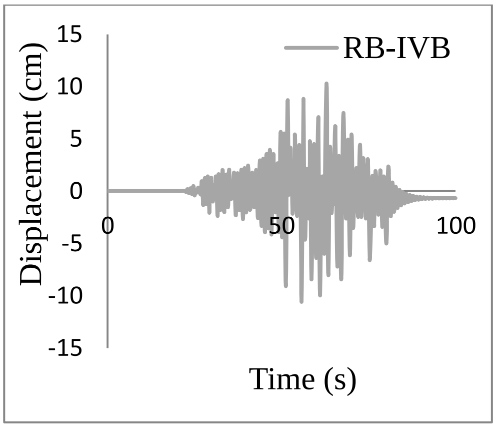

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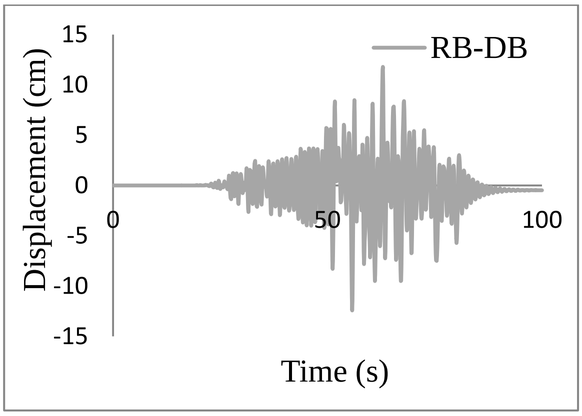

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Figure 4.12 Ninth story-displacement time history variations of the existing and retrofitted buildings

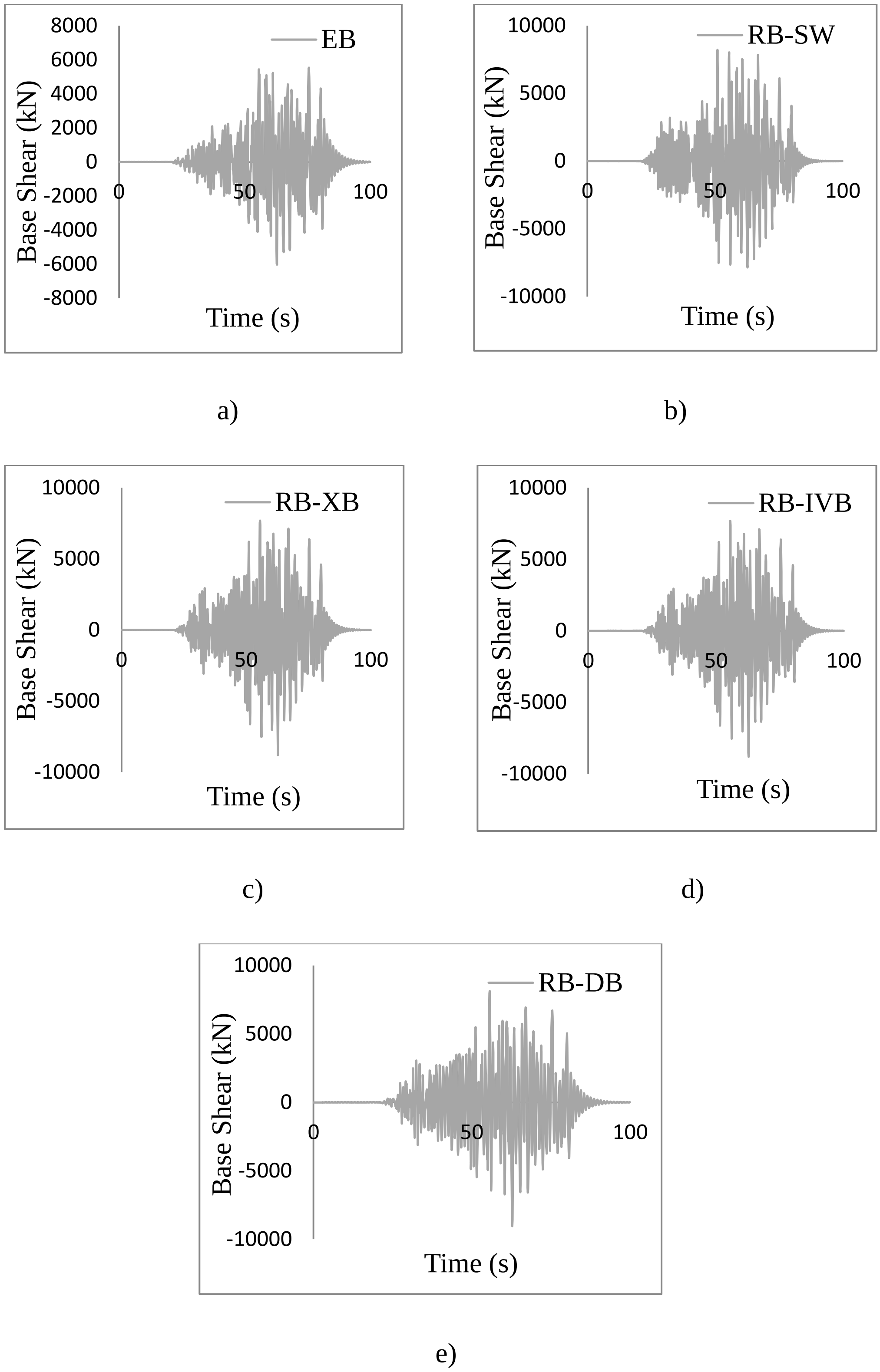

Figure 4.13 Base shear time-history of the existing and retrofitted buildings

CHAPTER 5

CONCLUSIONS

In this study, the nonlinear static pushover analysis and nonlinear time history analysis were performed using a finite element software for both existing building and retrofitted building of 9-story. In the dynamic analysis, 1999 Hector Mine was utilized as a ground motion. From these analyses, the capacity curves, performance level, interstory drift ratio, story displacements, and time history plots were obtained for each building with and without shear walls and concentric steel bracing. Depend on the results observed in this study, the following conclusion can be drawn:

- The nonlinear analysis carried out on the existing and retrofitted buildings indicated that the capacity curves had larger values of lateral strength and higher stiffness for the retrofitted buildings in comparison to the existing building. The performance level of the existing building improved from the collapse prevention (CP) to the life safety (LS) after retrofitting.

- The capacity curves for the nine story building showed that the retrofitting cases with shear wall and X-brace systems had about 2.32 and 2.23 times higher load carrying capacity than the existing building, respectively. In the case of using inverted-V brace and diagonal brace, these values were about 1.80 and 1.61 times higher load carrying capacity than the existing building, respectively.

- The behavior of the retrofitted buildings with X-bracing, inverted-V bracing, and diagonal bracing was comparable in the elastic region based on the capacity curves; however in the inelastic region it was different. The X- braced building was seemed to be capable of more lateral load carrying and energy absorption capacities than the inverted-V braced building, and also inverted-V braced building had more lateral load carrying and energy absorption capacities than the diagonal braced building due to the contribution of X-brace and inverted -V brace elements in which the load distributed more balanced between the braced members.

- Analysis of the results indicated that the use of the shear wall, X-bracing inverted-V, and diagonal braces as a retrofit strategy decreased considerably the value of maximum storey displacement demand.

- From the results of this study, it was also observed that the maximum inter-story drift ratio demand of the nine story retrofitted buildings with shear walls (RB-SW), X- bracing (RB-XB), inverted-V bracing (RB-IVB), and diagonal bracing (RB-DB) were about 0.42, 0.44, 0.49, and 0.57, respectively. While the maximum inter- story drift ratio of the existing building was approximately 0.82. Thus, it was concluded that the retrofitted buildings had significantly lower drift demand in comparison to the existing building.

- When the existing and retrofitted buildings were compared with each other, it was found that the former had significantly lower roof displacement than the latter under the ground motion record of 1999 Hector Mine. Among the retrofitting strategies, the retrofitted building with shear wall, X-brace system, and inverted-V bracing system gave close roof displacement and relatively lower than that with diagonal bracing system.

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APPENDIX

Appendix A: Deflected shapes

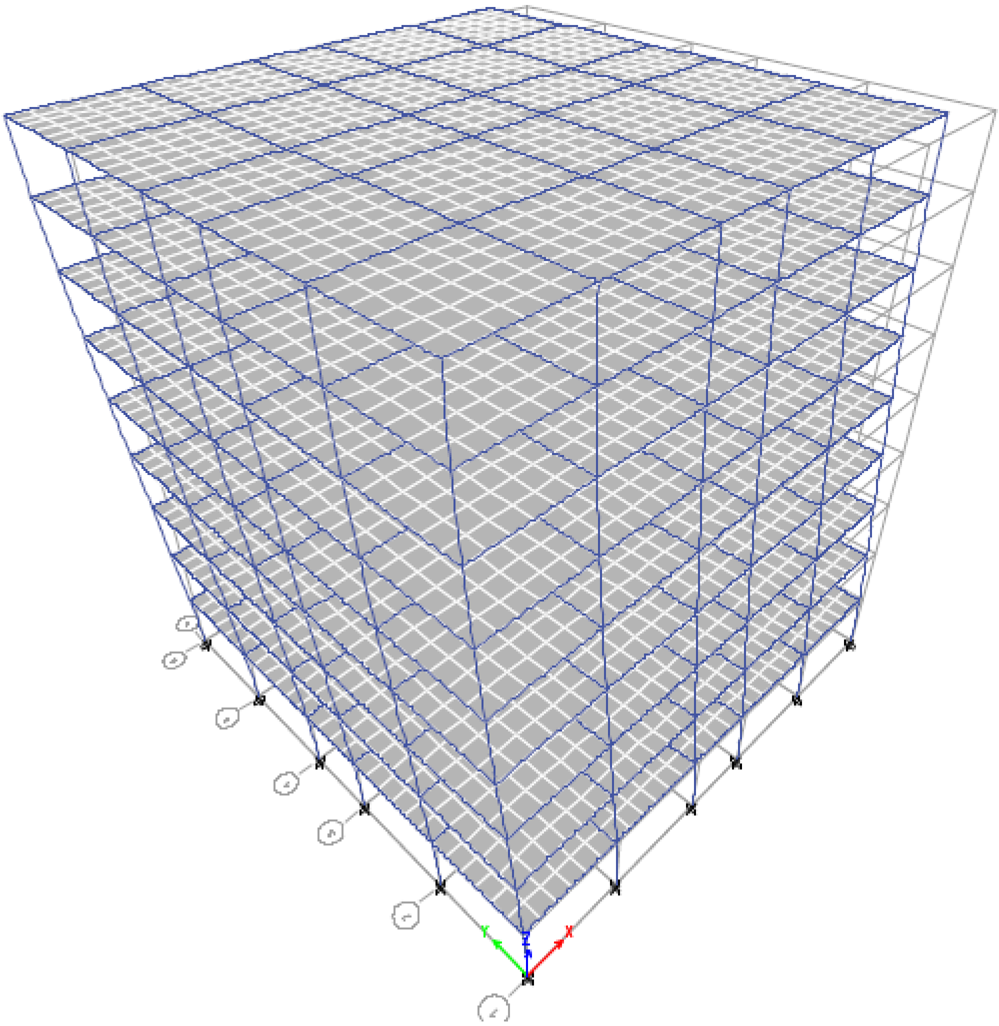

Figure A1 Mode shape of the existing building at T_1=0.1.17 s

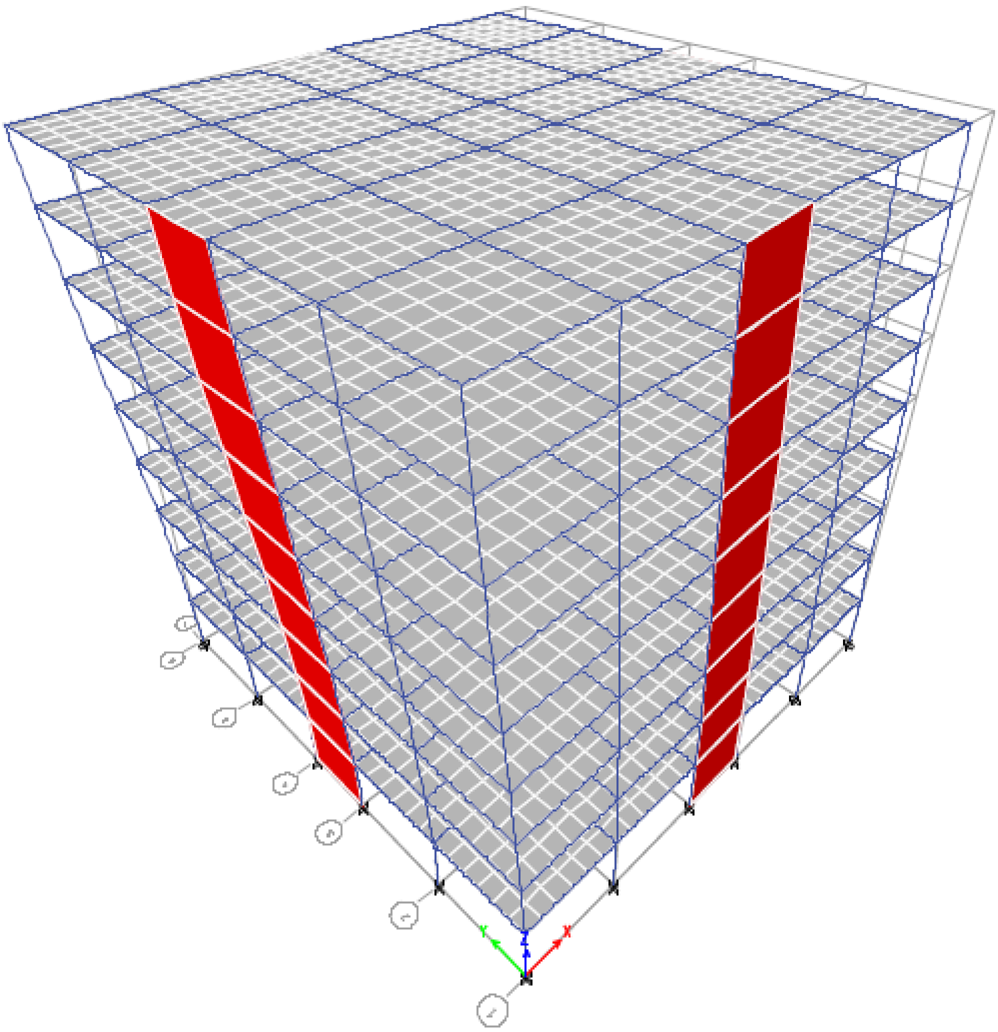

Figure A2 Mode shape of the retrofitted building with shear wall at T_1=0.90 s

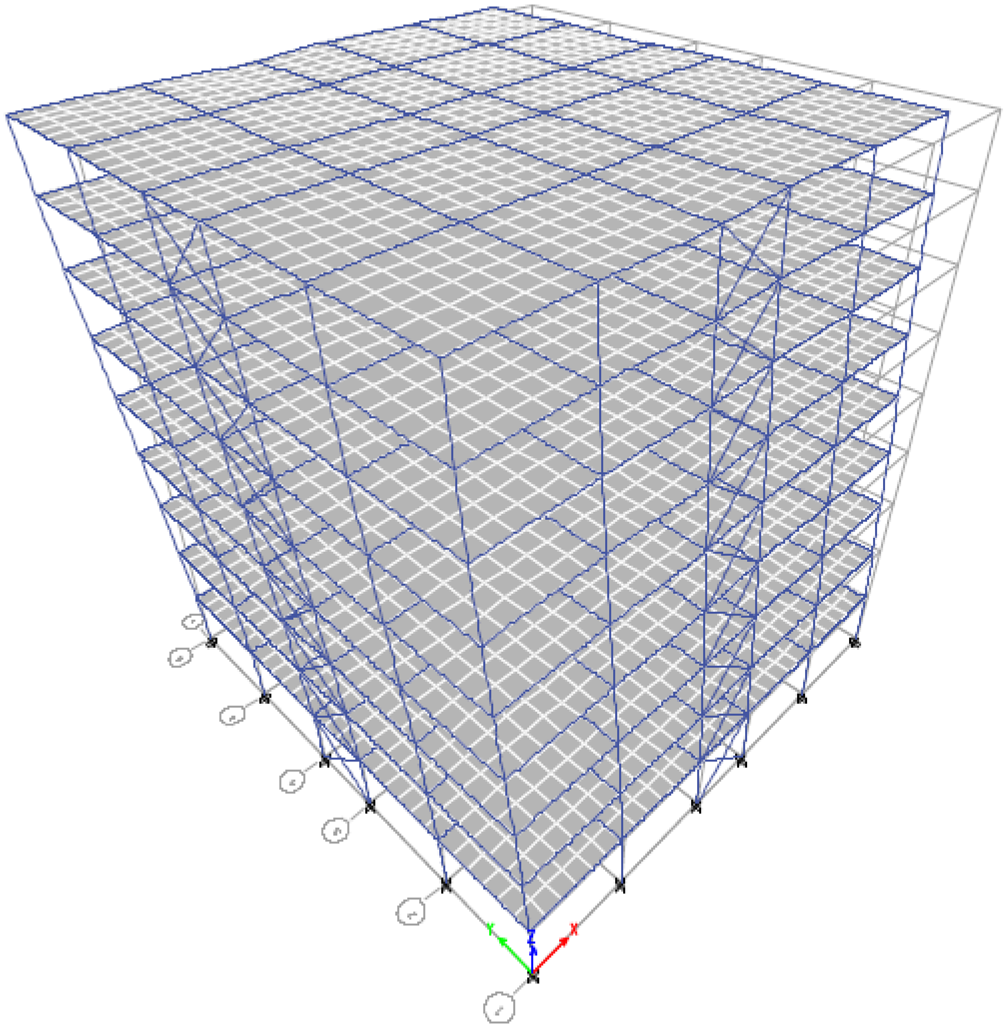

Figure A3 Mode shape of the retrofitted building with X-bracing at T_1=0.99 s

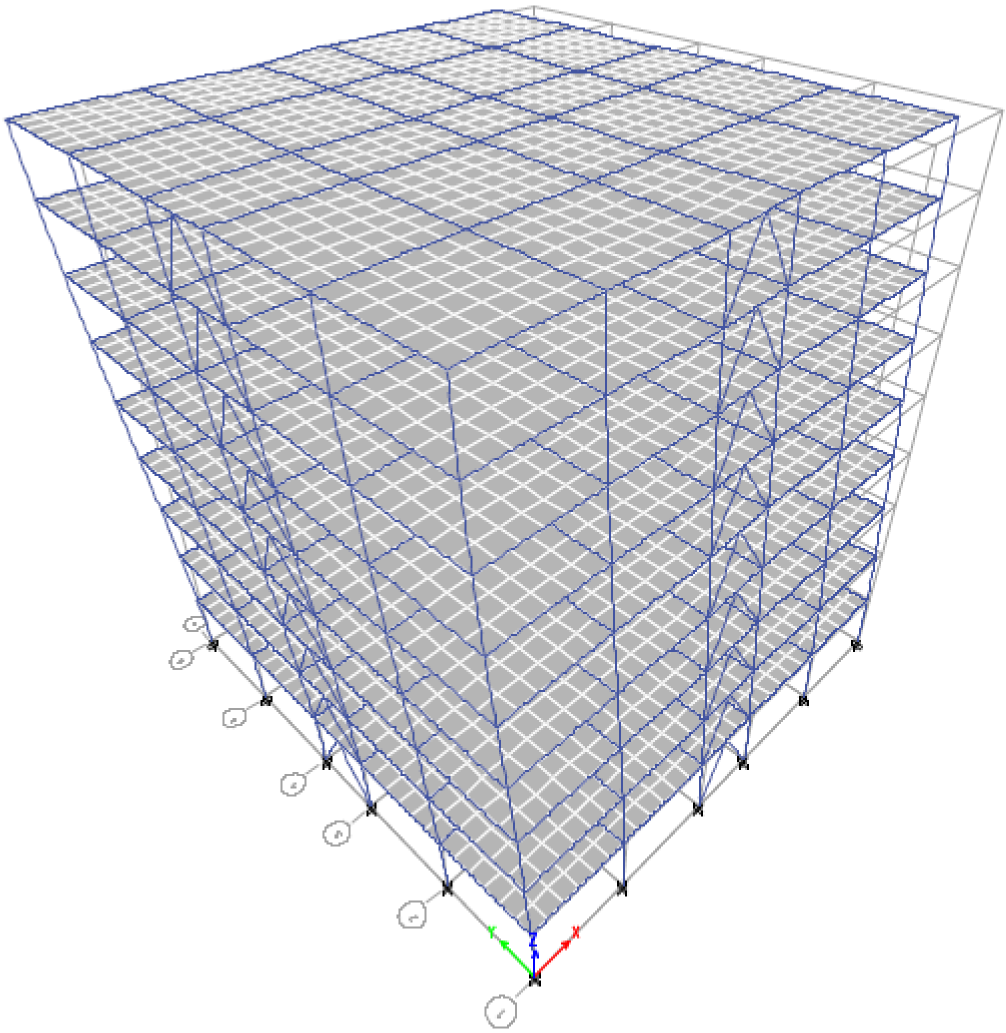

Figure A4 Mode shape of the retrofitted building with inverted V-bracing at T_1=0.99 s

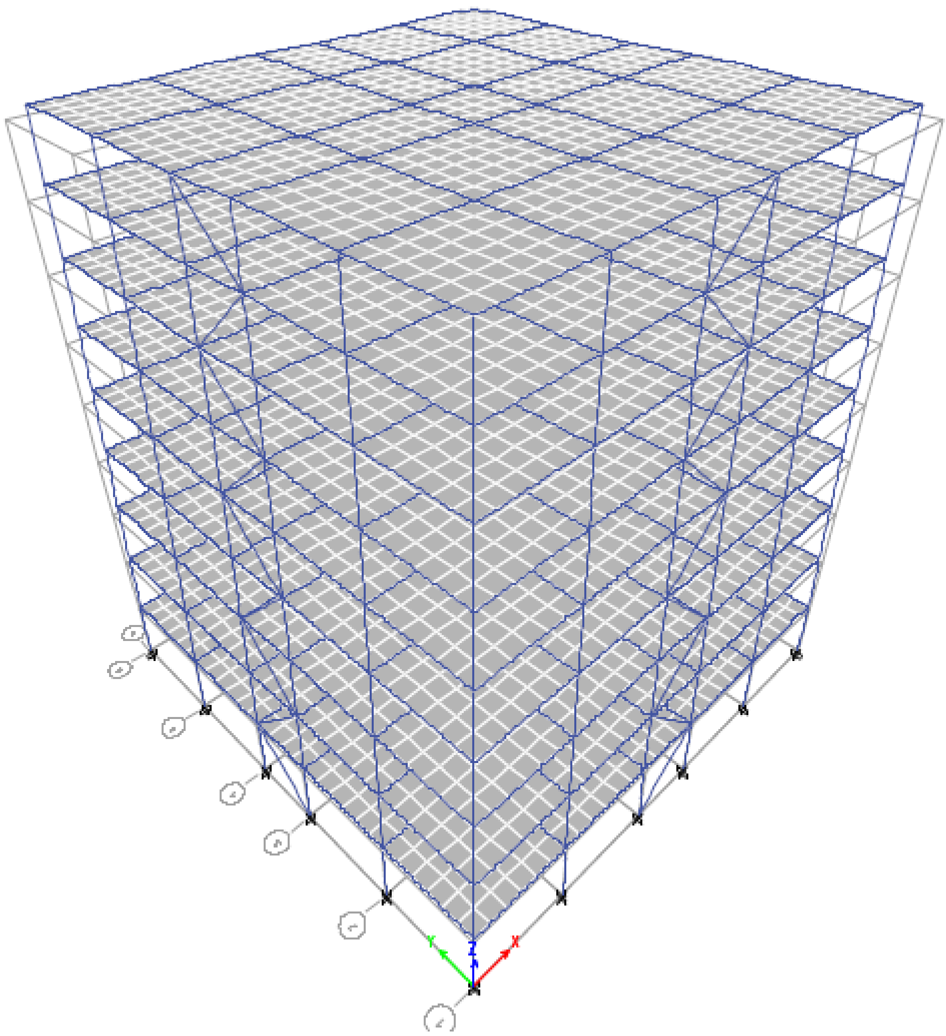

Figure A5 Mode shape of the retrofitted building with diagonal bracing at T_1=1.02 s

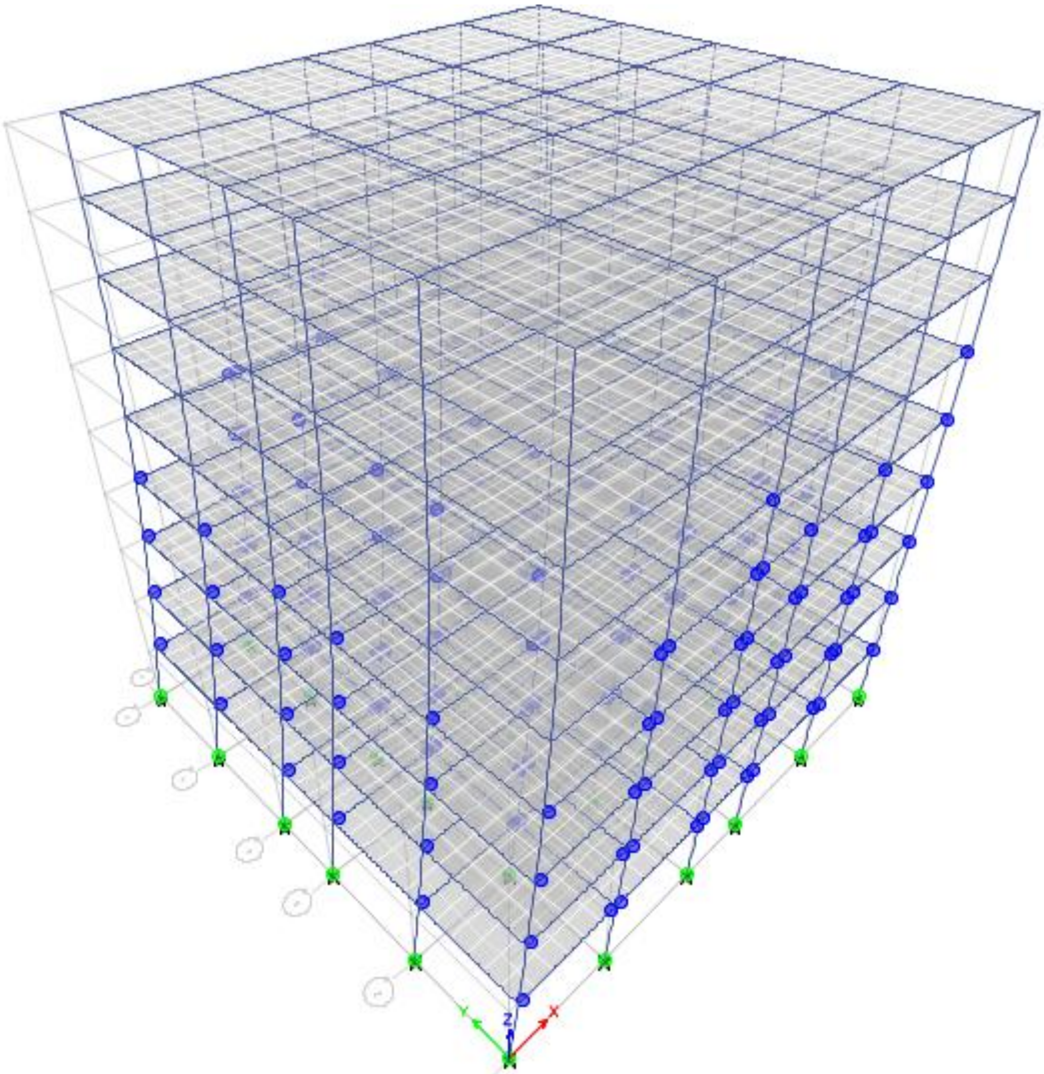

Figure A6 3D View of hinge formation of the existing building at the performance point

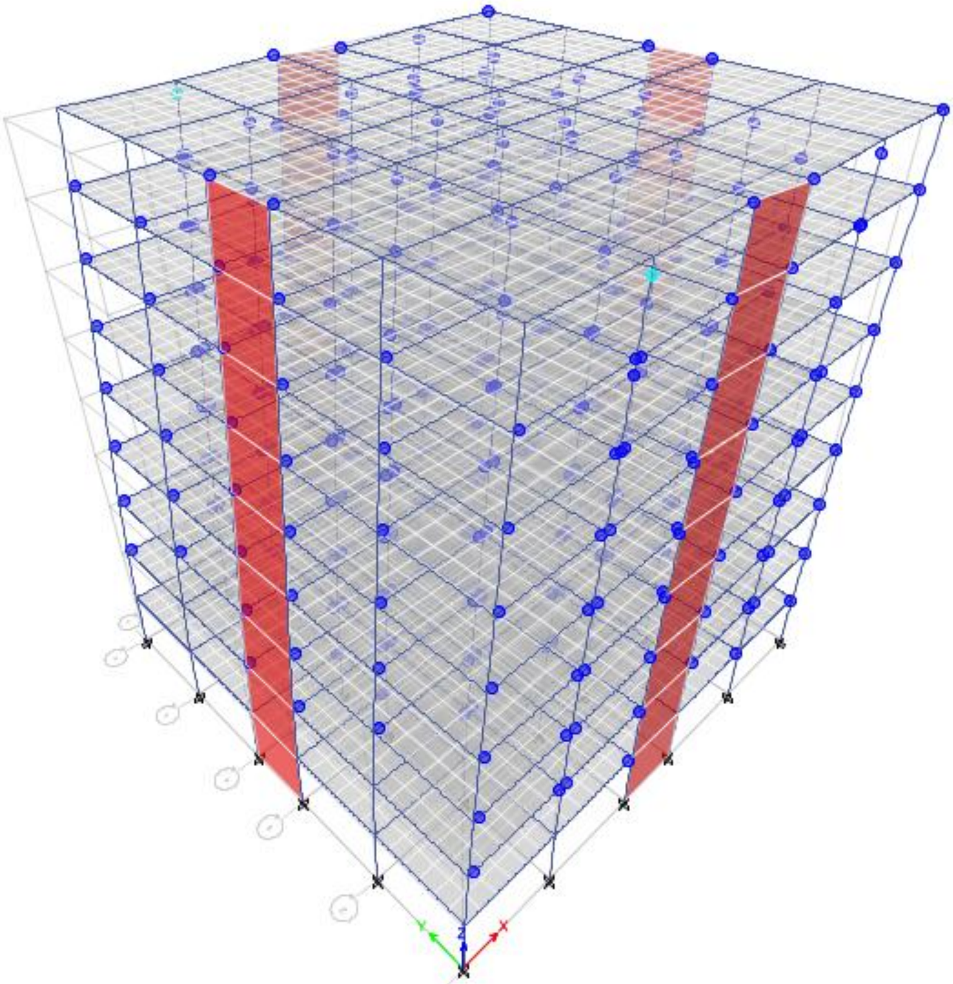

Figure A7 3D View of hinge formation of the retrofitted building with shear wall at the performance point

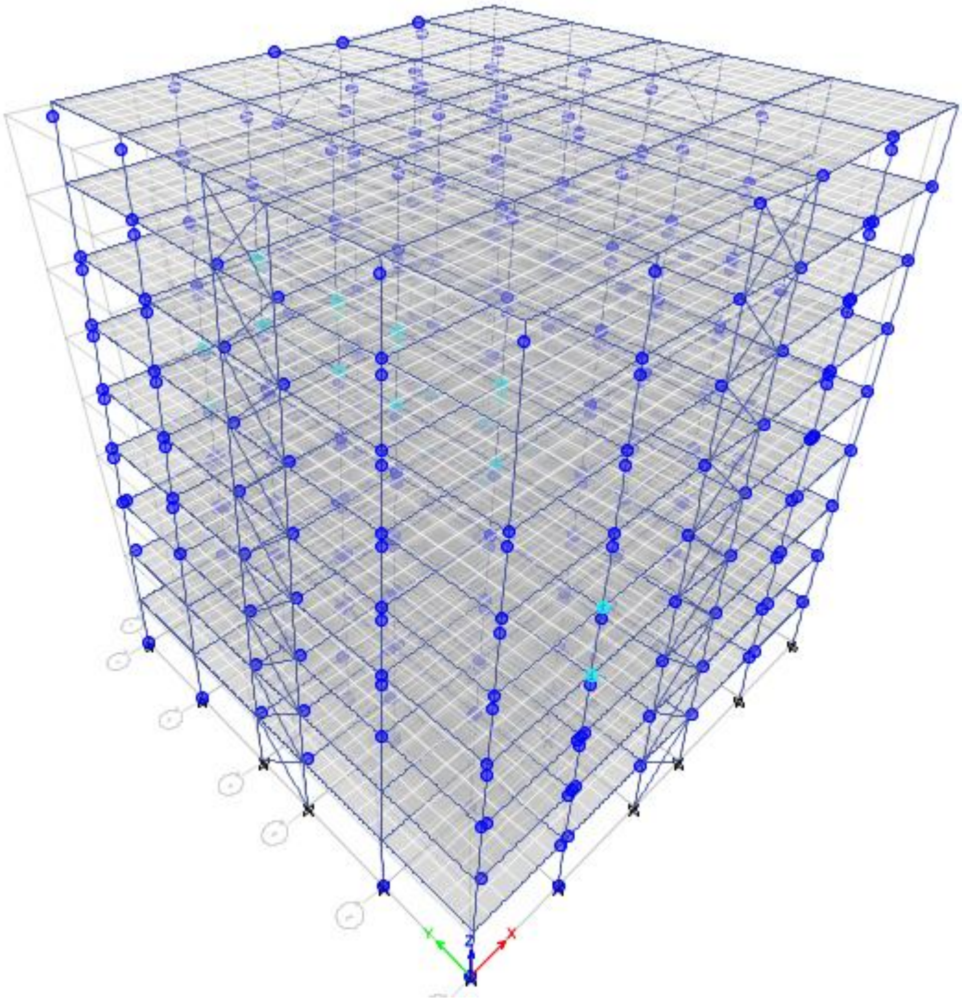

Figure A8 3D View of hinge formation of the retrofitted building with X-bracing at the performance point

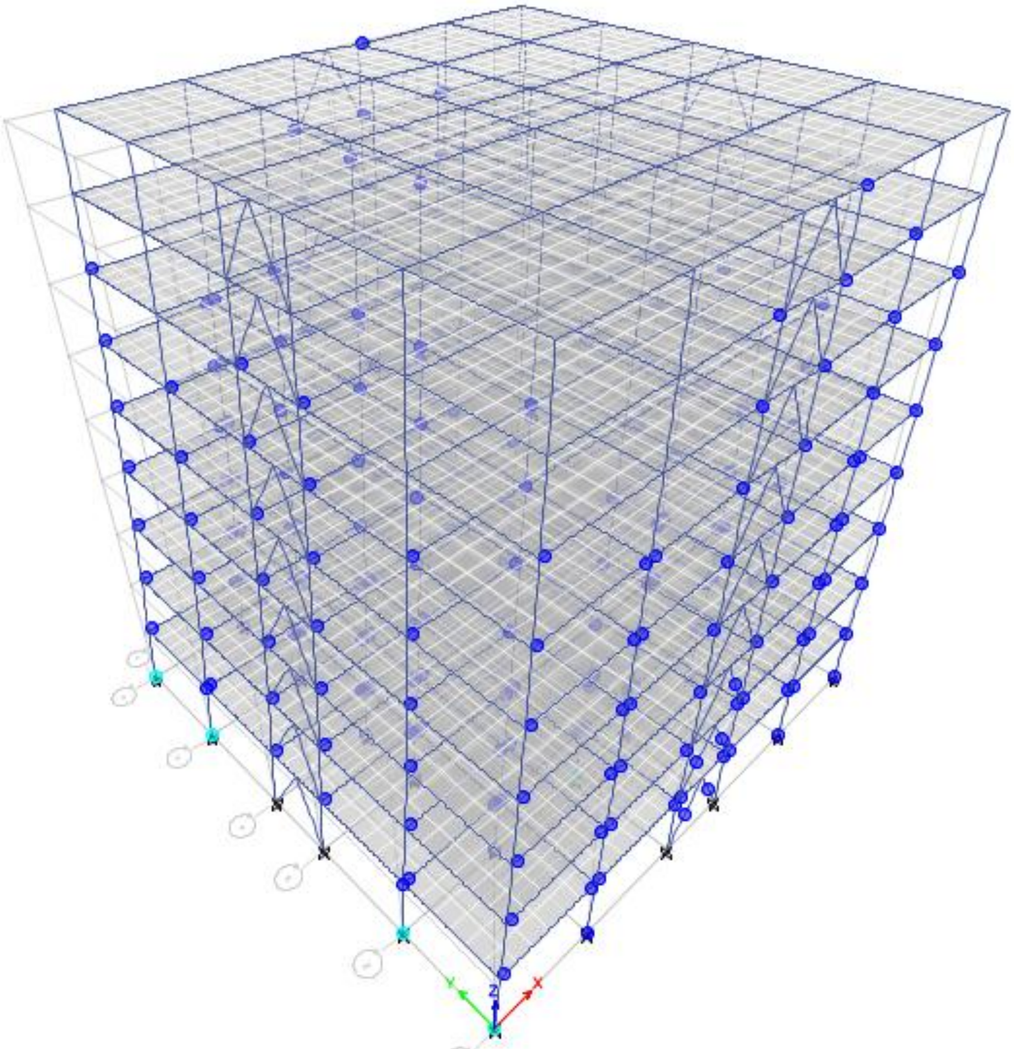

Figure A9 3D View of hinge formation of the retrofitted building with inverted V-bracing at the performance point

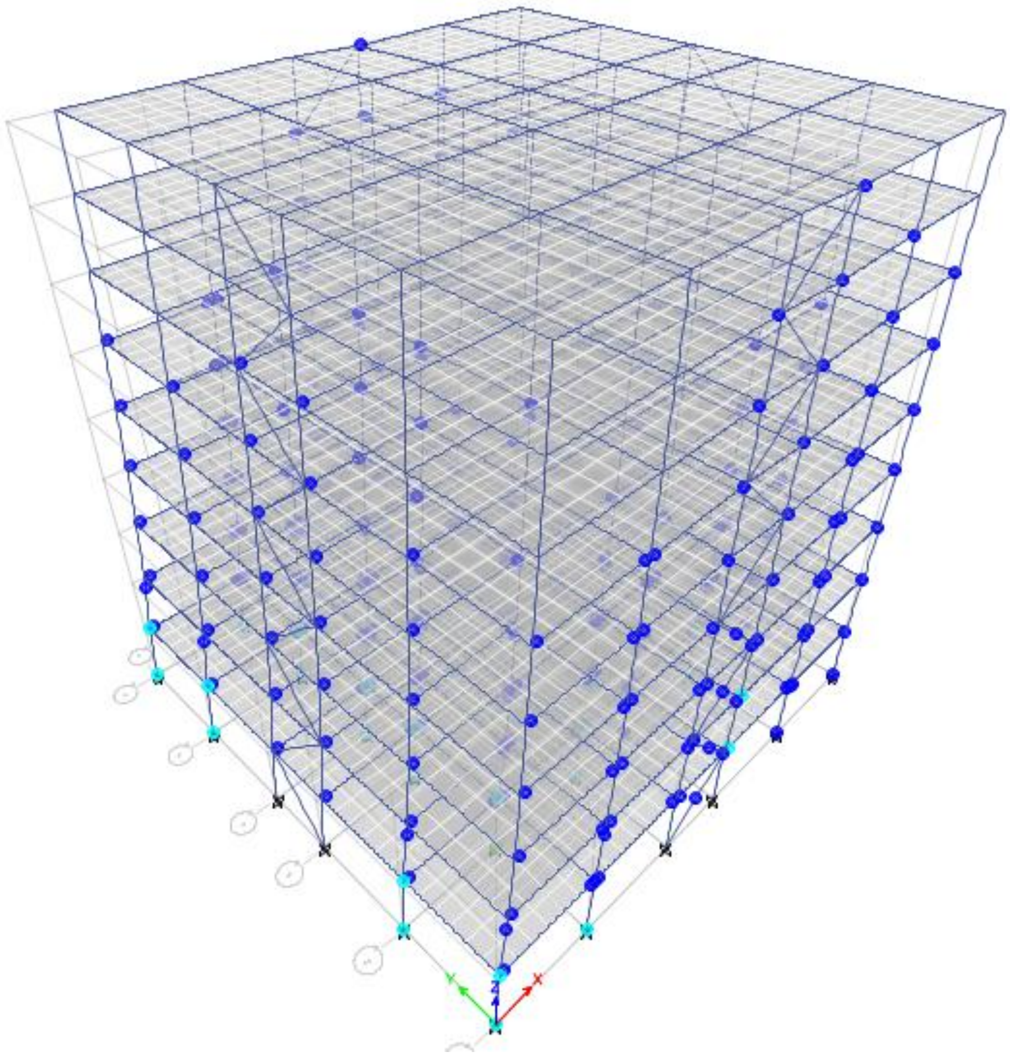

Figure A10 3D View of hinge formation of the retrofitted building with diagonal bracing at the performance point