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GRADUATE SCHOOL OF
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A STUDY ON EFFECTIVENESS OF BUCKLING RESTRAINED BRACES IN MITIGATING EARTHQUAKE RESPONSE OF FRAMED BUILDINGS

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Rawsht ABDALLAH

ABSTRACT

A STUDY ON EFFECTIVENESS OF BUCKLING RESTRAINED BRACES IN MITIGATING EARTHQUAKE RESPONSE OF FRAMED BUILDINGS

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Steel bracing members are widely used in the structures to reduce lateral displacement and dissipate energy during earthquake motions. However, under cyclic loads such as earthquake, the bracing members are subjected to cyclic tension and compression axial loads which results in buckling of bracing members, increase in member lateral displacements and reduction in the load resistance capacities. In recent years, to overcome this negative behavior of bracing members, significant research efforts have been intended for developing buckling restrained bracing with stable hysteretic behavior, significant ductility, and large energy dissipation. In this study, a numerical investigation is presented on the effectiveness of BRBs to protect and mitigate response of structures subjected to lateral loading. For this, nonlinear static analysis was performed in order to examine the seismic behavior of different ordinary and buckling restrained braced frame buildings. As a case study, 2, 4, 6, and 8 storey ordinary moment resisting steel frame buildings were designed with lateral stiffness insufficient to comply with drift limitations given in FEMA 356. Frames of each building had equal four bays of 5 m. Then, BRBs with different configurations, namely chevron, diagonal, split-X, and V-bracing systems were inserted into the exterior bays of each building. The performances of the frames with and without BRBs were investigated through nonlinear analysis. As a result of the analysis, inter-storey drift index, global damage index, capacity curves, plastic hinge formations, and deflected shape were obtained for the bare frames and buckling restrained braced frames and discussed comparatively.

Keywords: Buckling-restrained brace; Ordinary frame; Performance characteristics; Structural response.

ÖZET

BURKULMASI ÖNLENMİŞ ÇAPRAZLARIN BİNALARIN DEPREM TEPKİLERİNİ İNDİRGENMESİ ÜZERİNDEKİ ETKİNLİĞİNİN ARAŞTIRILMASI

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Çelik çapraz elemanlar deprem hareketi süresince yanal ötelenmeyi azaltmak ve enerjiyi sönümlemek için yapılarda yaygın olarak kullanılmaktadır. Bununla birlikte deprem gibi tekrarlı yükler altında, çapraz elemanlar burkulmaya, yanal ötelenmeye ve yük taşıma kapasitelerinin düşmesine neden olan eksenel çekme-basınç etkisine maruz kalırlar. Son yıllarda, aşırı yüklemeler altında oluşan bu olumsuz davranışların üstesinden gelebilmek için, kararlı histeretik davranışa, yüksek süneklik düzeyine ve yüksek oranda enerji sönümleme kapasitesine sahip burkulması önlenmiş çapraz sistemlerin geliştirilmesi için önemli çalışmalar gerçekleştirilmiştir. Bu çalışmada, burkulması önlenmiş çaprazların (BÖÇ), yanal yüklemeye maruz kalan yapıların korunması ve tepkilerin azaltılması üzerindeki etkisi ile ilgili analitik (sayısal) bir araştırma sunulmuştur. Bunun için sıradan ve burkulması önlenmiş çapraz sistem bulunduran farklı çerçeveli yapıların sismik davranışları doğrusal olmayan statik analiz yöntemiyle incelemiştir. Vaka çalışması olarak FEMA 356'da belirtilen kat ötelenme oranı sınırlarını sağlamayan 2, 4, 6 ve 8 katlı sıradan moment aktaran çelik çerçeve binalar tasarlanmıştır. Her bir binanın çerçevesi 5 m'lik 4 eşit açıklığa sahiptir. Daha sonra, burkulması önlenmiş çaprazlar farklı şekillerde, ters-V, diyagonal, ayrık-X ve V-çapraz sistemler olarak her bir binanın dış çerçevelerine yerleştirilmiştir. BÖÇ bulunduran ve bulundurmayan binaların performansları doğrusal olmayan analiz ile araştırılmıştır. Sonuç olarak, farklı özelliklere sahip çerçevelerin katlararası ötelenme indeksleri, genel hasar indeksleri, kapasite eğrileri ve plastik mafsal oluşumları karşılaştırılmalı olarak değerlendirilmiştir.

Anahtar kelimeler: Burkulması önlenmiş çapraz; Sıradan çerçeve; Performans özellikleri; Yapısal tepki.

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LIST OF SYMBOLS/ABBREVIATIONS

δ_{max}	Maximum inter-story drift
BF	Braced frame
BF-BF	Braced frames in both horizontal direction
BRB	Buckling restrained brace
BRBF	Buckling restrained braced frame
C_1	Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear 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-delta
CBF	Conventionally braced frame
CHBF	Chevron braced frame
C_o	Modification factor to relate spectral displacement and likely building roof displacement
CP	Collapse prevention level
D	Roof displacement
DBF	Diagonal braced frame
EBF	Eccentrically braced frame
h	Storey height

H	Total height of the building
IO	Immediate occupancy level
KBF	Knee braced frame
LS	Life safety level
MBF	Mega braced frame
MRF	Moment resisting frame
OCBF	Ordinary concentrically braced frame
OF	Ordinary frame
S_a	Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration
SCBF	Special concentrically braced frame
SXBF	Split-X brace frame
$T_{e:}$	Effective fundamental period of the building in the direction under consideration
UB	Unbonded brace
UF	Un-braced frame
UF-BF	Un-braced frames in one horizontal direction and braced frames in the other direction
UF-UF	Un-braced frames in two horizontal direction
VBF	V-braced frame

CHAPTER 1

INTRODUCTION

1.1 General

Earthquake action brings concern in structural design in earthquake-prone countries. For many years, various design and construction technologies have been advanced, aiming at enhancing the seismic behavior of building structure. To resist large displacement during serve excitation, and desire unique concentration to boundary damage and avoid problems associated with P-Δ effects steel moment-resisting frames are susceptible. As economic and practical concept, engineers have growingly revolved to concentrically braced steel structures to decrease the impact of earthquake and wind forces to enhancing the lateral strength and stiffness of steel building for resisting earthquake loads, but harm to this braced frames in past earthquakes such as 1985 Mexico (Osteraas and Krawinkler, 1989; Kim and Goel, 1992), 1994 Northridge (Tremblay et al., 1995; Krawinkler et al., 1996), and 1995 Hyogo-ken Nanbu (AIJ, 1995; Tremblayat el., 1996; Hisatoku, 1995;) earthquakes increases affects about the ultimate deformation capacity of the frames.

Under cyclic loading, single braces regularly obtain only limited ductility capacity. In tension and compression, brace hysteresis response is unsymmetrical, and while, load monotonically in compression or cyclically, typical substantial strength deterioration was shows, (Asgarian et al., 2008), as exhibits in Figure 1.1. Use of these braces in concentrically braced frames (CBFs) has long been studied to be prone to various non-ductile styles of response when exposed to great ductility demands. Such modes include connection and member fracture, More specifically, severe loss of strength, stiffness due to beam ductility resulting from unbalanced tension and compression strengths and unable to dissipate energy have been observed in concentrically braced frames (AISC, 2002). Furthermore, it has also been well known that the lateral buckling of braces may substantial to damage and instability of building. Prompted to these concerns and faults of concentrically braced frames, seismic design needs to enhance the compressive capacity and symmetric hysteretic response of braces. As a result, a modern sort of brace named buckling restrained brace (BRB) with a perfect nonlinear behavior such as symmetrical hysteresis behavior, large energy dissipation, and significant ductility has been developed, and that is by providing lateral support to ordinary braces that prevent buckling deformation, as shown in Figure 1.2a. Hence, the brace shows the same nonlinear behavior in both tension and compression (Kumar et al., 2007).

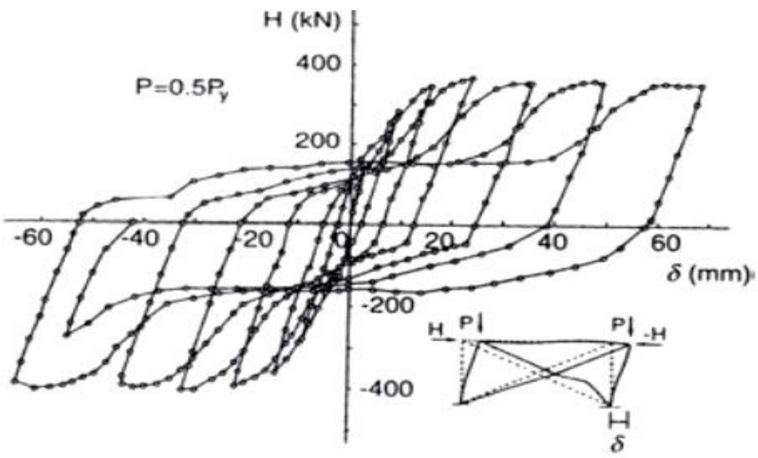
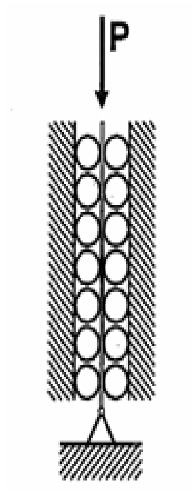
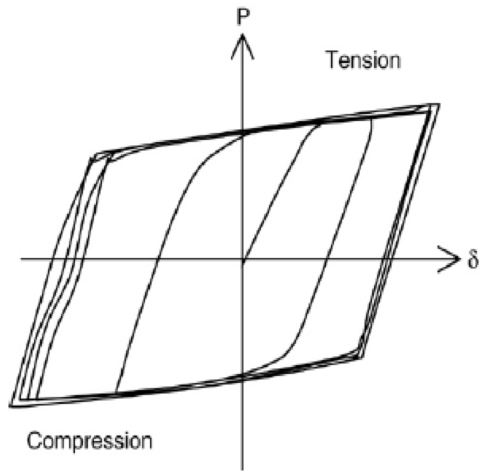


Figure 1.1 Hysteresis behavior of conventional brace (Bruneau et al., 1998)



a)



b)

Figure 1.2 Illustrative diagram of: a) BRB and b) balanced hysteresis of BRBs (Kumar et al., 2007)

These braces have an ultimate compressive strength as equal as tension strength which shown in Figure 1.2b. For BRBFs an interesting design ways has been proposed by Wada (1992), in which, during seismic response, the basic structural framework is designed to stay elastic, and all of the seismic damage (yielding) happens enclosed by the braces. BRBFs are desirable for rehabilitation for their select ductile performance and seismic design (Wada et al., 1994).

In the literature, several researches have been conducted for evaluating the seismic behavior of buckling restrained braced frames (BRBFs) with different configuration subjected to strong ground motions. Asgarian et al. (2008) presented the effect of design loads in the seismic performance of BRBFs, Qiang (2005) performed various types of BRBs with different configuration, Duixian et al. (2011) introduced the location effecting of the braces to the lateral displace of steel frames, Kumar et al., (2007) utilized the action of frames with non-buckling bracing under earthquake loading, and Seifi et al. (2008) also illustrated the use of BRBs for earthquake resistant design of buildings.

1.2 Buckling restrained brace (BRB)

Buckling restrained braced frames (BRBFs) are a kind of braced frame, and unique due to the configuration of the brace elements. The BRBF is a proportionately new type of concentrically braced frame system. As exhibited in Figure 1.3, BRB generally includes of the two main parts, the steel core resists axial stresses and the outer concrete filled steel casing prevent buckling stresses, and the casing restrains the steel core from buckling thereby growing almost uniform axial strains in tension and compression (Sabelli and Lopez, 2004).

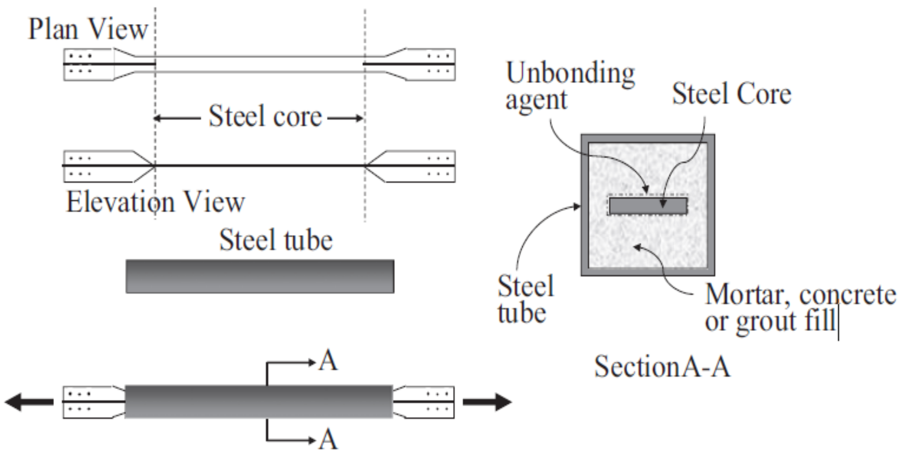


Figure 1.3 Graphics configurations of BRBs (Tremblay et al., 2006)

BRBs have been used widely in Japan within moment-resisting frames as hysteresis dampers. These braces were established to U.S. design experimental in 1999, and their use has been mostly as a building’s initial seismic-load resisting technique. Since their introduction from Japan to the United States in the late 1990’s, buckling-restrained braces have undergone extensive testing by researchers, demonstrating good performance in both tension and compression (Inoue et al., 2001; Black et al., 2004; Sabelli et al., 2003; Tremblay et al., 2006).

Although buckling restrained brace testing indicates the potential for undesirable failure modes within connection regions at large deformations, these failure modes include: fracture of the beam-to-gusset and column-to-gusset welds, beam local buckling, and column local buckling as seen in Figure 1.4 (Aiken et al., 2002; Roeder et al., 2006).

Buckling of braces represents a severe limitation to their ductility and the performance of the system. Because the strains are not concentrated in a limited region such as a plastic hinge, the braces can dissipate large amounts of energy. BRBF has full balanced hysteresis loops with compression yielding similar to tension yielding behavior. This is achieved through the decoupling of the stress resisting and flexural buckling resisting aspects of the compression strength (Sabelli and Lopez, 2004).

Due to containing and limiting in elastic behavior in BRBFs, do not prevent the conventional frame to remain essentially elastic. Furthermore, BRBs decrease the effective period of the frame due to softening and yielding and thus effectively decrease the total base shear. It is needed to select proper size of the steel core and iterative way in designing BRBFs is important to obtain desire performance. Too small area of a core steel may not produce sufficient toughness and stiffness to the frame to which increase lateral displacement. On the other way, too large area of core steel may prevent yielding of the brace, so that it is affect in the design basis earthquake which occur increase the design of the base shear (Hussain et al., 2005).

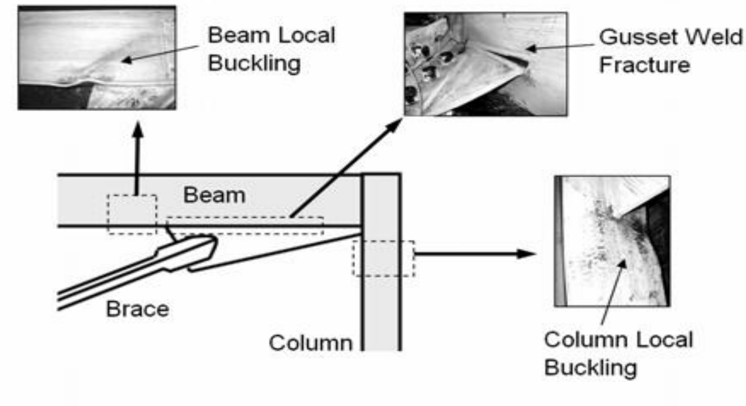


Figure 1.4 BRBF connection failures (Christopolus, 2006)

1.3 Objective and scope

A numerical investigation is presented on buckling restrained braces and nonlinear analysis was performed in order to evaluate and compare the seismic behavior of ordinary structures and those with different configurations of buckling restrained braces, namely chevron bracing, diagonal bracing, split-X bracing, and V-bracing systems. Four ordinary frames (OFs) with 2, 4, 6 and 8 stories in height were designed with lateral stiffness insufficient to satisfy code drift limitations given in FEMA 356 for steel moment resisting frame systems in seismic regions. Then, BRBs were inserted to the original frames with different configurations (DBF, CHBF, SXBF, and VBF) and different distributions (Case-1, Case-2, Case-3, and Case-4) over the elevation of the frames. Thus, a total of 68 (4 OFs and 64 OFs with BRBs) of different cases were taken into consideration within the scope of this study. As a result of analysis, inter-storey drift index, global damage index, capacity curves, plastic hinge formations, deflected shape, and base shear were investigated for each ordinary frame and frame with buckling restrained braces and discussed comparatively.

1.4 Outline of the thesis

The major effective of this thesis is to provide a description through nonlinear pushover analysis procedure for the different type of BRBs with different configurations of braced frames and assess their performances.

Chapter 1-Introduction: Aim and objectives of the thesis are introduced.

Chapter 2-Literature review: This chapter focuses on the historical background on practical application, previous studies on braced frames, and different types of BRBs.

Chapter 3-Methodology: This chapter covers the description of the different frames and loading conditions of the frames. Also, in this section, the various models are illustrated to describe the nonlinear pushover as the analysis way. Aspects of BRBs, different configurations, and different height of the frames are also expressed.

Chapter 4-Results and discussion: In this chapter, the various models are analyzed, also presents and discussed the results obtained from nonlinear analysis for computing the structural performance and the parametric of each frame system conducted in this study in term of inter-storey index, global index, capacity curves, and etc.

Chapter 5-Conclusion: This chapter presents the most relevant conclusions of this work, and dealing the overall results from all chapters.

Appendix A: OFs and BRBFs with different configuration and distribution: Presents OFs and types of BRBFs with different configurations and different height of the frames.

Appendix B: Hinge location and performance level of BRBFs: Presents the location of the OFs and BRBFs hinges with different configurations and different height of the frames.

Appendix C: Pushover curves for BRBFs and OFs: Presents the capacity curves of OFs and BRBFs for different configurations and height of the frames.

Appendix D: Deflected shape of the frames: presents the deflected shapes of the OFs and BRBFs for different configurations and height of the frames.

CHAPTER 2

LITERATURE REVIEW

2.1 General background

Earthquake action brings concern in structural design in earthquake-prone countries. For many years, various design and construction technologies have been developed, aiming at enhancing the seismic performance of building structure. To resist large displacement during serve earthquake ground motion and require special attention to limit damage, steel moment-resisting frames are susceptible. Lateral displacements on structural buildings have been of great concern for engineers. In order to minimize the effect of earthquake and wind forces different engineers have used different techniques. A brief review is presented here. Lloyd (1917) used the layer of roller and talcum powder at foundation and received the patent for it. Oka (1934) constructed buildings using isolation as sliding and roller system. Kawai proposed the timber log placed in several layers in longitudinal and transverses direction as a base isolation system says Izumi (1988). In 1968, large block of hard rubber were used to isolate three storey building at Skopje, Republic of Macedonia, reported by Jurukovski (1995) and Kumar (2004). Martel (1929) proposed the concept of flexible first storey for structural isolation of building. Modification to this approach, as a soft first storey was proposed by Fintel and Khan (1969). This concept was shown impractical by Chopra et al. (1973), since the post yielding stiffness of the columns would have to be impractical if shear force in the upper storey were to be reduced. Matsushita and Inzuma (1977) proposed a structural system involving a double basement design and special construction over three lower floors involving bearing end device. This study comes under passive control where no external source of energy is required. Shukla and Datta (1999) presented the method for optimal use of viscoelastic dampers for control of seismic force. Michael et al. (2000) proposed adaptive base isolation system for building which consist of sliding isolation bearing in combination with hydraulic damper. According to Soong et al. (1997), it was reported that the theory and application of active structural control (which became the subject of intensive research) were needed to highlight.

Inaudi and Kelly (1990) investigated active base isolation with electro hydraulic actuator giving application to four storey building model. Zuk (1968) presented actively controlled structure where external power is required for working of technology. Skinner et al. (1975a, 1975b) led a number of base isolation concept and hysteretic dampers (Skinner et al., 1993; Skinner et al., 1980). Kelly (1987) proposed hybrid control strategy consist of base isolation system with active controlled actuator. The work on active structural control includes prestressed tendon to stabilize and control of tall building by cables attached to jacks (Constantinou et al., 1998). Jangid and Londhe (1998) investigated elliptical rolling rods for multistory building to control the displacements. (Kelly, 1999) proposed low cost fiber reinforced seismic isolation system for developed nation in which steel plates are replaced by carbon fiber mesh. Yoshioka et al. (2002) proposed smart base isolation system with sponge magneto-rheological (MR) damper for near and far field earthquake. David et al. (2001) and David and Stevan (2002) got patent for story isolation in which the response of gravity frame during earthquake motion is controlled by surrounding reaction frames, with spring and dampers connection in between them. Some different technique to control this displacement is the bracing system in the structure.

2.2 Braced frames as seismic systems

If the land area remains constant and the ratio to the people from the country is increasing, engineers having no option other than going for upstanding growth of building. As these vertical structures become slender and slender, the effect of earthquake on these structures became most important. These structures are susceptible to large lateral displacements or to collapse due to severe earthquake ground excitations and require special attention to limit this displacement. This displacement can be brought into a limit by providing the ductility in the structure. This ductile behavior can be achieved by the stable plastic deformation of structural members. To control this lateral displacement, distinct planners have used various techniques. Lessons learned with regard to steel moment resisting frames in the past earthquakes, accelerated as an optional system the research deeds with the purpose to obtain the braced frame structures (Deulkar et al., 2010).

The bracing system consists of providing the inclined members in the frames of building in addition to the structural members, beams and columns. And that is due to the relatively low-cost and easy to construction. Other characteristics of the braces system or an appealing different of the steel system are shear resisting members. Therefore, to obtain the strength of steel moment resisting framed structures and the global stiffness, the bracing system becomes a very effective global upgrading strategy (Deulkar et al., 2010).

Many shapes of braced frames possibly will be used for seismic rehabilitation of existing steel, reinforced concrete building structures and composite steel-concrete, e.g. (Bartera and Giacchetti, 2004; Di Sarno and Elnashai, 2005) among others. The most frequently used systems contain (EBFs) eccentrically-braced frames, (CBFs) concentrically braced frames, and the knee-brace (KBFs) frames. Common configurations for concentrically braced frames include inverted-V and V bracings, diagonal bracing and X, K bracings (Bruneau et al., 1998). In addition, macro-bracing can be used for stiffening and strengthening of steel-framed structures. They are often working to form mega braced frames (MBFs) which intensified ductility and display high stiffness, (Di Sarno and Elnashai, 2009) illustrated to brace configuration with mega brace frames (MBFs) to retrofit a medium-rise steel MRF with insufficient lateral stiffness. Nonetheless, severe earthquakes, e.g. those in the 1985 Mexico (Osteraas and Krawinkler, 1989; Kim and Goel, 1992), 1994 Northridge (Tremblay et al., 1995; Krawinkler et al., 1996), and 1995 Hyogo-ken Nanbu (AIJ, 1995; Tremblayat el., 1996; Hisatoku, 1995;) demonstrates that buckling of the diagonal members and poor detailing of the connections (e.g. column to base, brace to beam, brace to column, beam to column) may erode seismic performance as a whole. (Naeim et al., 2000; Watanabe et al., 1998b; Nakashima et al., 1998; Elnashai et al., 1995; Broderick et al., 1994).

After the 1995 Hyogoken-Nanbu (Kobe) earthquake, a survey was conducted by Youssef et al. (Youssef et al., 1995) in relation to the outcomes of their study, the damage to connections with respect to frame’s kind, constructional elements and the distribution of damage level is illustrated in Figure 2.1 (Di Sarno and Elnashai, 2009).

Damaged structures are categorized as having braced frames (BFs) or un-braced frames (UFs). Therefore taking into consideration the two principal planning directions of a structure, the observed buildings contain the next selections: (braced frames in one direction and horizontally un-braced frames in the other direction) UB-BF, (un-braced frames in both horizontal directions) UF-UF and (braced frames in both horizontal directions) BF-BF, (Youssef et al., 1995).

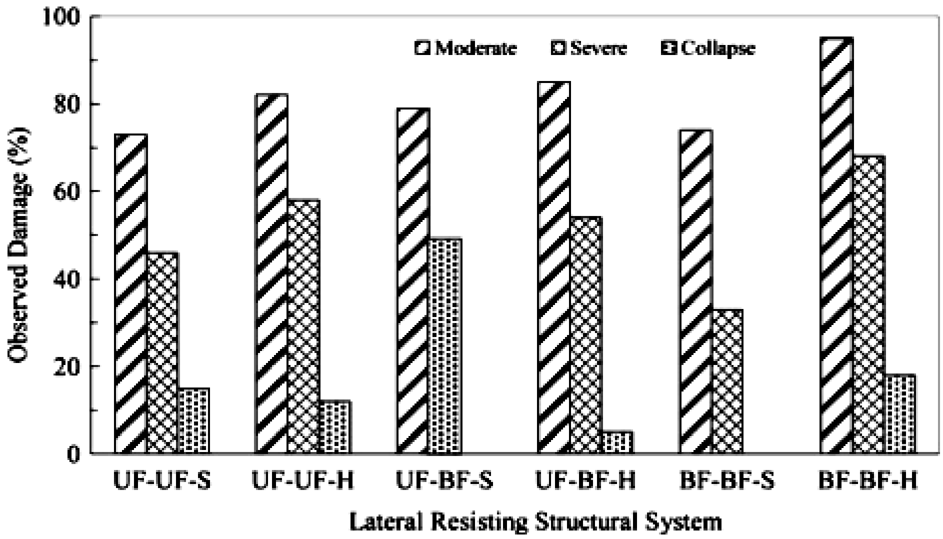


Figure 2.1 Allocation of harm level with aspect to constructional type (Di Sarno and Elnashai, 2009)

Most of the beams made with wide flange section and the columns with wide flange (H) sections, also some structural systems used square-tube sections (S). A total of 988 damaged building were considered in that survey, the statistics according to class of the frame were as follow: a) 134 (13.6%) are UF-BF, b) 432 (43.7%) are UF-UF, c) 34 (3.4%) are BF-BF and d) 388 (39.3%) having unidentified framing systems. These statistics showed that few damages had occurred in the braced frames and most of the damages had occurred in the un-braced frames (Youssef et al., 1995). Location of the damage, namely beams, columns, braces, beam to column connection and column bases, with the type of frame is shown in Figure 2.2. Following observations made from the collected data are as follows (FEMA 355E, 2000):

- In the case of UFs most of the damage occurred in the column compared to other parts of the frames, while in the case of BFs most of the plastic deformation had concentrated in the brace elements,
- Also UFs experienced significant damage in the column bases and the beam to column connections,
- UFs utilized hollow sections for the columns had experienced significant damage in the beam to column connections, and
- Columns made with wide flange sections experienced relevant damage in the case of UFs.

The observation and discussion made for the above surveyed data is representative for steel frames damaged by moderate to severe earthquake excitations (FEMA 355E, 2000).

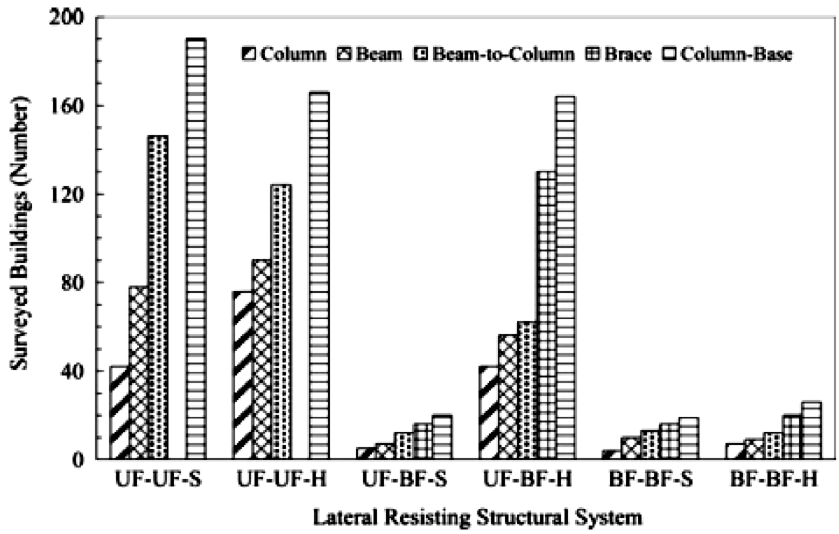


Figure 2.2 Destruction to constructional elements and connections with respect to structural sort (Di Sarno and Elnashai, 2009)

Buckling deformation of the braces may result in eroding the capacity of the structures, degradation of strength and stiffness and sudden change in the dynamic characteristics of the lateral load resisting system (Di Sarno and Elnashai, 2009). Figure 2.3 depicted the brittle fracture for brace to column connections and beam to column that has resulted in reducing the performance and energy dissipation capacity under earthquake excitation. As a result, the braces and column to beam connections should be taken during the capacity design so that to provide sufficient ductility (Bruneau et al., 1998; Nakashima et al., 2000; Tremblay, 2002; Broderick et al., 2005).

a) b)

Figure 2.3 View of a) Rupture in column-to-beam connections in the Northridge earthquake and b) web tear-out in bolted column-to-brace connections during the 1995 Kobe earthquake (Di Sarno and Elnashai, 2009)

Engineers are turning to the use of braced steel frames due to response of some economic and practical issues. Whenever hysteretic dampers are utilized, it is anticipated that the braces can reduce the demand and/or increment the energy absorption of structures imposed by earthquake loads. Structures are expected to resist safely the lateral load induced by an earthquake and avoid the risk of brittle failure if their energy absorption capacity is augmented. Design demands on structural and nonstructural component are conceived to be smaller than their capacity when global modification is applied as shown in Figure 2.4 (Bozorgnia and Bertero, 2004; Di Sarno and Elnashai, 2009). In the buildings, lower demands may prevent the division of its functionality and/or decrease the hazard of brittle failure.

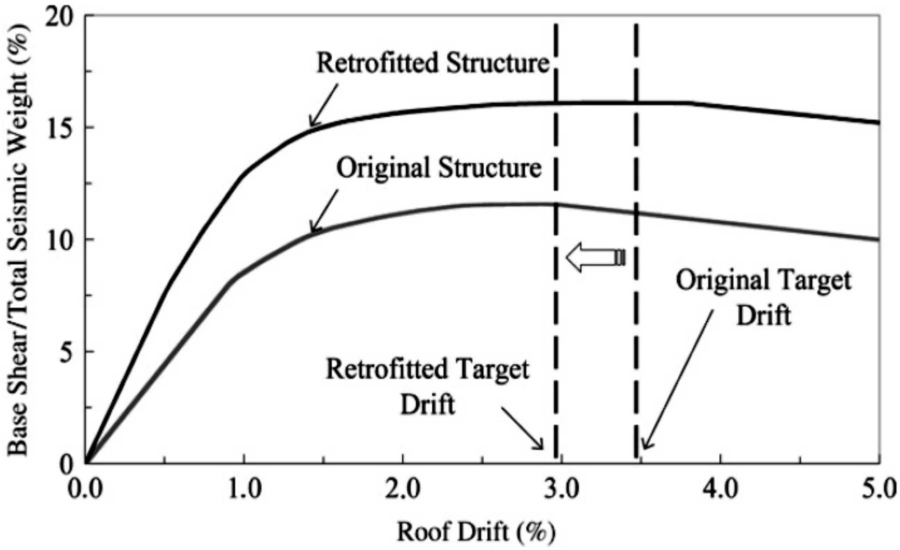


Figure 2.4 Qualities of universal involvement approaches in seismic retrofitting of buildings (Di Sarno and Elnashai, 2009)

During severe earthquakes, large amount of kinetic energy would be fed to the structures. It is not economic to dissipate energy only through the elastic capacity of the materials it is recognized in the all building codes. Thus, the best strategy to dissipate energy is to accept that yielding occurs in the structure but in such a way that plastic deformation would be concentrated at controlled locations or structural fuse and major structural members remain elastically (Deulkar et al., 2010).

In traditional braced frames, braces are considered the structural fuses that dissipate seismic energy through yielding in both tension and compression. However, buckling in the compression led to progressive degrading behavior, and sudden loss of stiffness which limits the amount of energy dissipation. Many engineers have been tried to resolve this buckling problem. However, they were unsuccessful until professor Wada (Wada et al., 1999a; Wada et al., 1999b) found BRBs and his team put forth the concept of BRBs.

2.3 Buckling-restrained braced frames

2.3.1 Concept of BRBs

The damages of the concentrically braced frames (CBFs) methods be able to affected if the brace can yield for the time of both compression and tension without buckling. Researchers and engineers have been motivated to develop a new type of brace which exhibit more ideal elastoplastic behavior, a braced frame that merges this type of brace named buckling restrained braces (BRBs). Hence BRBF is a unique status of CBF that prevents braces buckling. The concept of the BRB is simple, providing lateral support to the brace so that buckling deformation is prevented, a similarity of the operation of conventional braces and BRBs are shown in Figure 2.5 (Clark et al., 1999).

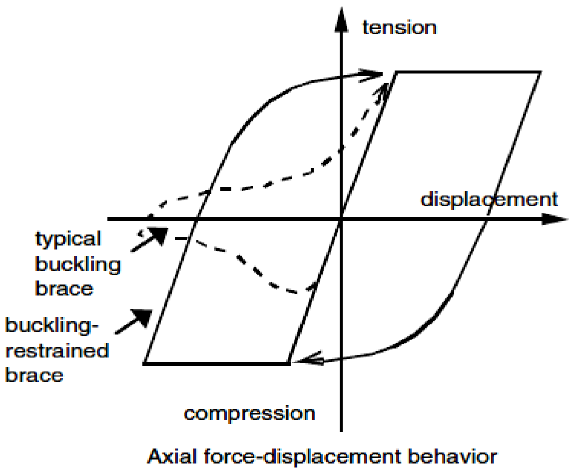


Figure 2.5 Response of conventional brace versus BRBs (Clark et al., 1999)

The BRB is composed of a ductile steel core that carries the entire axial load of the brace and a sleeve surrounding it that provide flexural rigidity and stiffness to prevent global buckling, the space between the steel core and the sleeve is filled with grouting or any other inert filler. As seen in Figure 2.6, the BRB provides a slip surface between the core brace and the encasing unit so that no axial force would be transferred to the sleeve, as a result, due to the mechanism of the prevent local and lateral buckling, this meeting makes the steel core to deform longitudinally. The geometry and materials in this slip sheet must be cautiously designed and built to permit proportional movement between the concrete and the steel element due to Poisson's and shearing effect, although simultaneously prevents localized buckling of the steel as it yields in compression (Sabelli et al., 2003; Kumar et al., 2007; Deulkar et al., 2010).

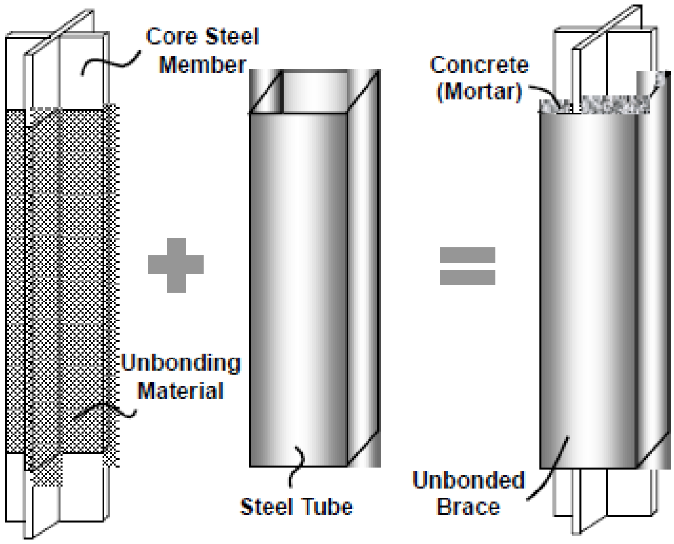


Figure 2.6 Schematic of buckling restrained brace (BRB or un-bonded brace, UB) (Tsai and Lai, 2002)

2.3.2 Component of BRBs

Figure 2.7 shows example of a BRB, A buckling restrained brace usually consists of the following parts:

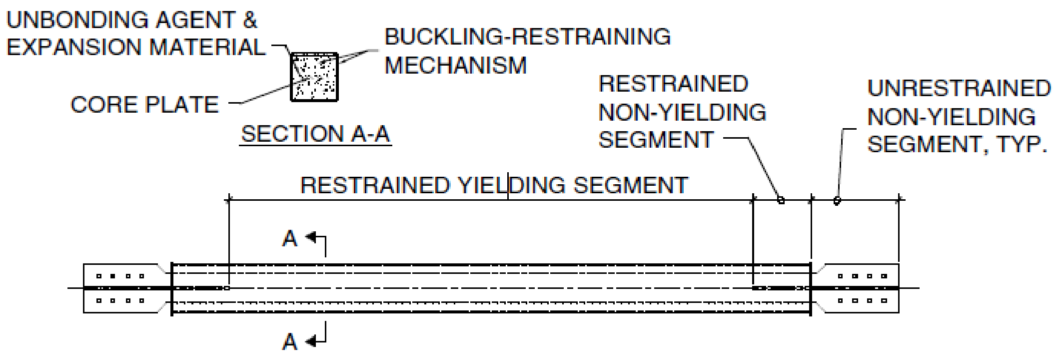


Figure 2.7 Component of BRBs (Wada et al., 1989)

1. Axial force-carrying unit (brace): In cross section this steel segment can be cruciform or rectangular, also this segment named by (restrained yielding segment). Usually BRBs are manufactured with low yield steels.

Previous researches (Kalyanaraman et al., 1994; Kalyanaraman et al., 1988a; Kalyanaraman et al., 1988b; Kalyanaraman et al., 2003) have shown that, to avoid buckling under compression the restricting unit contains sufficient elastic strength.

2. Stiffened transition segment (projection) which connects the brace and connection part. This part is generally an elongation of the axial force-unit (brace) but with an enlarged area to certify response, which is surrounded by the mortar and casing, this can be obtained by expanding the restrained yielding segment.

3. Buckling-restraining unit (encasing member), whose function is to prevent the brace from buckling.

4. Separation unit between brace and buckling-restraining units, which ensures the brace, can slide freely inside the buckling-restraining unit and that transverse expansion of the brace can take place when the brace yields in compression. Also this part called (un-bonded agent and expansion material).

5. Buckling-restraining mechanism: This mechanism is consists of steel casing and mortar. In this mechanism to obtain adequate compressive strength, the mortar must be doing a correct mix design and curing to get sufficient compressive strength of the mortar. Otherwise, if insufficient compressive strength obtains, the mortar cannot prevent buckling amplitude of the restrained yielding brace. Figure 2.8 exhibits a sample of the sticking out of steel casing due to insufficient strength of the mortar.

Figure 2.8 Bulging of steel casing (courtesy of Star Seismic, LLC) (Uang and Nakashima, 2004)

Previous researches (Saeki et al., 1995; Iwata et al., 2000; Yamaguchi et al., 2000; Black et al., 2002) has shown that the BRBs exhibit symmetric hysteresis behavior with high energy dissipation capacity through stable tension-compression yield cycles, as shown in Figure 2.9 and 2.10 shows the response and a number of buckling restrained braces respectively, awaiting testing at the Japanese E-defense shake table facility.

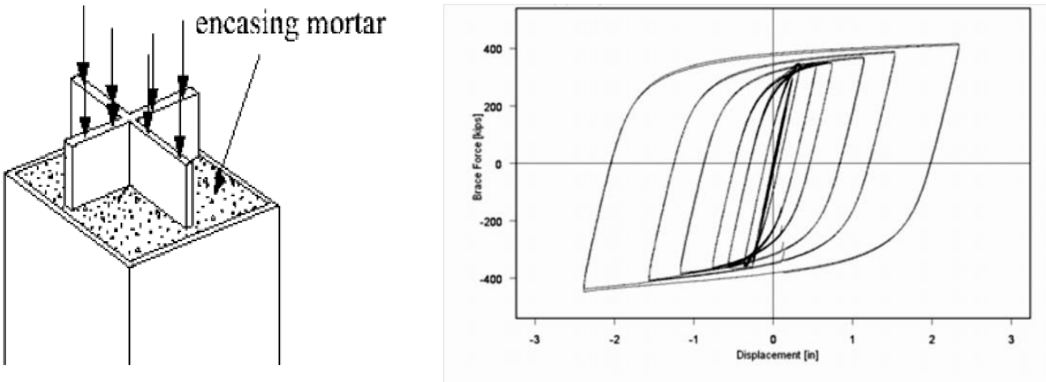


Figure 2.9 Detail and response of BRBs (Di Sarno and Elnashai, 2009)

Figure 2.10 Un-bonded braces (BRBs) a waiting testing at E-Defense (Abraham, 2006)

The BRBs subjected to many experimental tests (Nakashima et al., 2000; Tremblay, 2002; Bozorgnia and Bertero, 2004; Broderick et al., 2005), they have shown that due to the confinement effect of the restraining unit, the steel core brace can undergo an axial compressive strength about 10% to 15% greater than tensile capacity. Moreover, as shown in Figure 2.9, the initial yield deformation of the brace failure 300 times smaller than inelastic deformation (ductility) capacities (Sabelli et al., 2003; Di Sarno and Elnashai, 2009).

2.4 Previous research on BRBs

Yoshino and Karino (1971) carried out the first cyclic loading on the BRB test, forming an X-brace, as shown in Figure 2.11. They tested two different configuration for concrete panel that they called "*shear wall with braces*" under cyclic loading, each specimen consists of flat steel plate surrounded by reinforce concrete panels, the bond between them was broken by coating the steel plate with a debonding material, one of the specimens was provided with a 15 mm gap between the surrounding panel and the panel lateral sides, while no space is left in the second specimen.

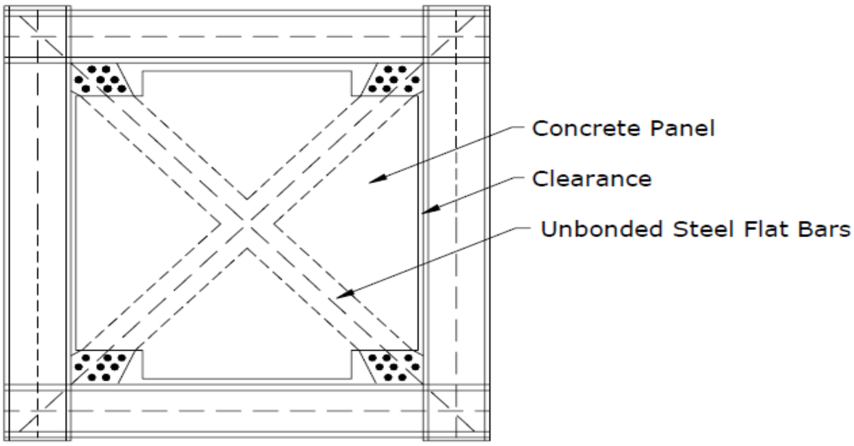
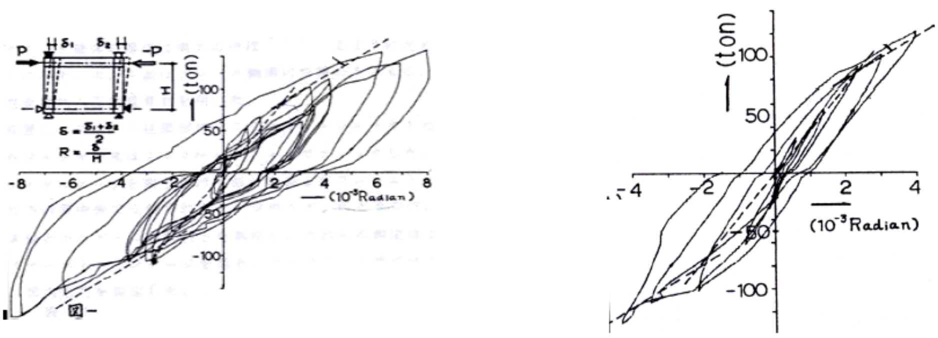


Figure 2.11 General configuration of the model (Yoshino and Karino, 1971)



a) b)

Figure 2.12 Response comparison between two specimens: a) with internal clearance and b) without clearance (Yoshino and Karino, 1971)

In the test result, the debonding material used is not specified. Figure 2.12 presents the hysteresis cycles for the specimens. Energy absorption, larger axial load, and ultimate displacement were enhanced in the specimen provided with the internal gap, highlighting the advantage of the mentioned clearance (Yoshino and Karino, 1971).

A pioneering research carried out by (Wakabayashi et al., 1973a), in their study, steel flat plates were used for the braces and they were surrounded by RC panel with a debonding material between them. They concluded from the test result that breaking the bond between the brace and the surrounded panel would make the brace carry considerable axial force while the surrounding RC panel served just to restrain the buckling deformation of the brace. Figure 2.13 illustrates typical configuration of the specimens.

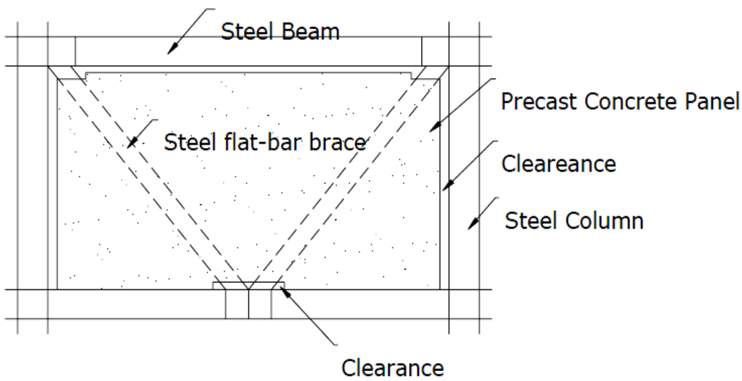


Figure 2.13 Typical configuration of a steel brace restrained against buckling by lateral reinforced concrete panels (Escudero, 2003)

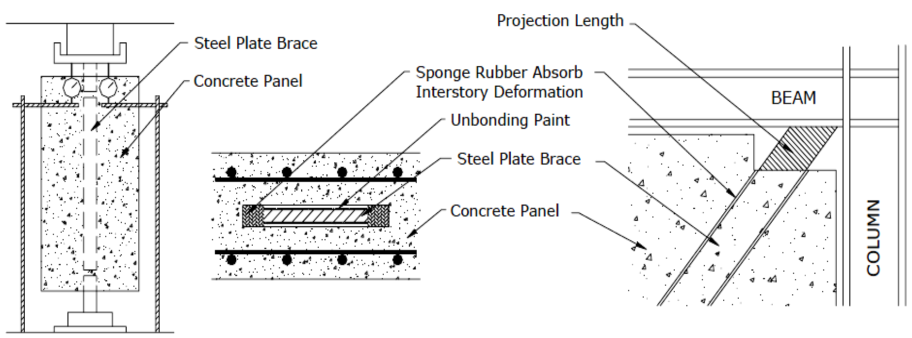
Their study consisted of the following multi-step experimental plan:

- For examining the unbonding effect, tests on the debonding material were conducted,
- Tests on brace were conducted to explore the effect of strengthening of PC panels with steel reinforcement and reinforcement at borders and nearby the plates,

- Decreased scale experiments on brace systems encased by PC panels, and
- Tests on large-scale two-storey structures with the suggested brace systems.

A debonding method consisting on a layer of silicon resin, epoxy resin, vinyl tapes, etc. were tested for exploring the effect of debonding and taking into account of other factors such as material durability and construction feasibility. Pull out tests were conducted on eleven samples with different debonding materials. The method of debonding of coating a layer of silicon resin on top of a layer of epoxy resin was utilized in the following tests to measure the effectiveness in reducing the bond stress and therefore the friction between the concrete panel and steel plate.

Twenty one specimens with different details of the plate reinforcement and details between the exposed and embedded parts (styrol foam, gaps) were tested under monotonic compressive force, as shown in Figure 2.14. From the tests results, they found that it was important to put small styrol foam in the gap and should be adequately sized in order not to restrain the stiffened ends from deformation in the PC panels and a large gap will allow the occurrence of local buckling. At the ends of the panel the tests also showed the effectiveness of reinforcement (Wakabayashi et al., 1973a).



a) b)

Figure 2.14 Views of a) monotonic test setup and b) gap disposition at the brace end (Wakabayashi et al., 1973a)

Fourteen half scale of X-brace and diagonal brace frame systems with bonded and un-bonded specimens were tested under cyclic loading in order to characterize the hysteretic behavior of BRB, Figure 2.15 and 2.16 shows the test set up and the hysteresis behavior for the diagonal-shaped steel core braces encased in reinforced concrete panels used in the studies of (Wakabayashi et al., 1973a; Wakabayashi et al., 1973b).

They concluded from the test results that bonded braces exhibited a smaller than the performance obtained for the un-bonded braces. Maximum lateral drift angle in the case of un-bonded brace was about of 0.03 rad which it was almost four times larger than that of the bonded brace. In restraining the lateral buckling of the steel brace concrete panels reinforced with spiral hoops showed to be effective.

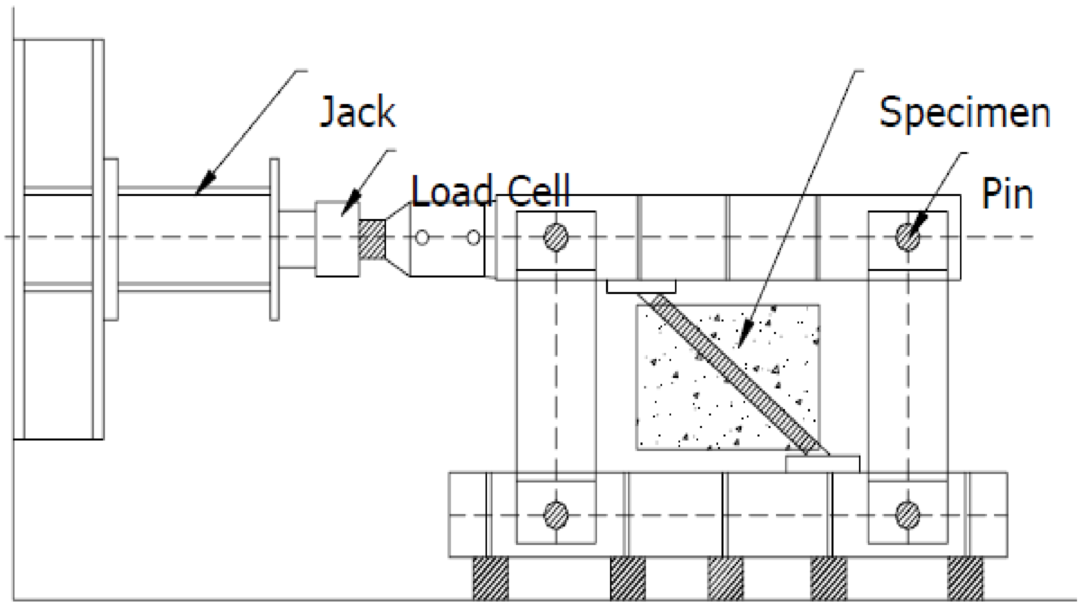


Figure 2.15 Cyclic loading test setup (Wakabayashi et al., 1973a; Wakabayashi et al., 1973b)

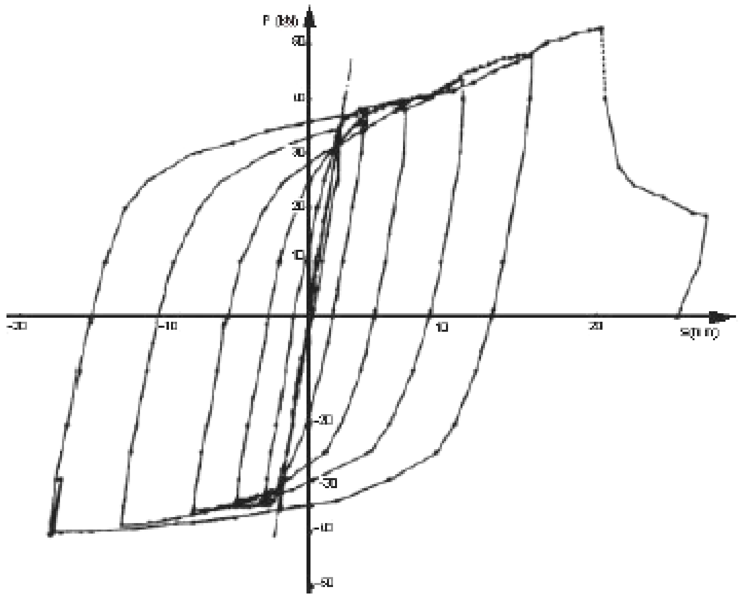
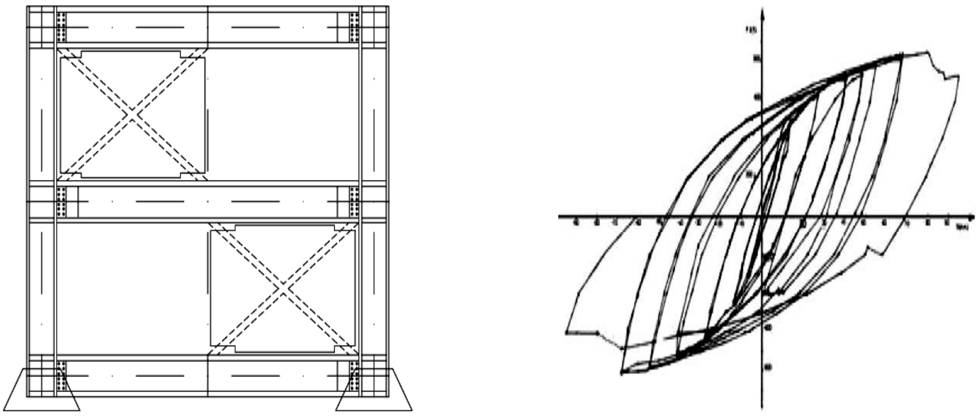


Figure 2.16 Hysteresis behavior in one of the test specimens (Wakabayashi et al., 1973a; Wakabayashi et al., 1973b)

In order to check the behavior of the BRB in real steel frames, two steel frame specimens (two stories and tow bays) with the braces located on the first and second stories, with a scale of 1/2 were also tested for the final demonstration, as shown in Figure 2.17. In the second storey cyclic horizontal force was applied on the center of the beam, using a push-pull jack.

According to the storey angle loading cycles were controlled. They observed that the behavior of the frames and hysteretic loops were stable, and good energy dissipation capacity before local buckling occurs in the steel brace at a lateral angle of drift of 0.025 rad, in the elastic stage of the brace, crack present in the concrete panel, at a drift angle of .001 rad the first crack along the axis of the brace was appeared (Wakabayashi et al., 1973a; Wakabayashi et al., 1973b).



a) b)

Figure 2.17 View of a) frame test using X-shaped steel brace core and b) hysteretic behavior of X-shaped BRB (Wakabayashi et al., 1973a; Wakabayashi et al., 1973b)

Kimura et al. (1976) conducted the first test on steel braces surrounded by mortar-in filled steel tubes under cyclic loading. However, no debonding materials were utilized for providing a slip surface for the brace. The encasing mortar in filled steel tube had showed some effect of preventing the buckling deformation of the steel brace core. The measured longitudinal strains in the steel tubes were approximately 10% to 15% of the longitudinal strain in the core braces. The test results had showed that the core brace could undergo an axial compressive strength greater than tensile strength.

In the subsequent research (Kimura et al., 1976), they tested four specimens shown in Figure 2.18 with full scale under cyclic loading but this time two of the specimens had some slit between the encasing mortar and the core brace, the results showed that the good stable hysteretic cycles and the core brace would not exhibit any buckling deformation and dissipate a considerable energy if the ratio of the Euler limit of the steel tube to the yielding strength of the core brace is greater than 1.9.

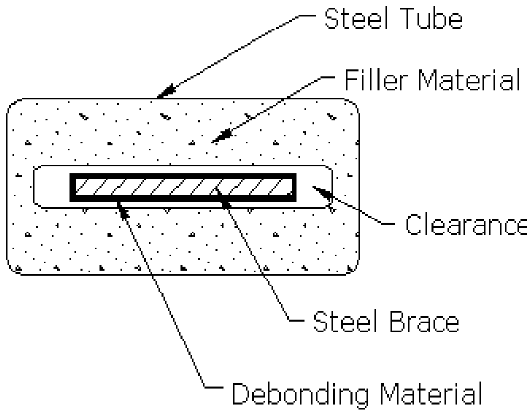


Figure 2.18 Typical cross section tested by Kimura et al. (1976)

Figure 2.19 shows the behavior of different results obtained for two of the specimens with the same steel core cross-section, but different concrete compressive strength.

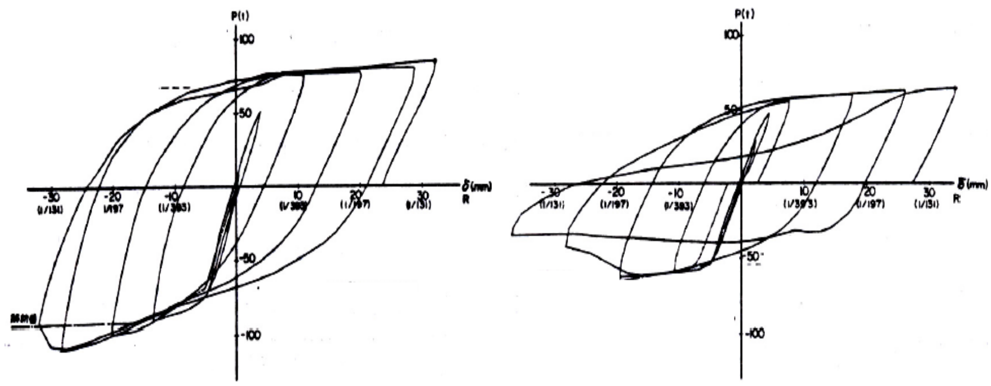


Figure 2.19 Hysteresis behavior result (Kimura et al., 1976)

Takahashi et al. (1979; 1980; and 1982) performed some experimental tests on the composite BRBs consisting of un-bonded braces encased in reinforced concrete square cross-section members under axial compressive loads, extending the work by (Tani et al., 1962). Figure 2.20 shows the general setup used in the tests and the different cross sections. In their study, a coefficient factor that represents the stiffness degradation of concrete panel after it cracks was used.

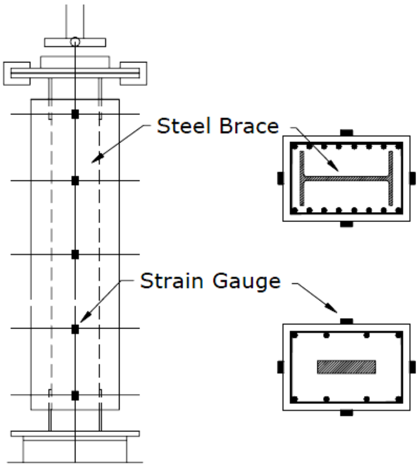


Figure 2.20 Example of cross section used in 1979 (top left), in 1980 and 1982 (bottom left) and general specimen test setup (Takahashi et al., 1979; 1980; and 1982)

In the studies of Fujimoto et al. (1988) and Watanabe et al. (1988a), a slip surface was provided for the core brace so that no axial force would be transferred to the restraining unit. Figure 2.21 shows the dimension and cross-sections of the specimens.

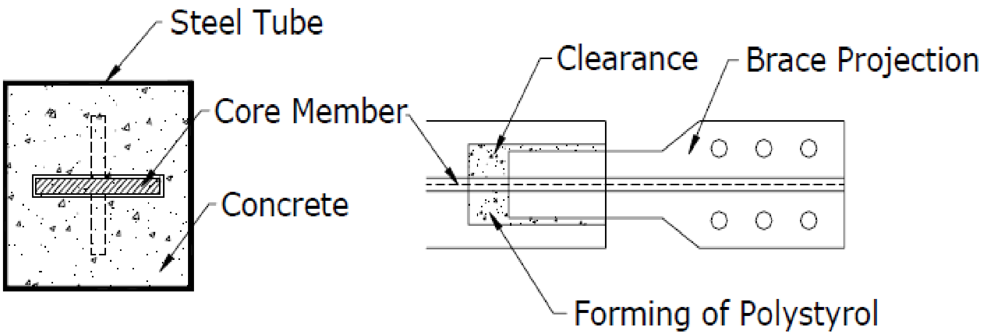


Figure 2.21 General configuration of the models (Fujimoto et al., 1988)

Figure 2.22 shows the behavior of the test results. It could be seen from the test results that specimen No. 3 did not exhibit buckling deformation when loaded with a large compressive force and showed a spindle hysteresis behavior that dissipate large energy. On contrary, specimen No.4 due to insufficient restraining, global buckling had occurred before reaching its yield load. When the Euler buckling load of the encasing member is larger than the yielding load of the steel core, hysteretic cycles were stable and yielding occurred at the yield load of the inner steel core. The important thing in the tests is that the friction force between the encasing member and the steel brace were almost eliminated.

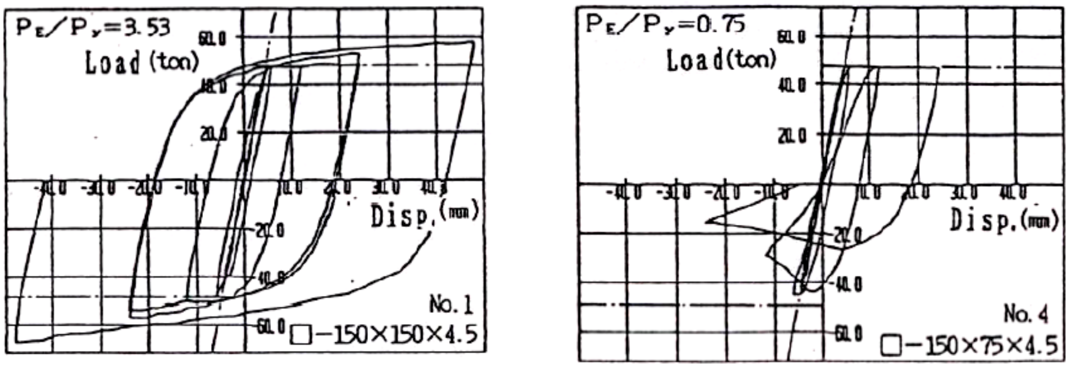


Figure 2.22 Test results for two different configurations (Fujimoto et al., 1988)

Nagao et al. performed some experimental and theoretical analyses on composite BRBs composed of square steel tubes (braces) or H-section steel cores covered by reinforced concrete members (Nagao et al., 1988; Nagao et al., 1989; Nagao et al., 1990; Nagao et al., 1991; Nagao et al., 1992). Figures 2.23 and 2.24 show different specimen's tests and test results.

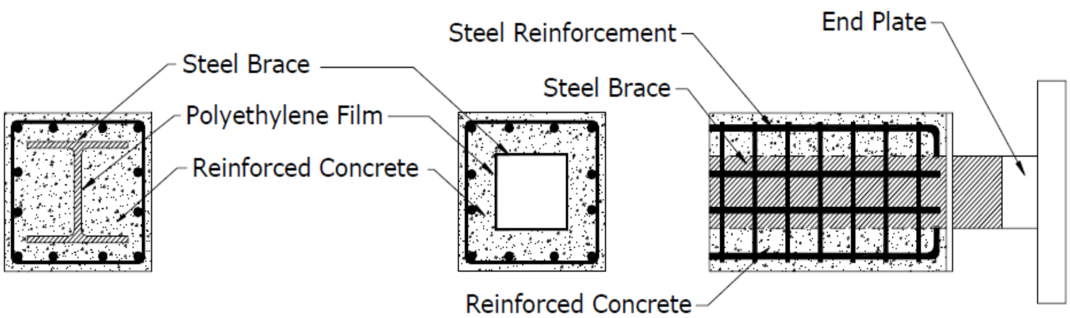


Figure 2.23 Specimens analyzed and tested (Nagao et al., 1988 to 1992)

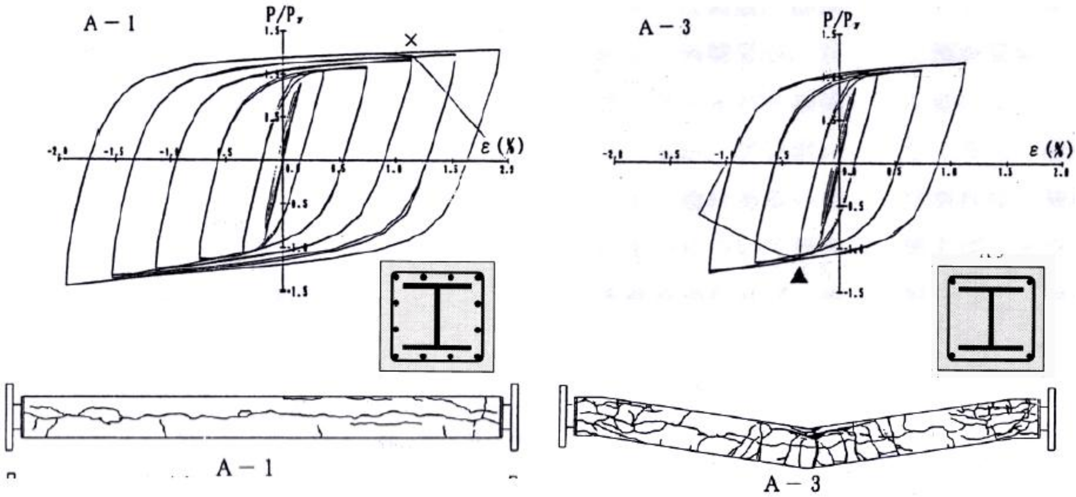
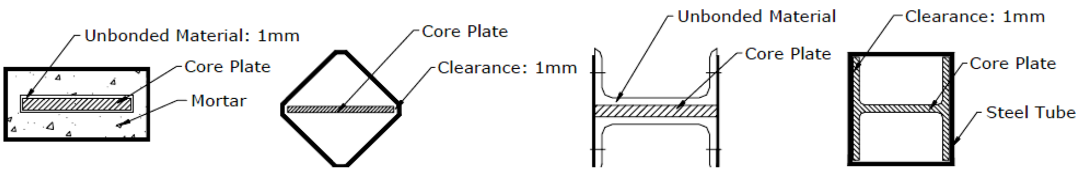


Figure 2.24 Test results for two different specimens (Nagao et al., 1988 to 1992)

Iwata et al. (2000) conducted experimental comparisons between 4 types of commercially available BRBs, shown in Figure 2.25. A 1 mm gap was left around the perimeter of the all specimens, In the 1 and 3 specimens where filler material is presented, and to mitigate the effect of friction forces with the brace core the debonding materials were applied to allow relative displacements between members. Specimen 3 and 4 consists of steel flat bar brace and steel wide flange section respectively.



a) Type 1 b) Type 2 c) Type 3 d) Type 4

Figure 2.25 Cross section of the model (Iwata et al., 2000)

Figure 2.26 shows the behavior of the test specimens were tested by (Iwata et al., 2000), based on hysteresis curves, tests conducted the specimen 1 exhibit the best behavior followed by specimens 3, 4 and 2. However, all braces performed well under the 1% strain limit.

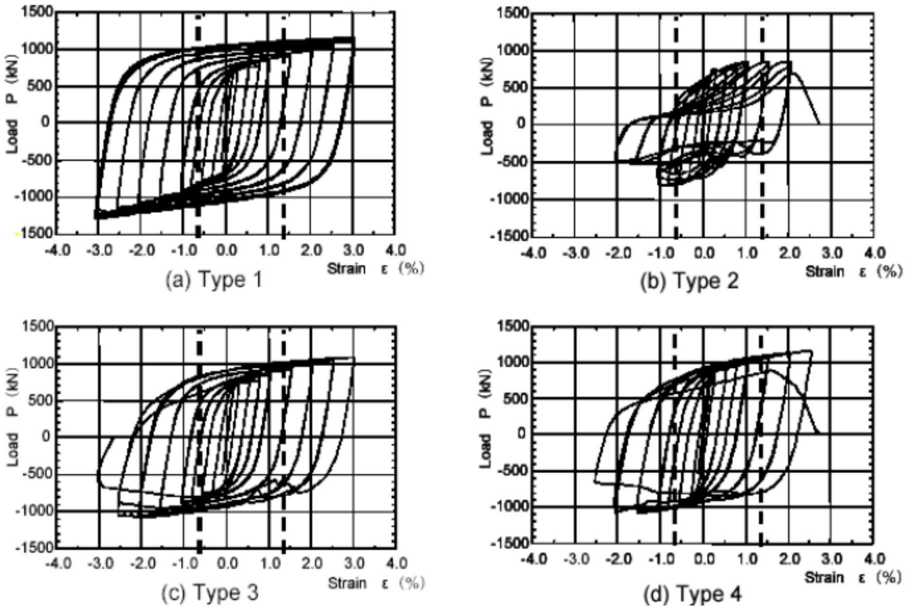


Figure 2.26 Hysteresis cycles for the four specimens (Iwata et al., 2000)

The effects in the response of a frame system when BRBs are inserted have been also studied. The following is a description of some tests performed to describe the braced frame response with BRBs. Yamaguchi et al. (2002) conducted a full scale test. Considering the behavior of both conventional moment resisting frames and braced moment resisting frames. A system provides the same natural vibration period than a medium rise building and a shaking table these are referred as a test setup. Figure 2.27 shows the schematic procedure of the test.

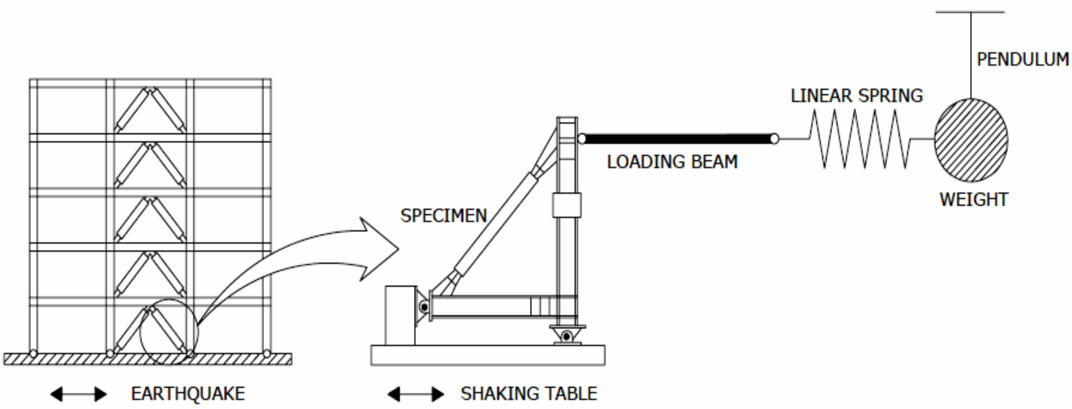


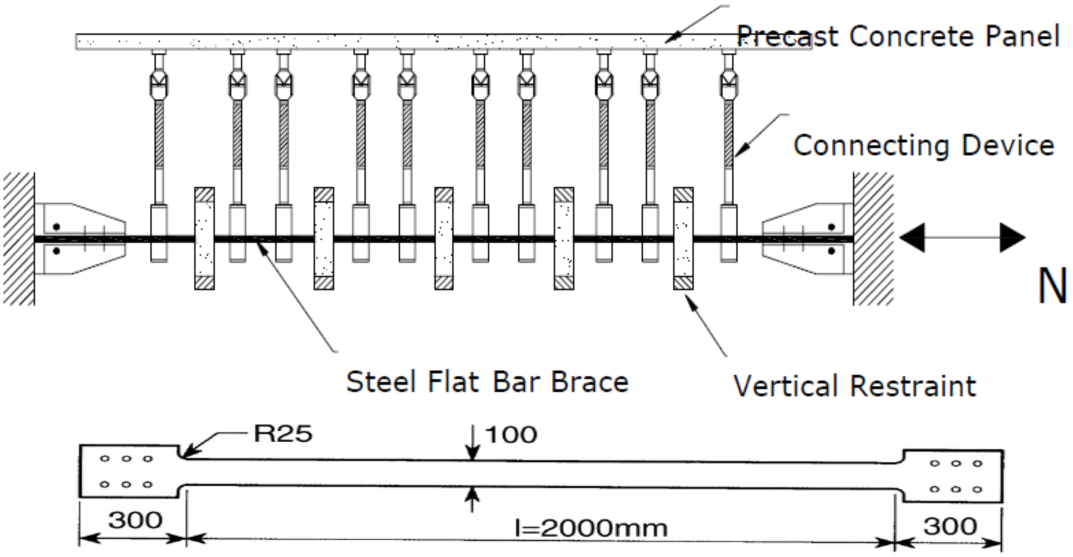
Figure 2.27 Schematic procedures of tests (Yamaguchi et al., 2001)

Kumar et al. (2007) displayed the effects of an analytical study carried out to get the seismic response of multi-storey moment resisting and non-moment resisting frames designed with non-buckling bracing systems. Lin et al. (2010) also evaluated the seismic design and behavior of buckling restrained braced frames and eccentrically braced frames in similarity with that of MRFs.

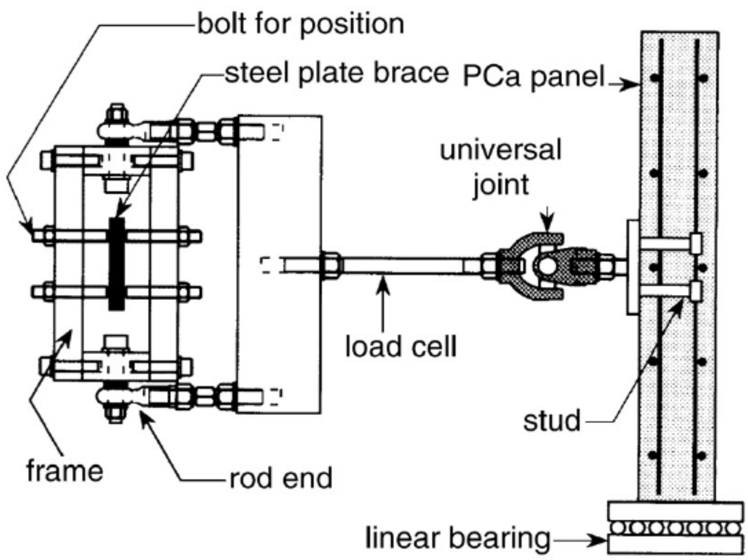
In order to prove the overall behavior of the brace, an experimental tests consisting of 9 specimens was taken. Figure 2.28 shows the experimental setup, it was conducted on panel BRBs and steel flat bar brace in order to confirm the global buckling restraining criterion. At that test, the steel plate brace was separated from the PC panels. Special bolt (Link devices) was used to connect them together and they were installed at interval of 10 cm which allow the modification of the initial deflection of the specimen. Then, the steel plate was subjected to axial load. The aims of separating the PC panels from the steel plate brace were (Inoue et al., 1992; Inoue et al., 1993; Inoue et al., 2001):

- To measure the stiffening force distribution of the brace directly, and
- To adjust steel brace initial deflection arbitrarily.

As shown in Figure 2.28, the rod end and universal joint were installed at both ends of the linking device and to measure the force in the brace, load sensors were installed at each linking device. Linear bearing that could slide freely in the direction orthogonal to the steel plate plane was used to support the reinforced concrete panels. The initial deflection can be changed through the location-adjustable bolts shown in Figure 2.28. By applying an initial bending moment of about 4% the yielding moment of the concrete panel the initial deflection was obtained.



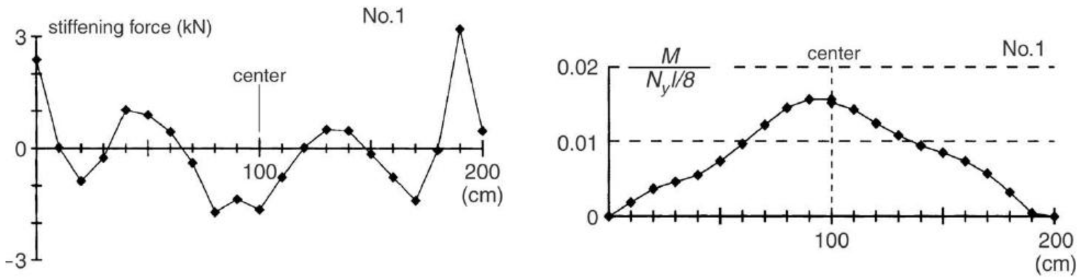
a)



b)

Figure 2.28 Test on panel BRB a) specimen and loading system and b) linkage between steel plate and PC panel (Inoue et al., 1992)

Moment diagram and brace force distribution at the instance of buckling of the steel plate brace is shown in Figure 2.29. Bending moment M in the precast concrete panels was determined based on the force distribution in the steel plate brace. Force distribution of the brace was very complicated, but at the ends of the brace the value was large. The figure illustrates the bending moment distribution of the precast concrete (PC) panel and it is similar to the initial deflection distribution of the steel plate brace. At mid of the precast concrete panel, maximum bending moment occurred. Moreover, if the divided load is assessed as equally to the peak moment, the bracing force is around 1.5% of yielding axial strength (Inoue et al., 1992). Depending mainly on the particular configuration of the specimens, some of them were just at the limit or below the stiffening requirement whilst the others exceeded the stiffening requirement.



a) b)

Figure 2.29 View of a) stiffening force distributions and b) moment-bending of PC panel at overall buckling (specimen B9) (Inoue et al., 1992)

V-shaped buckling restrained braced frames (BRBFs) of three large scale single bay conducted to assess the performance of the double-T to gusset connection details, constructed with the introduced BRBs have been experimented in NCREE (Huang and Tsai, 2002). The purposes of the study contain the following item:

- Investigating the analytical and experimental responses of the V-shaped single bay BRBFs each constructed with two BRBs in three different length aspect ratios,
- Investigating inter-story drift relationships versus the steel BRB core strain, and
- Providing guidelines for severe seismic applications for the analysis and design of BRBF.

The frame elevations of these specimens are demonstrated in Figure 2.30.

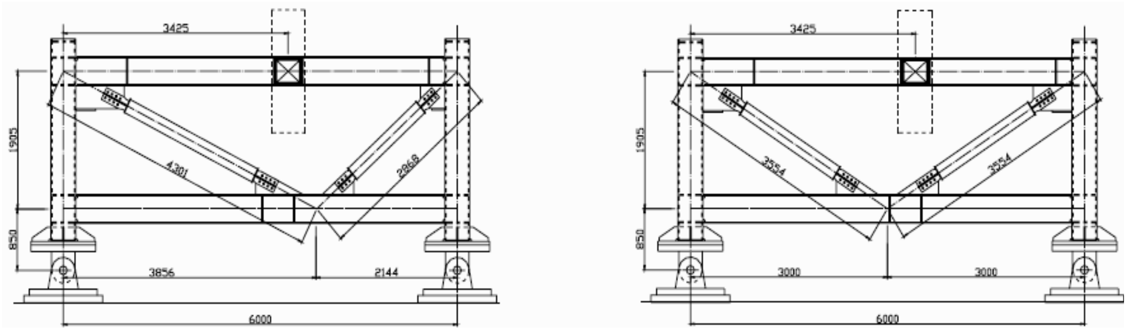
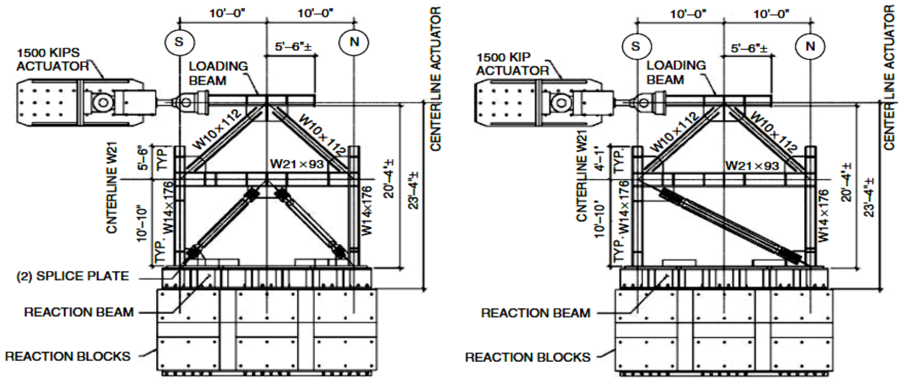


Figure 2.30 Elevation of the specimens for the test (Huang and Tsai, 2002)

The recommended provisions recognize the potential for important interactions between the surrounding structural frame and buckling-restrained brace, and therefore define tests of individual braces (which may involve only uniaxial loading), as well as brace sub-assemblage tests (which must incorporate axial and rotational demands). Tests soon to be performed at the University of California, Berkeley, in support of the use BRBs in a new laboratory building, involve a frame sub-assemblage containing both single-diagonal and chevron configurations of BRBs (see Figure 2.31). The provisions define a loading program that consists of fully-reversed cycles of loading at increasing amplitudes of deformation. In the test, the maximum brace deformation shall be at least 1.5 times the brace deformation comparable to the design story drift. Alternative loading protocols may be used, so long as they are shown to be of equal or greater severity in terms of maximum deformation and cumulative plastic demand. An acceptable brace test is one in which the test specimen shows increasing force with increasing deformation, there is no fracture, brace instability or brace connection failure, and the ratio of maximum tension force to maximum compression force is with a specified limit (Aiken and Kimura, 2001).



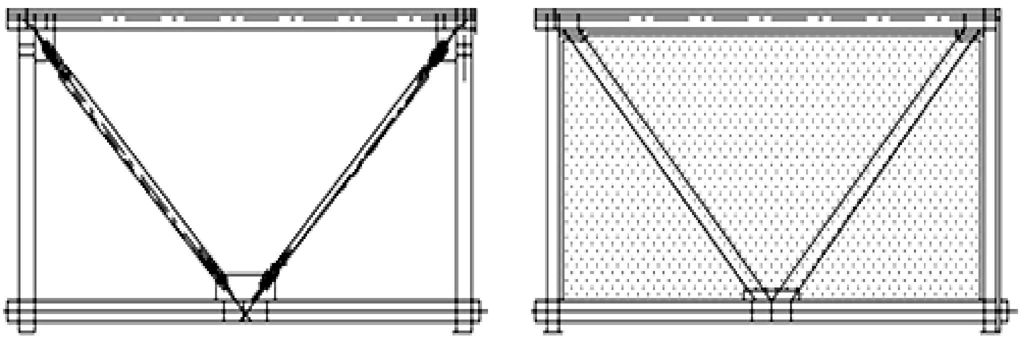
a) b)

Figure 2.31 Subassembly test setup a) test 1 and b) tests 2 and 3 (Lopez et al., 2002)

2.5 Configurations of BRBs

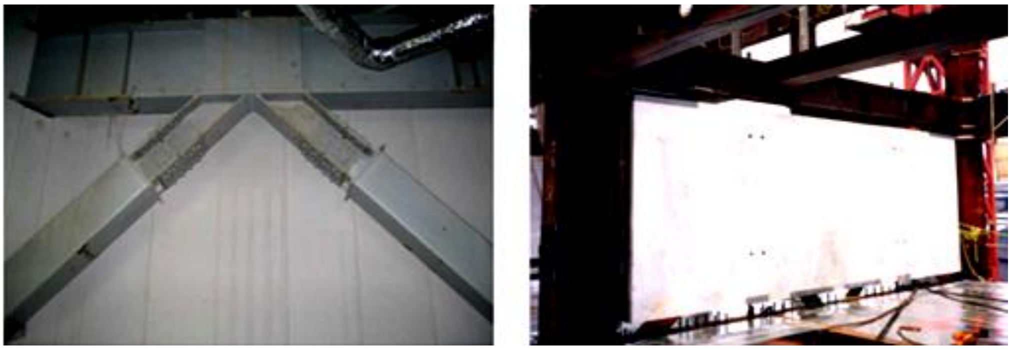
As shown in Figures 2.32 and 2.33, in general, BRBs categorized into two main and wide covering types:

- One typical type is a steel brace restrained by reinforce concrete or combination of concrete and outer steel member, and
- The second one is a steel plate restrained by PC panels.



a) b)

Figure 2.32 Views of buckling-restrained braces in frame system a) normal BRB layout and b) panel BRB shape (Qiang, 2005)



a) b)

Figure 2.33 Photos of BRBs a) normal tube of BRB and b) panel BRB (Qiang, 2005)

Figure 2.34 shows several standard cross-sections of the BRBs introduced by various investigators (Qiang, 2005). BRBs or un-bonded braces UBs generally construct are manufactured from encasing a flat bar member into a steel tube or core steel cross-shape and confined by infill concrete. It is seen that the cross-section of the steel core member is usually bi-axially symmetric, can be a cruciform, an H or a flat bar shape. The buckling restraining part can be constructed from mortar filled in the tube, reinforced concrete, reinforced concrete covered with fiber reinforced polymer (FRP) or all-metallic steel tubes.

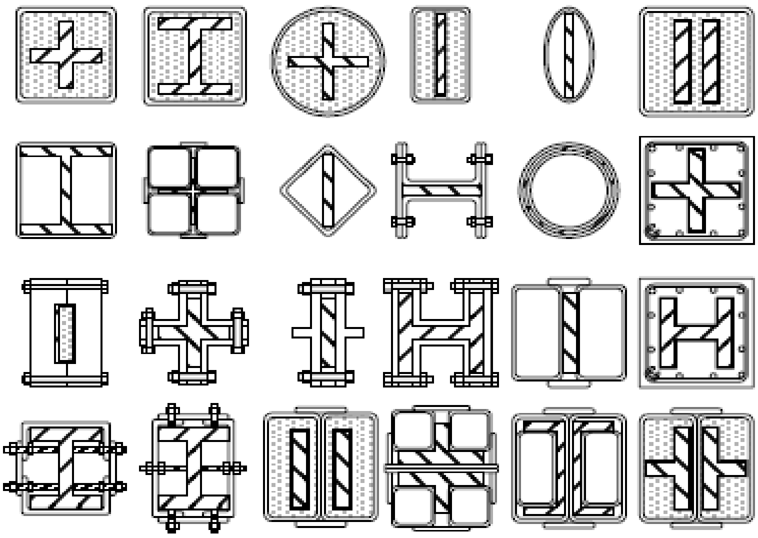


Figure 2.34 Typical configuration of BRBRs (Tsai and Lai, 2002)

2.6 Example application of BRBs

As discussed previously, the early invention of BRB was in Japan by (Wakabayashi et al., 1973a; Wakabayashi et al., 1973b) and then many researches took place on the behavior of BRB. As a result of its superior behavior, it was implemented in practical applications; such as: "Raguza Tower" a 26 storey building in Osaka which had utilized BRBs encased by PC panels,″ Passage Garden" and "Harumi 1 chome" in shibuya, Tokyo. Utilized BRB encased by reinforce concrete member (Qiang. 2005).

Implementation of BRBs was not ended in Japan but also transferred to the other countries such as USA, Italy, Taiwan and etc. The first application in USA was in 1999 for the new laboratory building at University of California Davis. However, nine years later, several of the seismic retrofit of existing buildings and new high-rise steel frames utilized BRBs. Now about 150 building structures in USA utilized BRBs as seismic load resisting systems. Construction projects to date include those manufactured by Corebrace, Nippon Steel Corporation, Star Seismic (Walterio and López, 2008). Figures 2.38 and 2.35 show some of the projects in USA manufactured by Corebrace. To enhance the seismic performance of frames many retrofit projects and new projects in the world have selected difference type of BRBs as the energy dissipation elements. A 46-storey office frame in Tai-Chung, Shee-Hwa United World Tower is planned before 1999, being seismically promoted to provide an increased seismic hazard level, a double-cored BRBs are being fixed in the two opposite circumference bays along the longitudinal direction of the structure, the total piece of BEBs was used are 80 pieces.

Figure 2.36 also exhibits the structural details of Tzu-chi culture building in Taipib of a 14-storey. The number of BRBs was fixed in this building are 96 pieces of a double-core of BRBs (Tsai and Lai, 2002).

A gymnasium building at Chinese Culture University in Taipei of 10 storeys has been proposed and it is constructed. The earthquake force and lateral force prevented by build-up truss moment resisting frames in the transverse direction and mega braced frames in the longitudinal direction was prevent lateral forces, as utilizes in Figure 2.37 (Tsai and Lai, 2002).

Figure 2.35 Application of BRBs (Walterio and López, 2008)

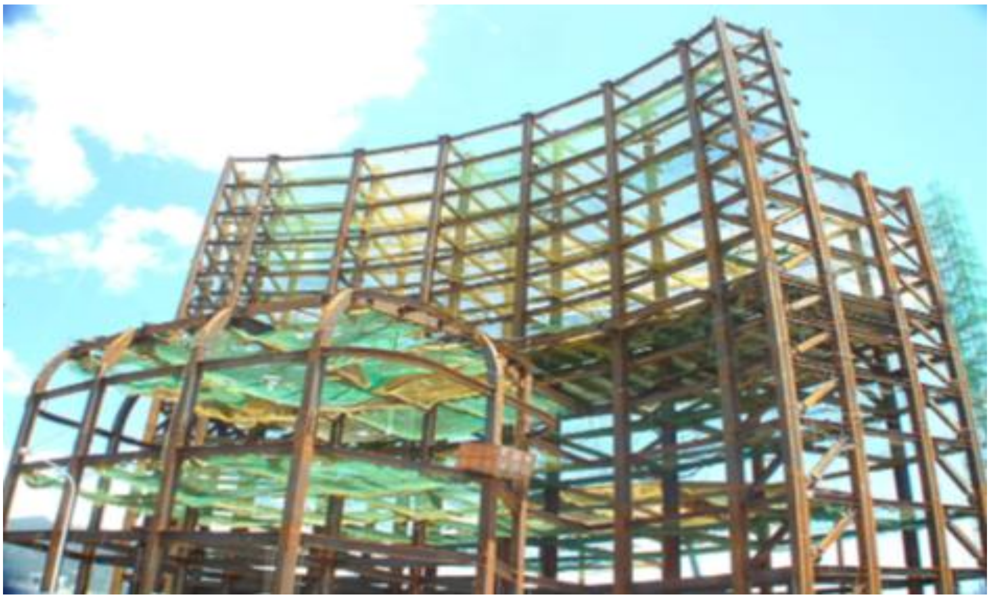


Figure 2.36 Manufacture of the Tzu-Chi culture frame (Tsai and Lai, 2002)

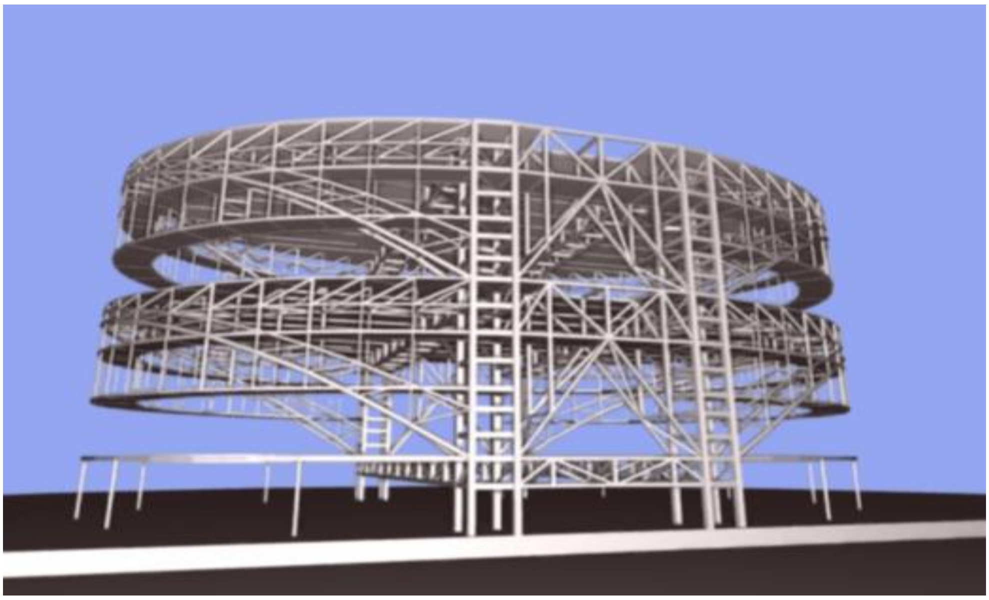


Figure 2.37 Three dimension of the gymnastic (Tsai and Lai, 2002)

Figure 2.38 Use of BRB in world market center III, Las Vegas, Nevada (Corebrace, 2002)

High-rise steel building implementation of dampers in Japan from 1995-1999 is illustrated in Figure 2.39. It can be noticed from the figure that high-rise steel buildings utilized 60% of BRBs.

The ratio of the three main sort of hysteresis dampers which include BRBs, seismic wall, and shear panels implemented in Japan in 2000 is shows in Figure 2.40. It can be focused from the figure that BRBs were the most widely used among the other types (Qiang, 2005).

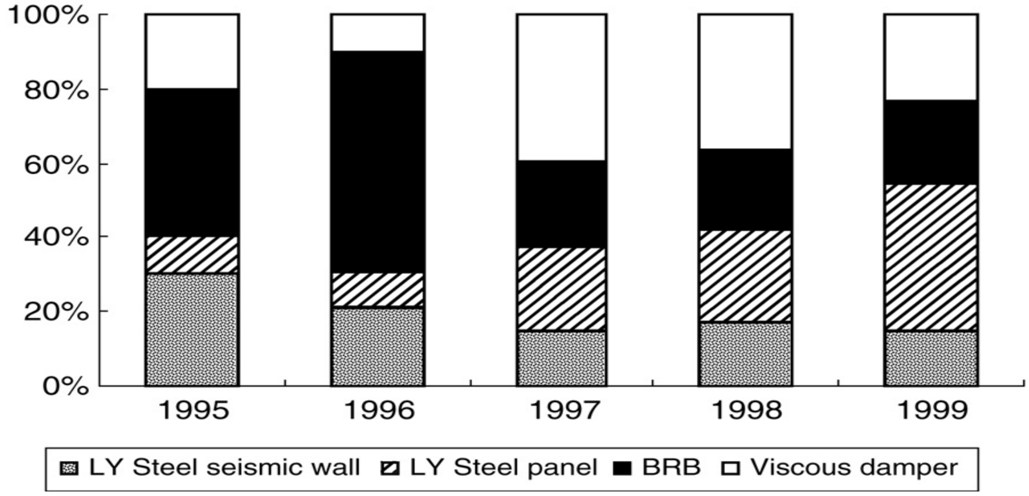


Figure 2.39 From 1993 to 1999 in Japan, BRBs occupation in high-rise steel frame (BCJ, 2002)

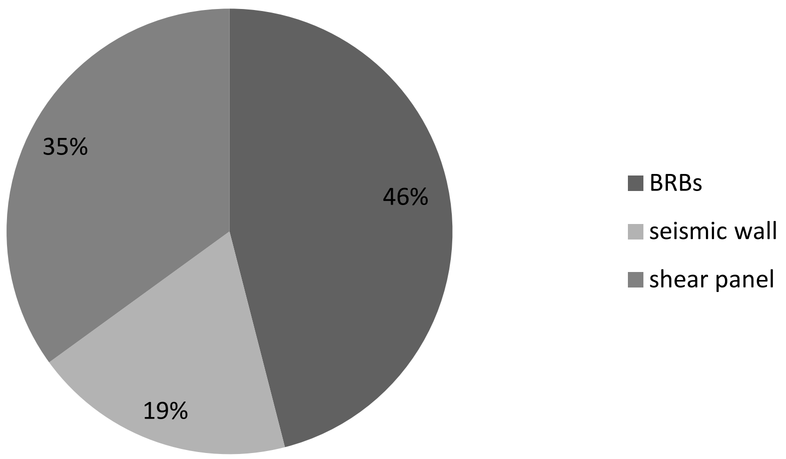


Figure 2.40 Ratio of three kinds of dampers of high-rise steel building in 2000 in Japan (Qiang, 2005)

CHAPTER 3

METHODOLOGY

3.1 Description of the frame structures

In this study, 4 ordinary moment resisting frames with different heights (2, 4, 6, and 8 stories) were considered as a case study. Figure 3.1 illustrates the elevation view and plan of the structure under investigation. They were designed based on UBC97-LRFD code for steel structures with a response modification factor of R=5 (AISC, 1999). A response spectrum curve from IBC 2006 showed in Figure 3.2 was utilized for performing a linear response spectrum analysis and the earthquake region was assumed to be in areas with high seismic zone. The parameters of design spectral acceleration of Ss and S1 are 2.05 s and 0.81 s, respectively, also coefficients of the site are F_a and F_v which are equal to 1 and 1.5, respectively, and important factor I=1 with soil type for the frames is site class D.

The design dead loads are 10 and 12 kN/m^2 for roof and floors respectively, and design live loads are 1.5 and 4 kN/m^2 for roof and floors, respectively, for calculating the seismic load only (D.L+50%L.L) was calculated for floors, for the roof live load was not conducted. The frames are square in plan and 2, 4, 6, and 8 stories which contain four equal 5 m bays in each direction. The inter storey height in the models was 3.2 m for every floors except in the first floor in which the inter storey height was 4.2 m. The beams were built with IPE profile while the columns were taken as HE profile. The column and beam sections varied at the stories in the frames, the section profiles for all frames given in Table 3.1.

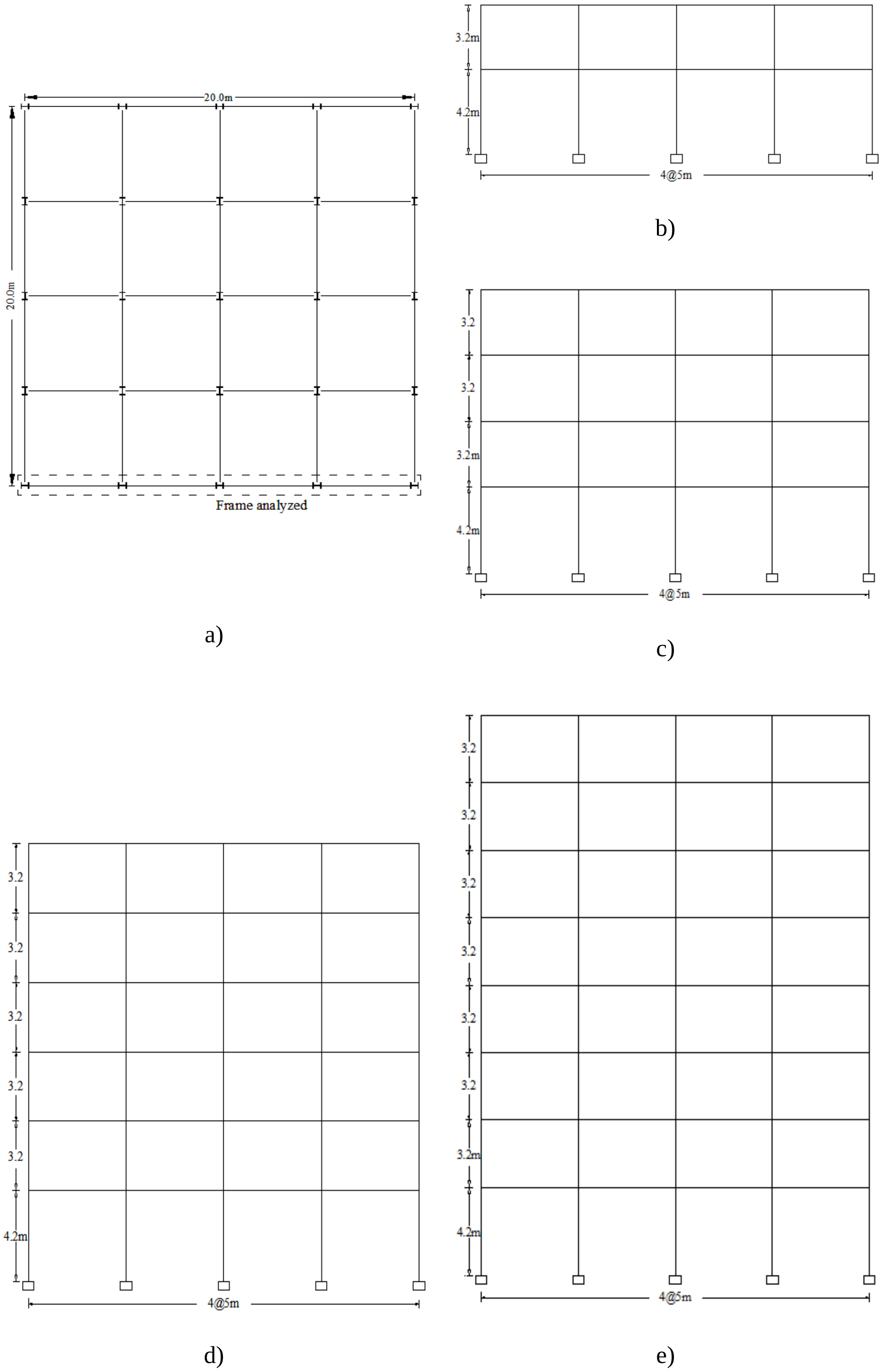


Figure 3.1 Typical plan & elevation of bare frames: a) plan view, b) two storey, c) four storey, d) six storey, and e) eight storey structures

Table 3.1 Properties of 2, 4, 6, and 8 storey ordinary frames

Structure	Columns		Beams	
	Exterior	Interior	Exterior	Interior
2-Storey frames properties:				
1	HE 220A	HE 220A	IPE 300	IPE 300
2	HE 180A	HE 160A	IPE 2400	IPE 2400
4-Storey frames properties:				
1	HE 260A	HE 260A	IPE 300	IPE 300
2	HE 260A	HE 260A	IPE 300	IPE 300
3	HE 180A	HE 200A	IPE 300	IPE 300
4	HE 180A	HE 200A	IPE 2200	IPE 2200
6-Storey frames properties:				
1	HE 240A	HE 300A	IPE 270	IPE 270
2	HE 240A	HE 300A	IPE 270	IPE 270
3	HE 200A	HE 240A	IPE 270	IPE 270
4	HE 200A	HE 240A	IPE 270	IPE 270
5	HE 180A	HE 180A	IPE 270	IPE 270
6	HE 180A	HE 180A	IPE 240	IPE 240
8-Storey frames properties:				
1	HE 260A	HE 320A	IPE 270	IPE 270
2	HE 260A	HE 320A	IPE 270	IPE 270
3	HE 220A	HE 260A	IPE 270	IPE 270
4	HE 220A	HE 260A	IPE 270	IPE 270
5	HE 200A	HE 220A	IPE 270	IPE 270
6	HE 200A	HE 220A	IPE 270	IPE 270
7	HE 180A	HE 180A	IPE 270	IPE 270
8	HE 180A	HE 180A	IPE 240	IPE 240

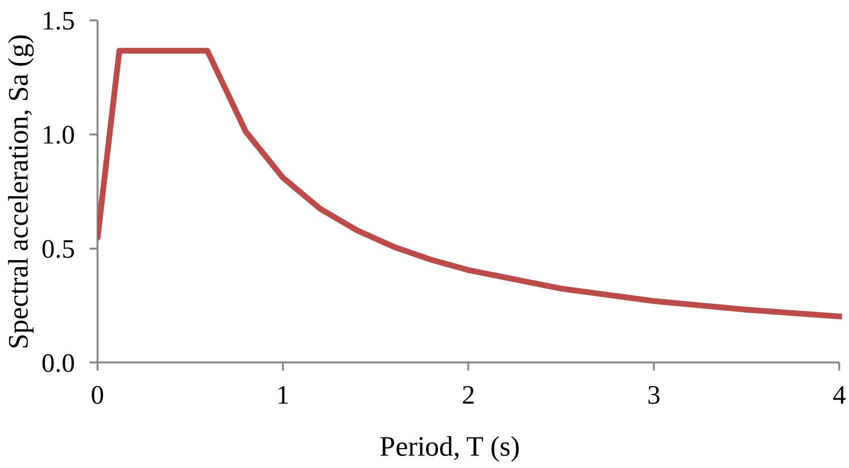
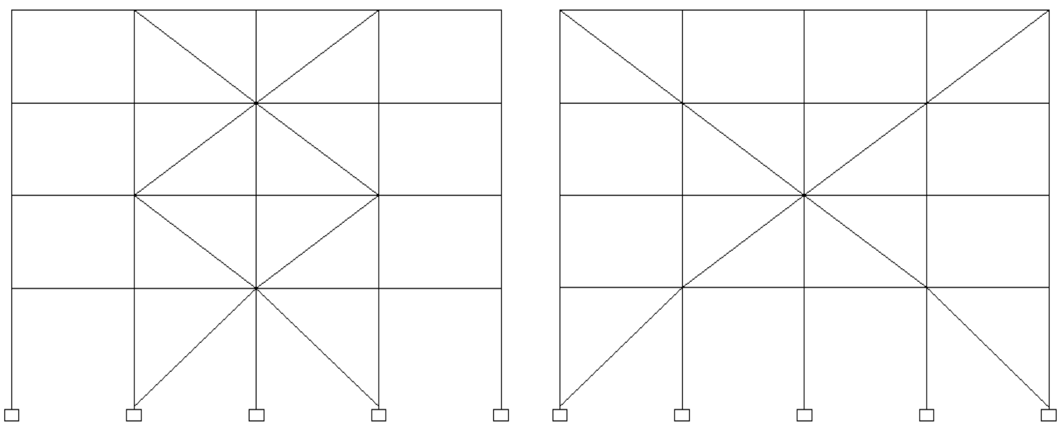


Figure 3.2 Design response spectrum curve (IBC, 2006)

Then, in order to strengthen the 2, 4, 6, and 8 storey existing structures, BRBs were inserted into the bays of ordinary frames considering different configurations such as diagonal (DBF), chevron (CHBF), split-X (SXBF), and V-bracing (VBF) systems. Each of these bracing systems was implemented with four different distributions over the height of the structures. Figures 3.3-3.6 shows the typical elevation of 4 storey BRBFs with different distributions (case-1, case-2, case-3, and case-4). Appendix A illustrates all details of BRBFs.

Thus, a total of 68 different cases were taken into consideration within the scope of this study. The cross-sectional area of cores of the BRBs in first case with the layout of DBF-case-1, CHBF-case-1, SXBF-case-1, and VBF-case-1 were designed with lateral stiffness, satisfying the drift limitations given in FEMA 356 (FEMA 356, 2000) for steel MRF systems in seismic regions. Whereas, for the rest of the other cases and layouts (i.e. DBF-case-2, CHBF-case-2, SXBF-case-3, VBF-case-4, and etc.) the same amount of steel calculated based on first design was used so that their performance could be compared. Tube sections were used for BRBs as the core and lateral support members. Table 3.2 shows the properties of the BRBs together with beams and columns properties when used in the brace frames.



a) b)

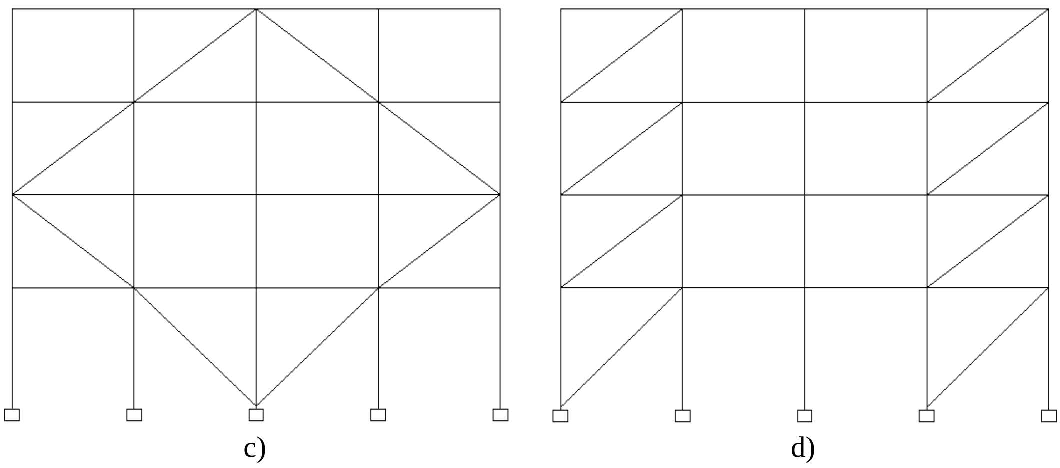


Figure 3.3 Typical elevation of 4 storey BRBFs: a) DBF-case-1, b) DBF-case-2, c) DBF-case-3, and d) DBF-case-4

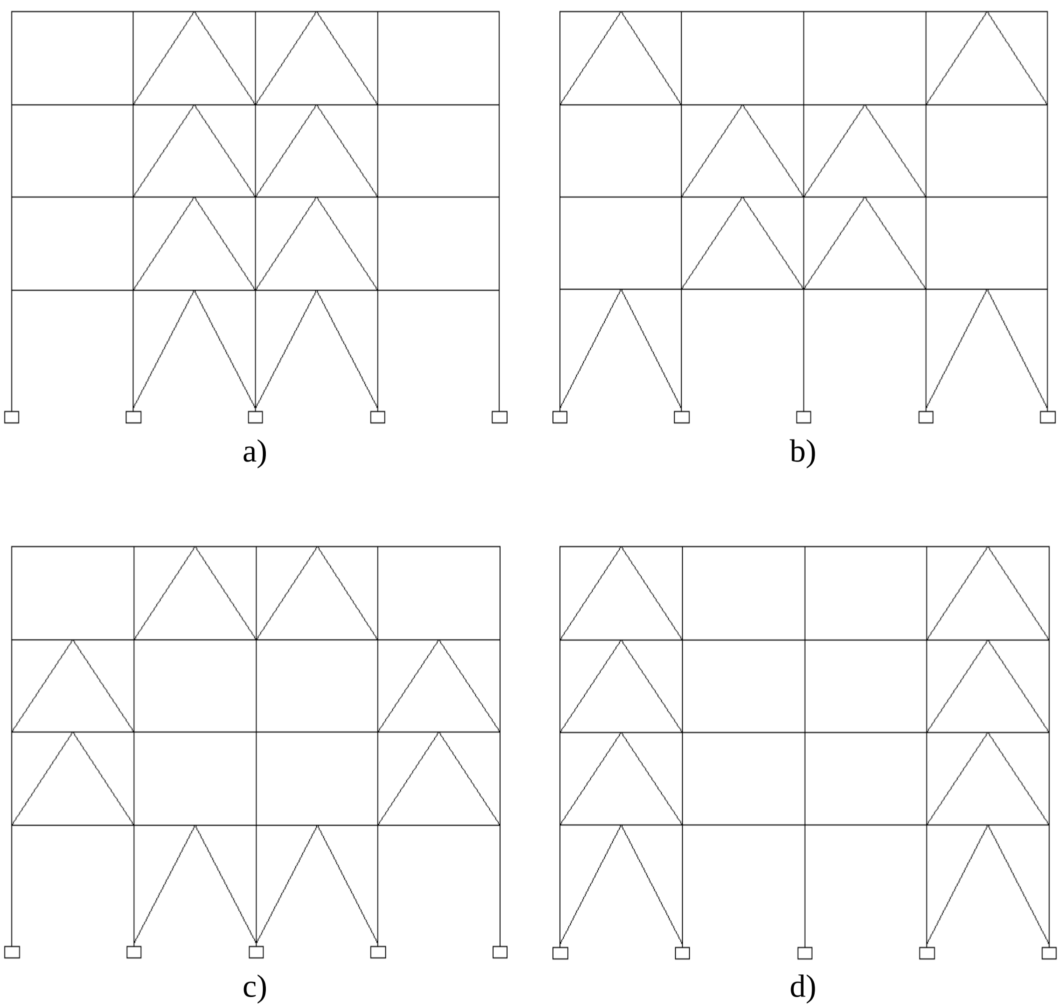


Figure 3.4 Typical elevation of 4 storey BRBFs: a) CHBF-case-1, b) CHBF-case-2, c) CHBF-case-3, and d) CHBF-case-4

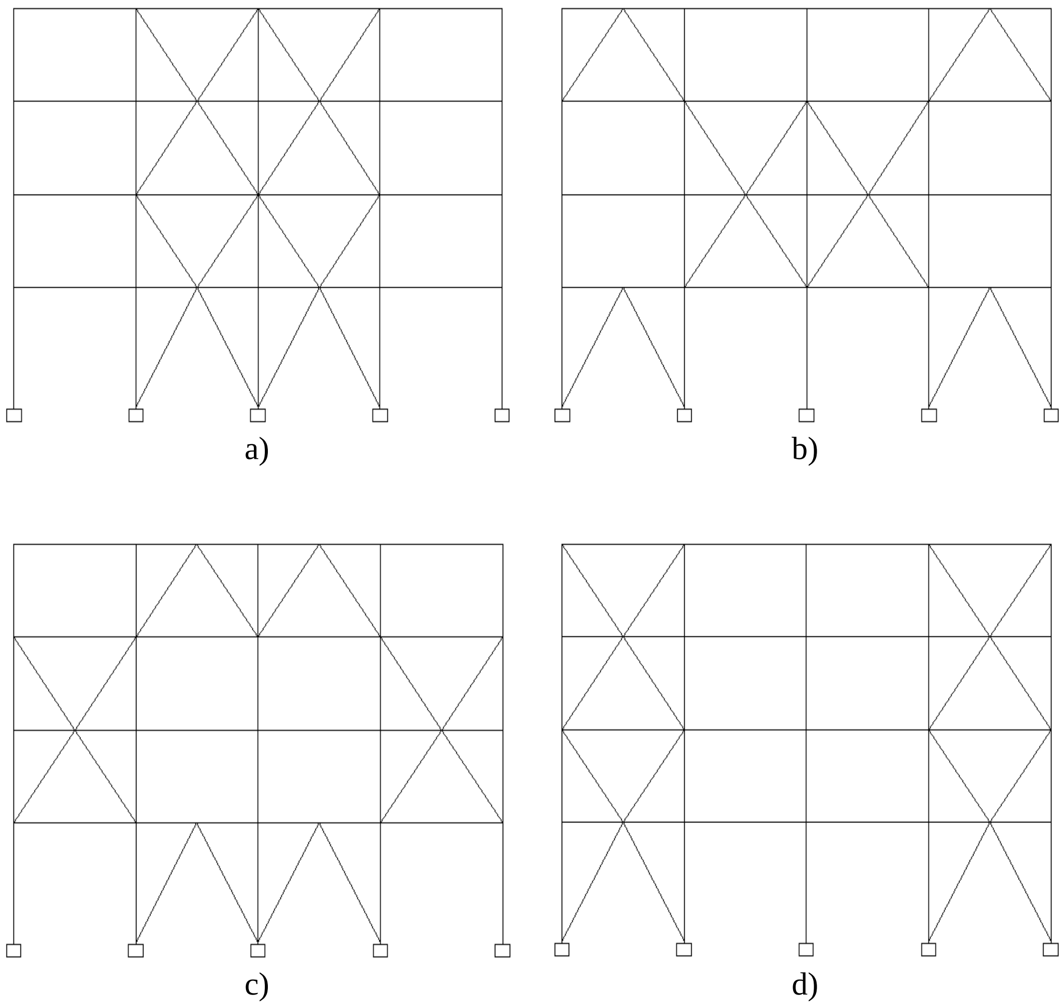
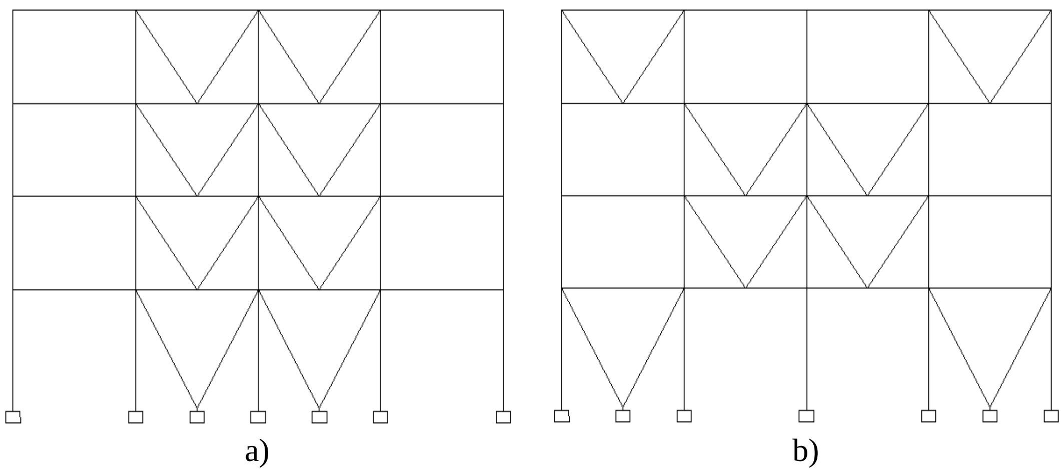


Figure 3.5 Typical elevation of 4 storey BRBFs: a) SXBF-case-1, b) SXBF-case-2, c) SXBF-case-3, and d) SXBF-case-4



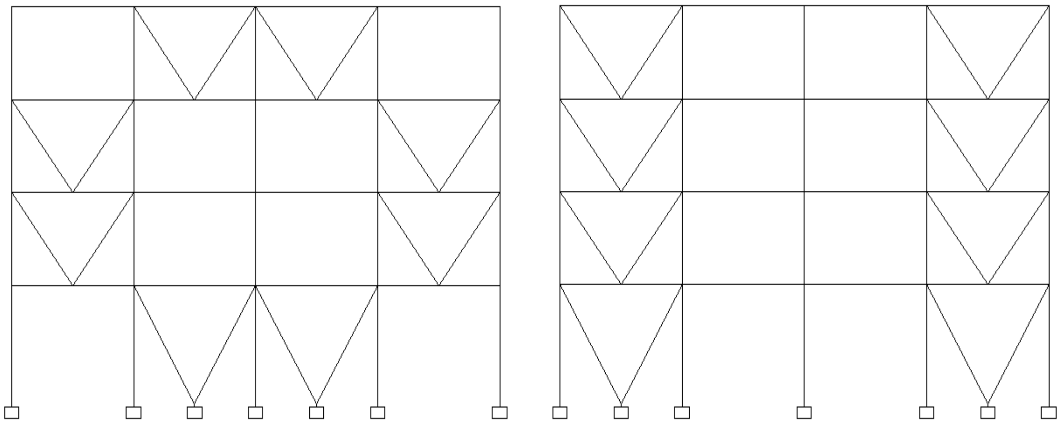


Figure 3.6 Typical elevation of 4 storey BRBFs: a) VBF-case-1, b) VBF-case-2, c) VBF-case-3, and d) VBF-case-4

Table 3.2 Properties of members in BRBFs and cross-sectional area of BRBs

Structure	Columns		Beams		Brace area (mm^2)	Brace property
	Exterior	Interior	Exterior	Interior		
2-Storey frames:						
Case-1-2-3-4 (DBF)						
1	HE 220A	HE 220A	IPE 300	IPE 300	2065	TUBO-D 139.7*4
2	HE 180A	HE 160A	IPE 2400	IPE 2400	1705	TUBO-D 168.3*4
Case-1-2-3-4 (CHBF-SXBF-VBF)						
1	HE 220A	HE 220A	IPE 300	IPE 300	1705	TUBO-D 139.7*4
2	HE 180A	HE 160A	IPE 2400	IPE 2400	1252	TUBO-D 114.3*3.6
4-Storey frames:						
Case-1-2-3-4 (DBF)						
1	HE 260A	HE 260A	IPE 300	IPE 300	4056	TUBO-D 244.5*5.4
2	HE 260A	HE 260A	IPE 300	IPE 300	2675	TUBO-D 193.7*4.5
3	HE 180A	HE 200A	IPE 300	IPE 300	2675	TUBO-D 193.7*4.5
4	HE 180A	HE 200A	IPE 2200	IPE 2200	1705	TUBO-D 139.7*4
Case-1-2-3-4 (CHBF-SXBF-VBF)						
1	HE 260A	HE 260A	IPE 300	IPE 300	2675	TUBO-D 193.7*4.5
2	HE 260A	HE 260A	IPE 300	IPE 300	2065	TUBO-D 168.3*4
3	HE 180A	HE 200A	IPE 300	IPE 300	1865	TUBO-D 152.4*4
4	HE 180A	HE 200A	IPE 2200	IPE 2200	1252	TUBO-D 114.3*3.6
6-Storey frames:						
Case-1-2-3-4 (DBF)						
1	HE 240A	HE 300A	IPE 270	IPE 270	3363	TUBO-D 219.1*5
2	HE 240A	HE 300A	IPE 270	IPE 270	3363	TUBO-D 219.1*5
3	HE 200A	HE 240A	IPE 270	IPE 270	2675	TUBO-D 193.7*4.5
4	HE 200A	HE 240A	IPE 270	IPE 270	2065	TUBO-D 168.3*4
5	HE 180A	HE 180A	IPE 270	IPE 270	1865	TUBO-D 152.4*4
6	HE 180A	HE 180A	IPE 240	IPE 240	1252	TUBO-D 114.3*3.6

Table 3.2 Continued

Case-1-2-3-4 (CHBF-SXBF-VBF)						
1	HE 240A	HE 300A	IPE 270	IPE 270	2065	TUBO-D 168.3*4
2	HE 240A	HE 300A	IPE 270	IPE 270	2065	TUBO-D 168.3*4
3	HE 200A	HE 240A	IPE 270	IPE 270	1948	TUBO-D 159*4
4	HE 200A	HE 240A	IPE 270	IPE 270	1546	TUBO-D 127*4
5	HE 180A	HE 180A	IPE 270	IPE 270	1252	TUBO-D 114.3*3.6
6	HE 180A	HE 180A	IPE 240	IPE 240	861.6	TUBO-D 88.9*3.2
8-Storey frames:						
Case-1-2-3-4 (DBF)						
1	HE 260A	HE 320A	IPE 270	IPE 270	3363	TUBO-D 219.1*5
2	HE 260A	HE 320A	IPE 270	IPE 270	2675	TUBO-D 193.7*4.5
3	HE 220A	HE 260A	IPE 270	IPE 270	2675	TUBO-D 193.7*4.5
4	HE 220A	HE 260A	IPE 270	IPE 270	2675	TUBO-D 193.7*4.5
5	HE 200A	HE 220A	IPE 270	IPE 270	1705	TUBO-D 139.7*4
6	HE 200A	HE 220A	IPE 270	IPE 270	1621	TUBO-D 133*4
7	HE 180A	HE 180A	IPE 270	IPE 270	1546	TUBO-D 127*4
8	HE 180A	HE 180A	IPE 240	IPE 240	1181	TUBO-D 108*3.6
Case-1-2-3-4 (CHBF-SXBF-VBF)						
1	HE 260A	HE 320A	IPE 270	IPE 270	2065	TUBO-D 168.3*4
2	HE 260A	HE 320A	IPE 270	IPE 270	1865	TUBO-D 152.4*4
3	HE 220A	HE 260A	IPE 270	IPE 270	1865	TUBO-D 152.4*4
4	HE 220A	HE 260A	IPE 270	IPE 270	1865	TUBO-D 152.4*4
5	HE 200A	HE 220A	IPE 270	IPE 270	1252	TUBO-D 114.3*3.6
6	HE 200A	HE 220A	IPE 270	IPE 270	1181	TUBO-D 108*3.6
7	HE 180A	HE 180A	IPE 270	IPE 270	1108	TUBO-D 101.6*3.6
8	HE 180A	HE 180A	IPE 240	IPE 240	861.6	TUBO-D 88.9*3.2

For beam, column, and BRB elements, nominal strength and modulus of elasticity were obtained to be 345 MPa (according to ASTM A992) and 200 GPa, respectively. Each model in the current study was named according to the brace configuration (DBF, CHBF, SXBF, and VBF), distribution (Case-1, Case-2, Case-3, and Case-4) of the brace frames, and storey height (2, 4, 6, and 8). For example, (DBF-case-2-4) referred to the model that utilized diagonal brace frame configuration (DBF) and second distribution (case-2) for four storey frame.

The performance of ordinary frames and various braced frames were investigated by nonlinear analyses to assess the performance of each case study using the finite element program of SAP2000 non-linear version 14 (CSI Analysis Reference Manual, 2009). Table 3.3 illustrates the periods of ordinary frames and frames with

BRBs. A critical damping ratio of 5% was considered for all analyses of frames. For the aims of comparison, it is matter that the ordinary and braced frames had the same damping ratio.

Table 3.3 Natural periods of the model structures

Definition	Periods			Definition	Periods		
	T_1 (s)	T_2 (s)	T_3 (s)		T_1 (s)	T_2 (s)	T_3 (s)
OF-2	1.465	0.564	0.104	OF-6	3.381	1.218	0.695
DBF-case-1-2	0.389	0.174	0.104	DBF-case-1-6	0.934	0.349	0.215
DBF-case-2-2	0.390	0.170	0.101	DBF-case-2-6	0.872	0.355	0.237
DBF-case-3-2	0.395	0.177	0.095	DBF-case-3-6	0.890	0.333	0.247
DBF-case-4-2	0.431	0.167	0.099	DBF-case-4-6	1.208	0.399	0.230
CHBF-case-1-2	0.387	0.153	0.093	CHBF-case-1-6	0.950	0.360	0.211
CHBF-case-2-2	0.393	0.151	0.093	CHBF-case-2-6	0.942	0.371	0.225
CHBF-case-3-2	0.395	0.155	0.095	CHBF-case-3-6	0.951	0.351	0.233
CHBF-case-4-2	0.389	0.157	0.095	CHBF-case-4-6	1.058	0.386	0.217
SXBF-case-1-2	0.384	0.160	0.089	SXBF-case-1-6	0.947	0.356	0.208
SXBF-case-2-2	0.397	0.156	0.106	SXBF-case-2-6	0.923	0.371	0.224
SXBF-case-3-2	0.402	0.163	0.114	SXBF-case-3-6	0.938	0.346	0.233
SXBF-case-4-2	0.389	0.168	0.089	SXBF-case-4-6	1.090	0.384	0.215
VBF-case-1-2	0.406	0.154	0.106	VBF-case-1-6	1.024	0.364	0.212
VBF-case-2-2	0.407	0.163	0.105	VBF-case-2-6	0.976	0.371	0.236
VBF-case-3-2	0.399	0.169	0.112	VBF-case-3-6	0.973	0.351	0.243
VBF-case-4-2	0.432	0.159	0.112	VBF-case-4-6	1.223	0.392	0.218
OF-4	2.186	0.797	0.481	OF-8	4.401	1.566	0.921
DBF-case-1-4	0.579	0.229	0.161	DBF-case-1-8	1.348	0.480	0.278
DBF-case-2-4	0.542	0.232	0.194	DBF-case-2-8	1.271	0.453	0.309
DBF-case-3-4	0.570	0.224	0.192	DBF-case-3-8	1.259	0.470	0.304
DBF-case-4-4	0.706	0.251	0.154	DBF-case-4-8	1.756	0.556	0.305
CHBF-case-1-4	0.590	0.229	0.143	CHBF-case-1-8	1.333	0.488	0.274
CHBF-case-2-4	0.594	0.226	0.169	CHBF-case-2-8	1.277	0.470	0.299
CHBF-case-3-4	0.610	0.221	0.169	CHBF-case-3-8	1.287	0.478	0.293
CHBF-case-4-4	0.619	0.239	0.146	CHBF-case-4-8	1.549	0.526	0.285
SXBF-case1-4	0.587	0.227	0.147	SXBF-case-1-8	1.337	0.480	0.269
SXBF-case-2-4	0.587	0.222	0.176	SXBF-case-2-8	1.251	0.463	0.303
SXBF-case-3-4	0.607	0.221	0.176	SXBF-case-3-8	1.287	0.473	0.301
SXBF-case-4-4	0.630	0.240	0.152	SXBF-case-4-8	1.600	0.519	0.312
VBF-case-1-4	0.641	0.229	0.143	VBF-case-1-8	1.439	0.493	0.275
VBF-case-2-4	0.605	0.227	0.193	VBF-case-2-8	1.320	0.470	0.309
VBF-case-3-4	0.618	0.227	0.188	VBF-case-3-8	1.341	0.485	0.306
VBF-case-4-4	0.723	0.241	0.152	VBF-case-4-8	1.762	0.537	0.287

3.2 Modeling approach

A two dimensional model of each structure was created using SAP2000 to study capacity curve, inter-storey drift index, global damage index, and plastic hinge formation. In this case, the non-linear static analysis considering displacement control was applied for each frame.

In the modeling approach of frame's plastic behavior, there are two approaches, first one is with non-linear links (NL-Link) and the other one is with plastic hinge. The natures of the plastic hinges are axial force (N), moment (M), and axial force and moment (N-M) interaction. In this study, columns and beams were modeled as frame elements with plastic hinges at the end and start of each element. The BRBs were modeled as non-linear plastic hinge and the elastoplastic force deformation property was used as shown in Figure 3.7 and 3% of strain hardening was considered. During analysis, P-delta effect was ignored.

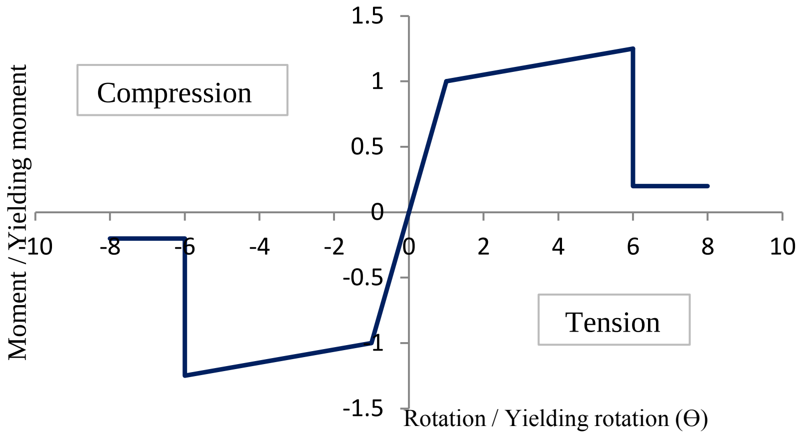


Figure 3.7 Plastic hinges bending hinge (M/M_y vs θ/θ_y) (Cesar and Barros, 2009)

Later designing and detailing the steel frame buildings, a nonlinear pushover analysis was carried out for assessing the seismic reaction of the structures. The pushover analysis includes of the displacement control, the applied displacement control is acceleration in the x direction appearing for the forces that would be experienced by the structures when exposed to IBC2006 spectrum. Under incrementally increasing pushing a frame, a number of elements may yield consequently, at each event, the structures experience a stiffness change.

3.3 Nonlinear behavior of structural elements

The nonlinear behavior of a building structure depends on the nonlinear responses of the elements that are used in the lateral force resisting system. Therefore, before applying any nonlinear analysis method on a building structure, a very important aspect in the analysis is the definition of the material model that is used to simulate the ductility of the structural members of the complete structure.

In FEMA-356 (FEMA 356, 2000), the simplified load deformation relationship used to model the columns and the beams elements, and the deformation criteria (for action controlled by deformation) for the several materials used, while, exhibiting nonlinear behavior is shown in Figure 3.8. After the member yields (when applied load/yield load proportion (Q/Qy) is equal to 1), the following strain hardening supplies the strain hardening in the load-deformation relation as the member deforms toward the expected strength. The horizontal axis of this diagram may either express curvature or strain.

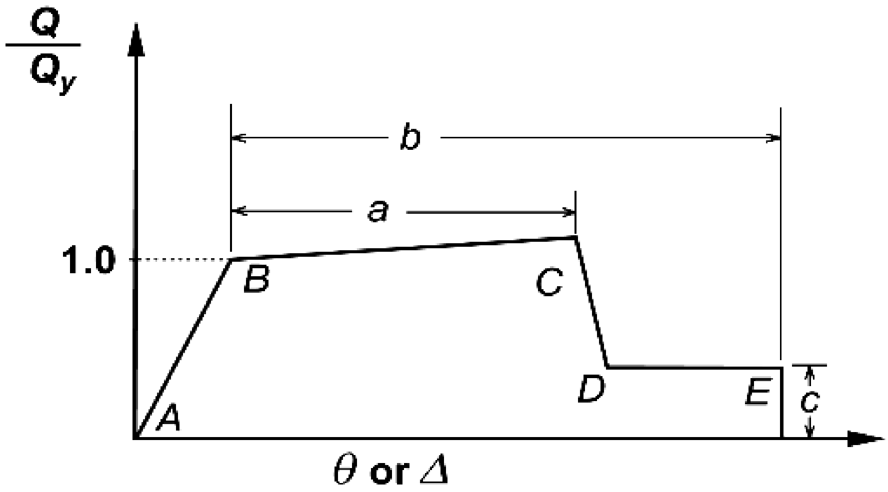


Figure 3.8 Generalized load deformation relation while exhibiting nonlinear behavior of a structural member (FEMA 356, 2000)

Point A corresponds to unloaded condition and point B represents yielding of the element, so the first line AB is shown a linear response. The ordinate at C corresponds to nominal strength and abscissa at C corresponds to the deformation at which significant strength degradation begins, so the inclination of the second line BC is usually low (0 to 10% of the value of the inclination of the elastic regime AB) and it represents some hardening. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable, The third line CD represents the degradation of the resistant capacity while the line DE corresponds to the plastification of the structural element, The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained.

The criterion of acceptable deformation is defined in ATC-40 and FEMA-356 codes depending on the plastic hinge rotations by considering various performance levels In Figure 3.9, the acceptance criteria on a force versus deformation diagram are given. In this diagram, the points marked as IO, LS and CP represent immediate occupancy, life safety and collapse prevention, respectively.

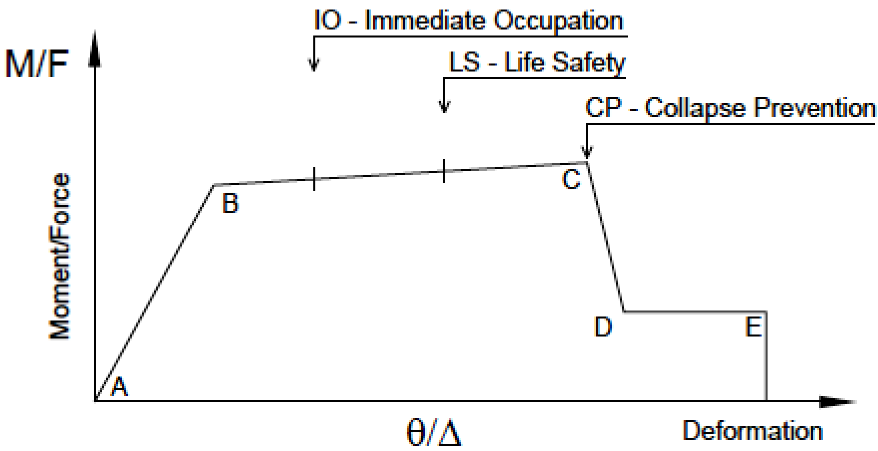


Figure 3.9 Acceptance criteria on a force versus deformation diagram (FEMA 356, 2000)

Hinges can be assigned at any number of locations (potential yielding points) along the span of the frame element as well as element ends. Uncoupled moment (M2 and M3), torsion (T), axial force (P) and shear (V2 and V3) force-displacement relations can be defined. As the column axial load changes under lateral loading, there is also a coupled P-M2-M3 (PMM) hinge which yields based on the interaction of axial force and bending moments at the hinge location. Also, more than one type of hinge can be assigned at the same location of a frame element.

In SAP2000, there are three kinds of hinge properties. They are user-defined hinge properties, default hinge properties, and generated hinge properties. When these hinge properties (default and user-defined) are distributed to a frame member the program automatically produces a new generated hinge property for every single hinge. Only default hinge properties and user-defined hinge properties can be assigned to frame members.

When default hinge properties are used, the program merges its built-in default criteria with the defined segment properties for each member to generate the last hinge properties. Default hinge properties could not be modified, and they section dependent. The built-in default hinge properties for steel FEMA-356 (FEMA356, 2000) criteria.

The generated hinge properties are used in the analysis. They could be observed but they could not be limited. User-defined hinge properties can be based on default properties, or they can be completely user-defined. When user-defined properties are not based on default properties, then the properties can be viewed and modified.

3.4 Pushover analysis

In the recent years, the pushover analysis is a simplified methodology to determine the non-linear behavior of the building structures and analysis response to seismic actions through a non-linear static analysis. This analysis, evaluates the performance of the structures through control of its displacements (at local and global levels), the relation of roof displacement and base shear shows the capacity curve. This curve expresses the response of the building structure under increasing base shear forces.

Pushover analysis, as a practical way of estimating the deformation and damage pattern of a structure, is getting increasingly more attention. The method includes of two sections first, a target displacement for the frame is constituted. The target displacement is an evaluation of the highest displacement of the structure when exposed to the design earthquake excitation. Then a pushover analysis is carried out on the building until the highest displacement of the structure equals to the target displacement (Tso and Moghadam, 1998).

The pushover analysis can be performed considering the control over the force or displacement control. Force control option was useful when the magnitude of the load was known clearly, and the structure was expected to support that load. The displacement control was useful when the magnitude of the load was unknown and displacements were searched. Also in the force controlled, pushover procedure have some numerical problems that affect the accuracy of results occur since target displacement maybe related with a very little positive or even a negative lateral stiffness because of the development of mechanism and P-delta (Sermin, 2005).

3.5 Target displacement

The target displacement for a building with rigid diaphragms at each floor level shall be estimated using an established procedure that account for likely nonlinear response of buildings, the “Coefficient Method” described in FEMA 356 (FEMA 356, 2000) is ready for use and tries to affected several of these shortcomings with the incorporation of practical “coefficients” in the computation of the displacement related with the demand curves. Using this way the demand curve is matured from,

$$\delta = C_o C_1 C_2 C_3 S_a \frac{Te^2}{4\pi^2} \mathrm{g} \tag{3.1}$$

Where:

T_e: Effective fundamental period of the building in the direction under consideration, sec

C_o: Modification factor to relate spectral displacement and likely building roof displacement.

C_1: Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear 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-delta, and

S_a: Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration, g.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 General

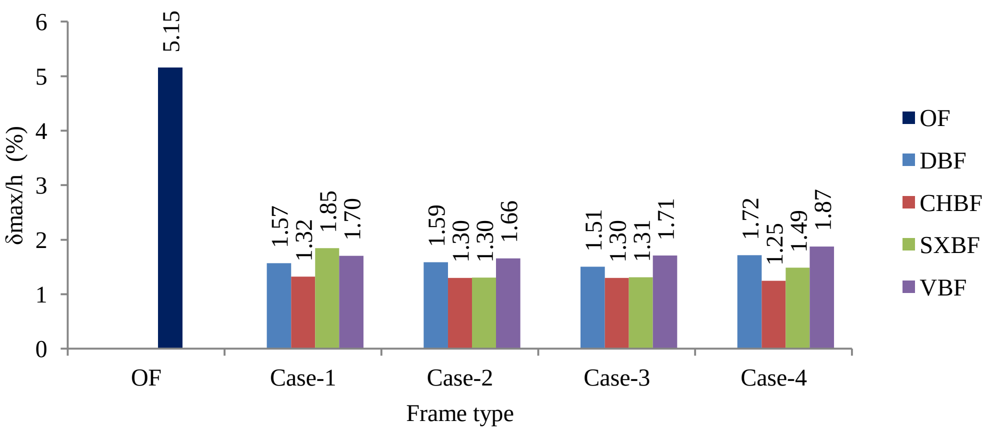
In this section, the results for ordinary frames (OFs), and buckling restrained braced frames (BRBFs) obtained from nonlinear static analysis were given and discussed the performances. In the present study, a total of 68 different cases were taken into account and structural performance of unbraced and braced frame systems having different configuration of buckling restrained braces and their distributions under the effect of lateral loading were evaluated. Performance characteristics in terms of capacity curves, inter-story drift index, global damage index, deflected shapes, base shear and plastic hinge formation were given.

4.2 Global deformations

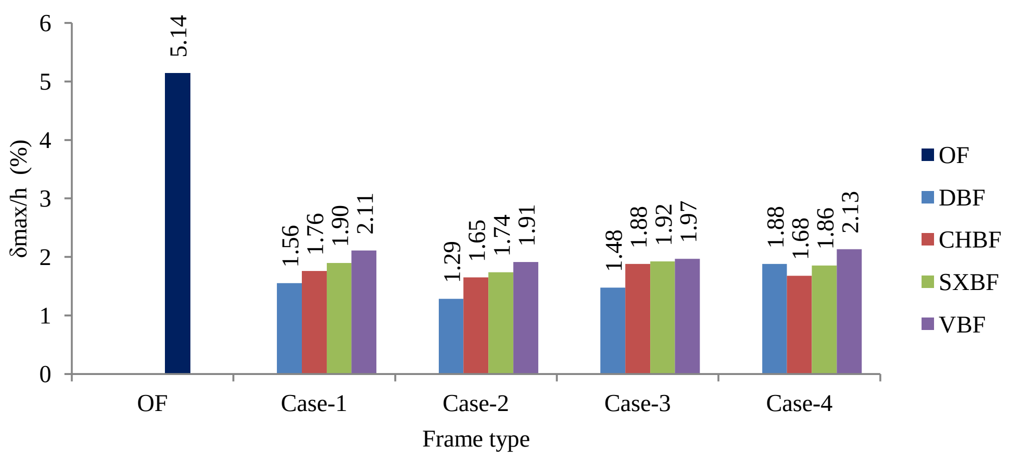
The maximum inter-storey drift (δ_{max}) divided by the storey height (h) is defined as maximum inter-storey index. The maximum inter-storey index computed for ordinary frames (OFs) and frames with BRBs is presented in Figure 4.1 for different storey height, brace configuration, and distribution. This index is a good mark of the damages experienced by the structural members. Drift limit requirements depend on the building code and analysis procedure that has been utilized. In order to provide context of the frame performance, for moment resisting frames (MRFs), FEMA356 recommendations specify inter-storey drift index thresholds of 0.7%, 2.5% and 5% for immediate occupancy (IO), life safety (LS) and collapse prevention (CP), respectively. However, for BRBFs, these values are limited to 0.5%, 1.5% and 2%.

In this study, buckling restrained braces were used as solutions to reduce the large drift of 2, 4, 6, and 8 storey MRFs, which exceeds the limit of (CP) performance static provided by FEMA356, i.e. 5%.

In general, computed drifts are below 2% for most of the buckling restrained braced frames as shown in Figure 4.1. Moreover, it was observed that some of the frame cases were performing better compared to the others and that was dependent on storey variations, brace configurations, and distributions. For example, the chevron configuration in the 2, 6, and 8 storey frames were marginally superior to other configurations considered, since they provide a corresponding inter-storey drift between 1.0 to 1.4% that correspond to LS performance level according to FEMA 356. Whereas, other brace configurations provided an inter-storey drift between 1.14 to 1.87% that correspond to the CP performance level.



a)



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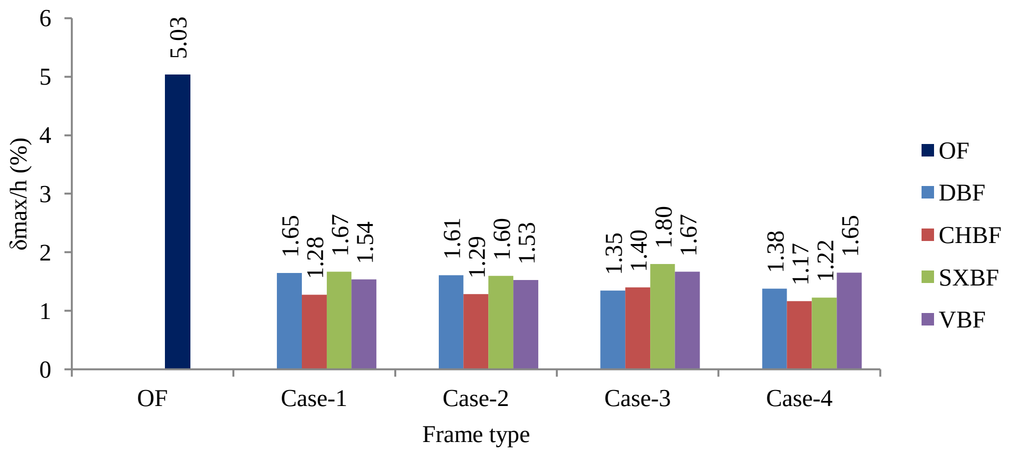

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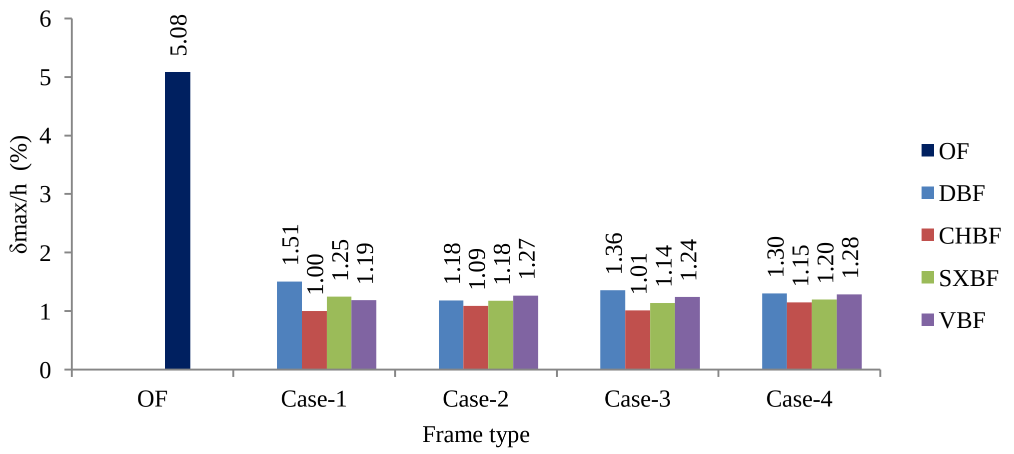
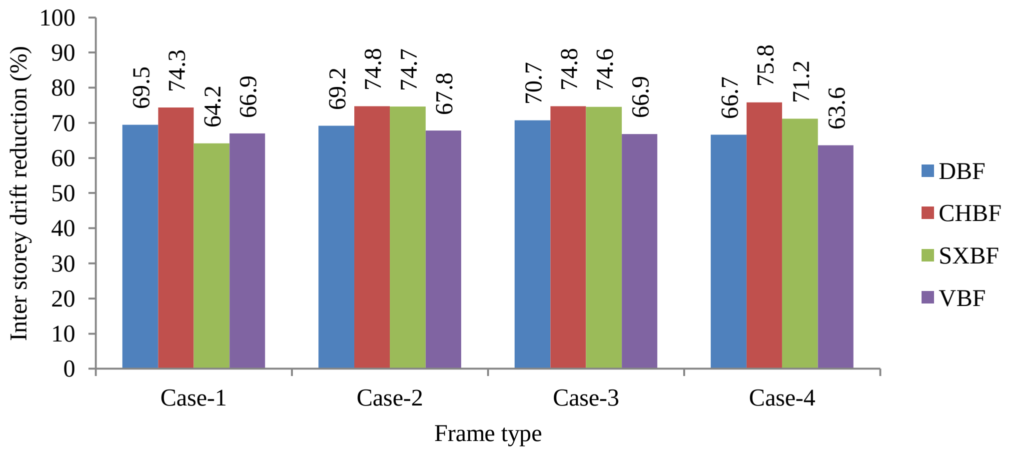


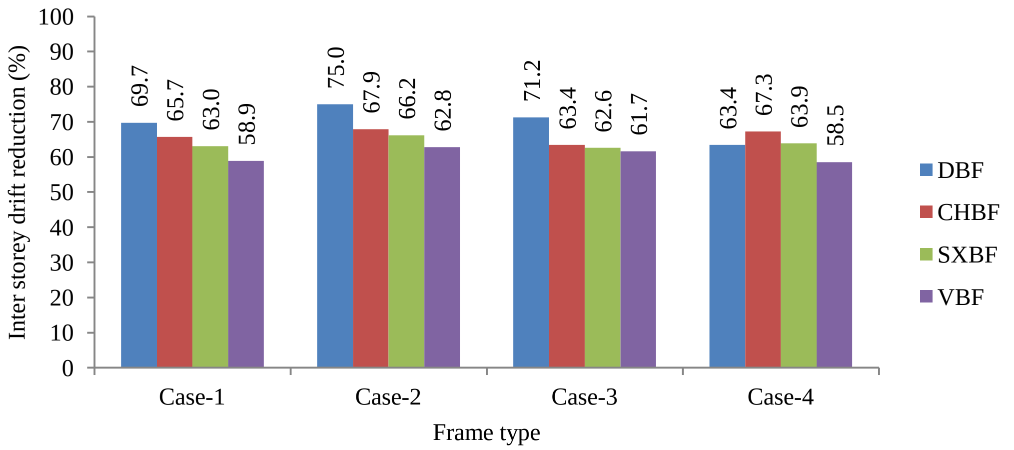
d)

Figure 4.1 Maximum inter storey index for ordinary frames and BRBFs: a) 2-storey, b) 4-storey, c) 6-storey, and d) 8-storey

Figure 4.2 shows the reduction in inter-storey index for different storey height, configuration, and distribution. For 2, 6 and 8 storey buildings, the results of the displayed response confirmed the beneficial effects of configurations with chevron bracings, due to the fact that the corresponding reduction of inter-storey drift with respect to the original MRFs varied between 74.5 to 79.1% for different storey height and distribution. However, for 4 storey building, it was found that inter-storey drift in configurations with diagonal bracing was proved to be more effective because they provided a reduction of 69.8%, whereas, in the case of chevron bracing, this reduction is only 66.1%.



a)



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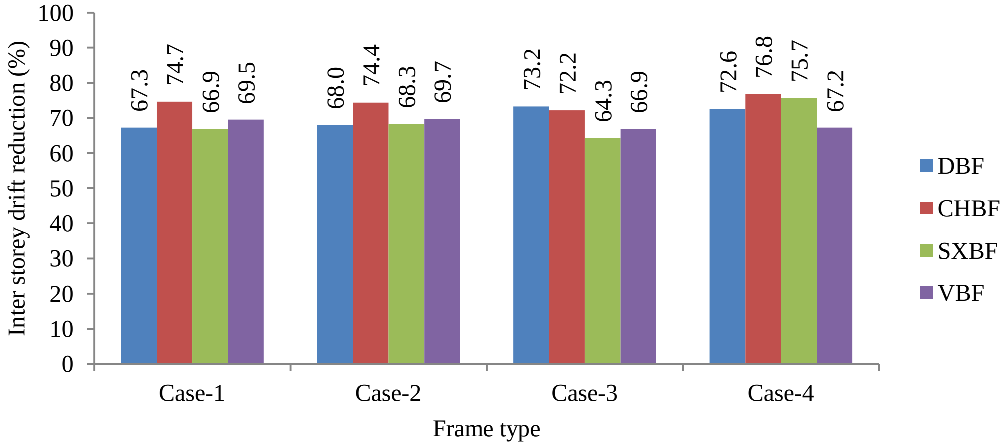
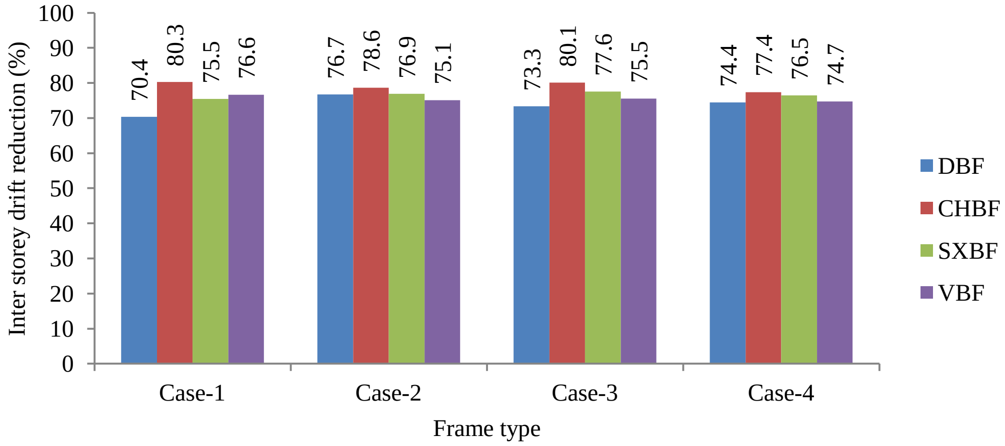


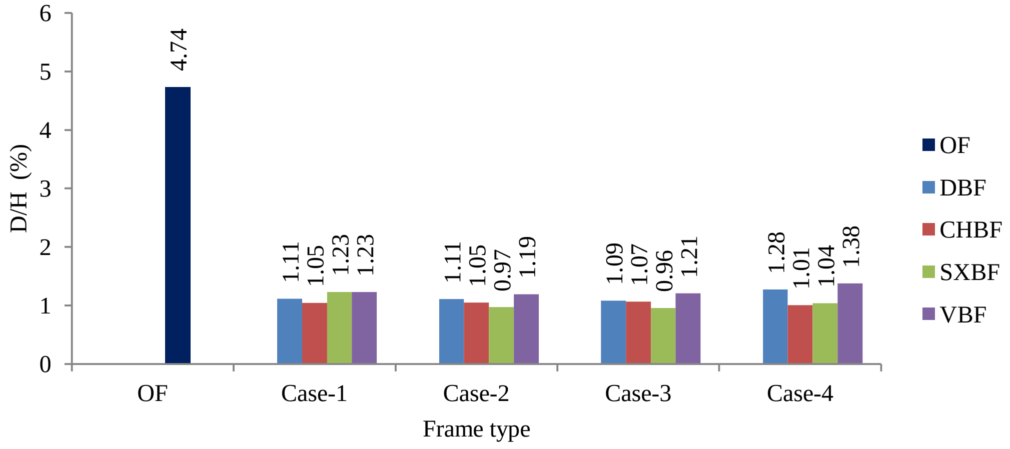
c)



d)

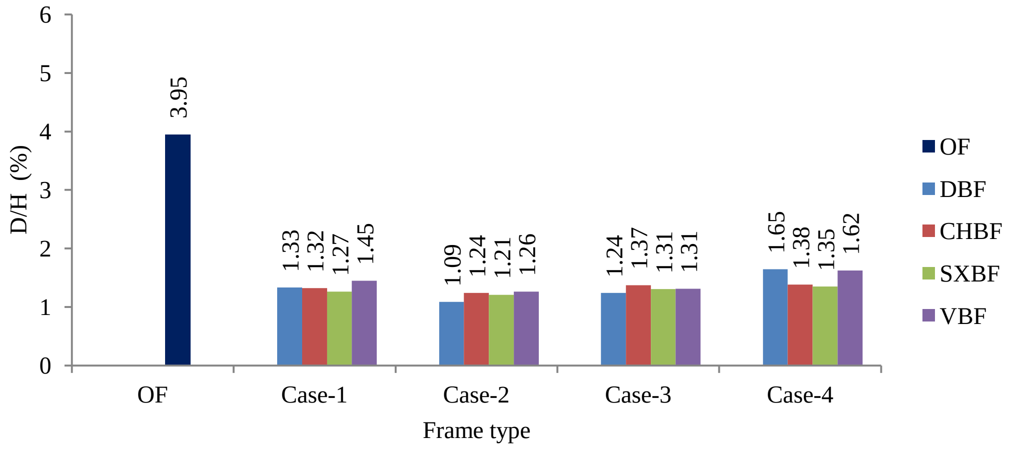
Figure 4.2 Reduction in inter-storey index with the use of BRBs: a) 2-storey, b) 4-storey, c) 6-storey, and d) 8-storey

The ratio of the roof displacement (D) over the total height of the building (H) is defined as the global damage index. Global damage index (or roof drift index, D/H) assessed for OFs and BRBFs was found for all cases. Figure 4.3 presented global damage index for OFs with different storey height, brace configuration, and distribution. Comparison of global damage index of the frame showed that the roof drift for V-brace was greater value than that other configurations in 2, 4, 6, and 8 stories. Also, all brace frames revealed better performance in comparison to the ordinary frames (OFs).



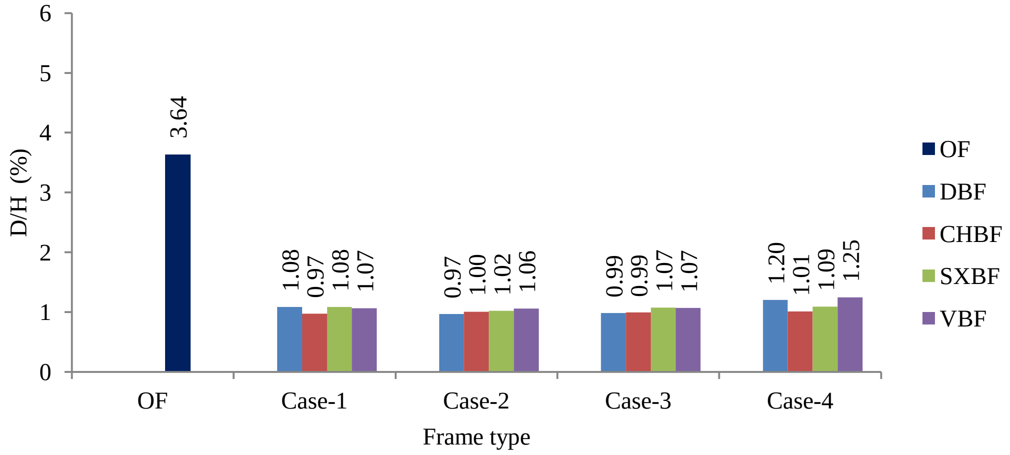
6
5
4
3
2
1
0
D/H (%)
4.74
1.11
1.05
1.23
1.23
1.11
1.05
0.97
1.19
1.09
1.07
0.96
1.21
1.28
1.01
1.04
1.38
OF
Case-1
Case-2
Case-3
Case-4
Frame type
OF
DBF
CHBF
SXBF
VBF

a)



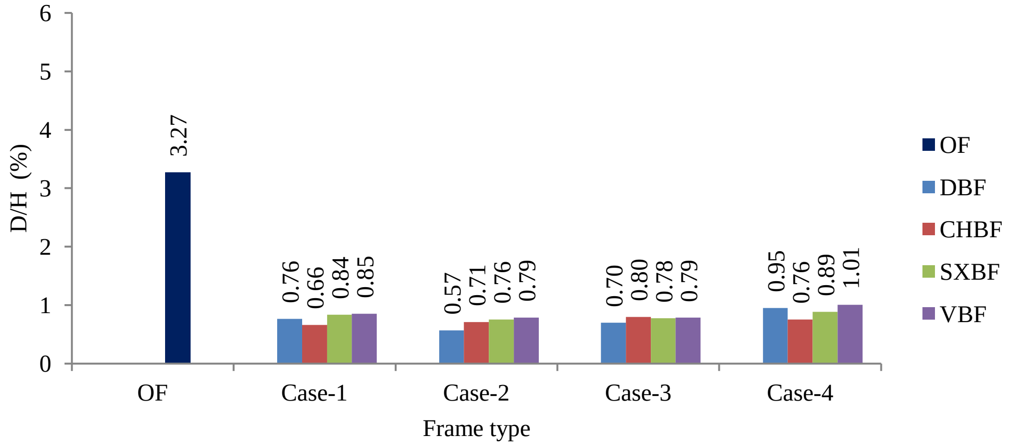
6
5
4
3
2
1
0
D/H (%)
3.95
1.33
1.32
1.27
1.45
1.09
1.24
1.21
1.26
1.24
1.37
1.31
1.31
1.65
1.38
1.35
1.62
OF
Case-1
Case-2
Case-3
Case-4
Frame type
OF
DBF
CHBF
SXBF
VBF

b)



6
5
4
3
2
1
0
D/H (%)
3.64
1.08
0.97
1.08
1.07
0.97
1.00
1.02
1.06
0.99
0.99
1.07
1.07
1.20
1.01
1.09
1.25
OF
Case-1
Case-2
Case-3
Case-4
Frame type
OF
DBF
CHBF
SXBF
VBF

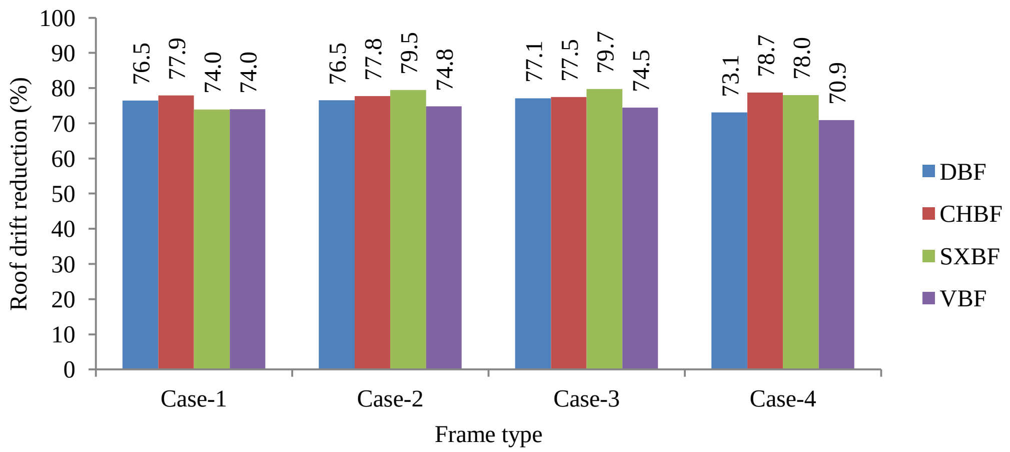
c)



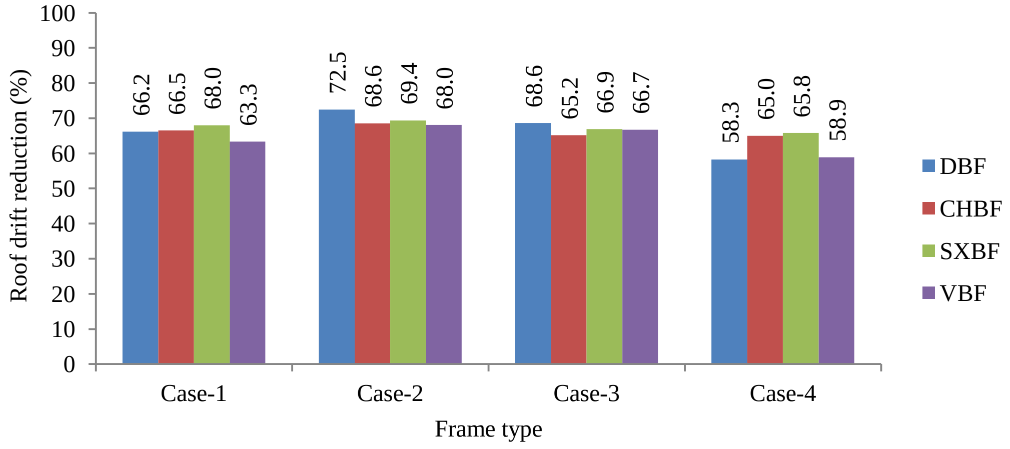
d)

Figure 4.3 Maximum global damage index for ordinary frames and BRBFs: a) 2-storey, b) 4-storey, c) 6-storey, and d) 8-storey

The results of global drift reduction are summarized in Figure 4.4. It was observed that for the roof lateral drift the presence of chevron braces was beneficial in terms of the maximum transitional displacements for 2, 6, and 8 storey frames, in which it provided a reduction between 72.7 to 78.0%. However, for 4 storey frames, split-X brace configurations (SXBF) were the most effective one since they provided a reduction of 67.5% on average for different distribution.



a)



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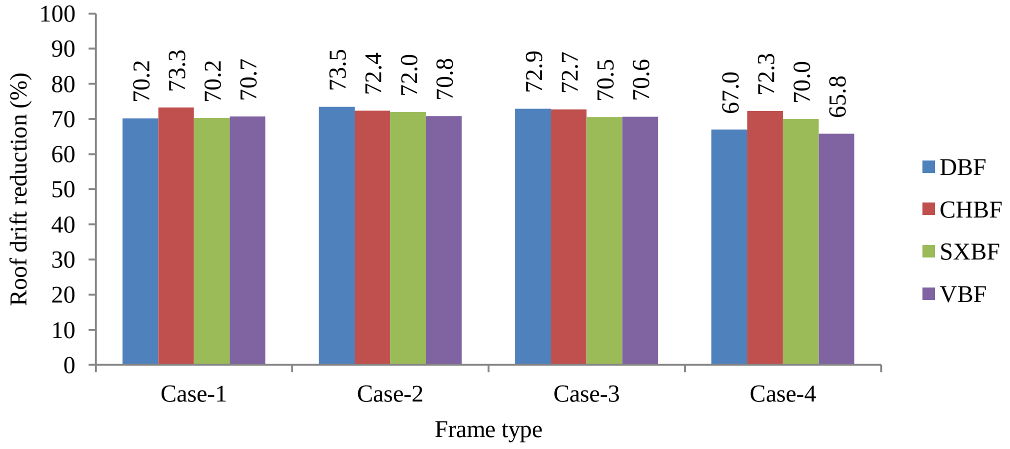

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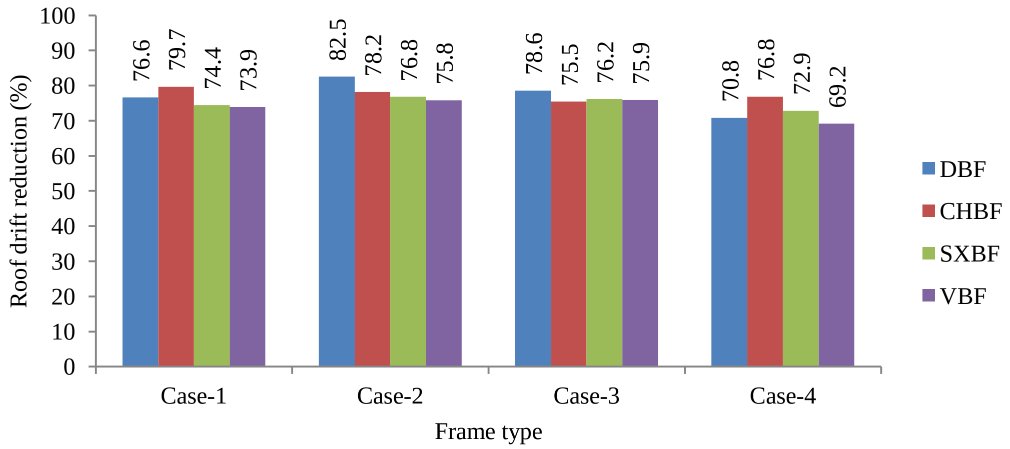


d)

Figure 4.4 Global damage index reduction for OFs and BRBFs: a) 2-storey, b) 4-storey, c) 6-storey, and d) 8-storey

4.3 Local deformations

For assessing structural performance, it is important to emphasis on location and number of plastic hinges as presented in Table 4.1. As expected, the worst performance corresponds to the ordinary frames was observed, and the analysis results implied that the use of BRB turned to be quite effective for retrofitting MRFs and that was due to improved nonlinear behavior of BRB that supports both compression and tension loading similarly and absorbing more energy in the inelastic range. As it can be seen from Table 4.1, the most of the inelastic deformations were concentrated in the BRBs and the other structural elements remained in an acceptable level of deformation.

Table 4.1 Number of hinges in the structural members

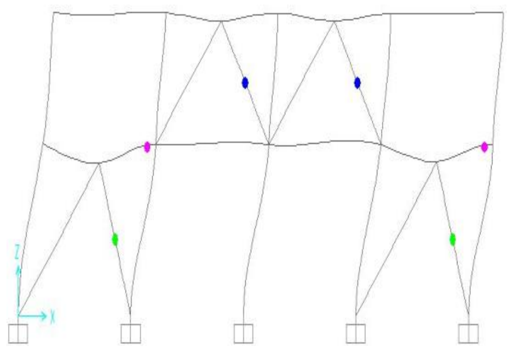
Definition	Columns					Beams					Braces				
	IO	LS	CP	C	D	IO	LS	CP	C	D	IO	LS	CP	C	D
OF-2	7	2	-	6	-	2	1	-	-	-	-	-	-	-	-
DBF-case-1-2	-	-	-	-	-	-	-	-	-	-	2	-	1	-	-
DBF-case-2-2	-	-	-	-	-	-	-	-	-	-	2	-	1	-	-
DBF-case-3-2	-	-	-	-	-	-	-	-	-	-	2	1	-	-	-
DBF-case-4-2	-	-	-	-	-	-	-	-	-	-	2	-	1	-	-
CHBF-case-1-2	-	-	-	-	-	2	-	-	-	-	2	-	2	-	-
CHBF-case-2-2	-	-	-	-	-	-	-	-	-	-	2	-	2	-	-
CHBF-case-3-2	-	-	-	-	-	1	-	-	-	-	2	-	2	-	-
CHBF-case-4-2	-	-	-	-	-	-	-	-	-	-	2	1	1	-	-
SXBF-case-1-2	1	-	-	-	-	-	-	-	-	-	2	-	2	-	-
SXBF-case-2-2	-	-	-	-	-	1	-	-	-	-	2	-	2	-	-
SXBF-case-3-2	-	-	-	-	-	1	-	-	-	-	2	-	2	-	-
SXBF-case-4-2	-	-	-	-	-	-	-	-	-	-	2	2	-	-	-
VBF-case-1-2	-	-	-	-	-	-	-	-	-	-	5	1	0	-	-
VBF-case-2-2	-	-	-	-	-	-	-	-	-	-	6	-	-	-	-
VBF-case-3-2	-	-	-	-	-	-	-	-	-	-	5	1	-	-	-
VBF-case-4-2	3	-	-	-	-	-	-	-	-	-	4	2	-	-	-
OF-4	1	5	-	7	-	8	3	5	-	-	-	-	-	-	-
DBF-case-1-4	-	-	-	1	-	4	-	-	-	-	7	-	-	-	-
DBF-case-2-4	-	-	-	-	-	-	-	-	-	-	7	-	-	-	-
DBF-case-3-4	-	-	-	-	-	4	-	-	-	-	7	-	-	-	-
DBF-case-4-4	1	-	-	1	-	5	-	-	-	-	2	1	-	-	-
CHBF-case-1-4	-	-	1	-	-	4	1	1	-	-	6	2	-	-	-
CHBF-case-2-4	-	-	-	-	-	4	2	-	-	-	6	2	-	-	-
CHBF-case-3-4	-	-	-	-	-	5	1	1	-	-	7	2	-	-	-
CHBF-case-4-4	-	1	-	-	-	4	2	-	-	-	7	2	-	-	-

Table 4.1 Continued

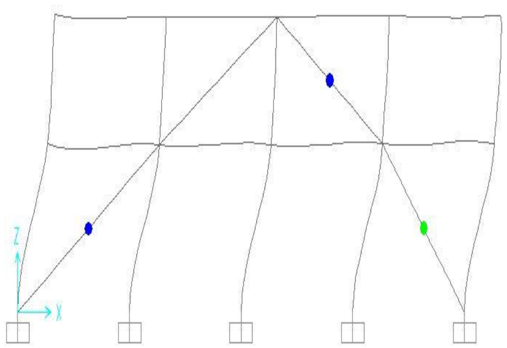
SXBF-case-1-4	3	-	-	1	-	4	-	-	-	-	10	-	-	-	-
SXBF-case-2-4	-	-	-	-	-	2	1	1	-	-	8	2	-	-	-
SXBF-case-3-4	-	-	-	-	-	2	1	1	-	-	9	2	-	-	-
SXBF-case-4-4	2	-	-	1	-	3	-	-	-	-	10	-	-	-	-
VBF-case-1-4	3	1	-	1	-	3	-	-	-	-	8	-	-	-	-
VBF-case-2-4	1	2	-	-	-	2	-	-	-	-	12	-	-	-	-
VBF-case-3-4	4	-	-	-	-	5	-	-	-	-	12	-	-	-	-
VBF-case-4-4	2	1	-	1	1	5	-	-	-	-	3	-	-	-	-
OF-6	5	-	-	4	-	16	4	8	-	-	-	-	-	-	-
DBF-case-1-6	1	-	-	-	-	6	-	-	-	-	9	-	-	-	-
DBF-case-2-6	-	-	-	-	-	3	-	-	-	-	9	-	-	-	-
DBF-case-3-6	-	1	1	-	-	1	-	-	-	-	9	-	-	-	-
DBF-case-4-6	-	-	-	-	1	6	-	-	-	-	6	-	-	-	-
CHBF-case-1-6	-	-	-	-	-	6	-	-	-	-	12	-	-	-	-
CHBF-case-2-6	-	-	-	-	-	6	-	-	-	-	9	3	-	-	-
CHBF-case-3-6	1	-	-	-	-	3	2	-	-	-	8	4	-	-	-
CHBF-case-4-6	-	-	1	-	1	3	-	-	-	-	11	1	-	-	-
SXBF-case-1-6	-	-	-	1	-	5	-	-	-	-	14	1	-	-	-
SXBF-case-2-6	-	-	-	-	-	2	2	-	-	-	11	2	2	-	-
SXBF-case-3-6	1	-	-	-	1	1	-	2	-	-	9	2	2	-	-
SXBF-case-4-6	-	-	1	-	1	3	-	-	-	-	8	-	-	-	-
VBF-case-1-6	1	-	-	1	-	5	-	-	-	-	11	-	-	-	-
VBF-case-2-6	-	-	-	1	-	1	-	-	-	-	14	-	-	-	-
VBF-case-3-6	1	-	-	1	-	3	-	-	-	-	15	-	-	-	-
VBF-case-4-6	1	-	1	1	1	6	-	-	-	-	5	-	-	-	-
OF-8	2	-	-	3	-	24	5	11	-	-	-	-	-	-	-
DBF-case-1-8	-	-	-	-	-	-	-	-	-	-	7	1	-	-	-
DBF-case-2-8	-	-	-	-	-	-	-	-	-	-	6	1	-	-	-
DBF-case-3-8	-	-	-	-	-	-	-	-	-	-	8	-	1	-	-
DBF-case-4-8	-	-	-	-	1	-	-	-	-	-	8	-	-	-	-
CHBF-case-1-8	-	-	-	-	-	1	-	-	-	-	16	-	-	-	-
CHBF-case-2-8	-	-	-	-	-	-	-	-	-	-	14	2	-	-	-
CHBF-case-3-8	-	-	-	-	-	3	-	-	-	-	14	2	-	-	-
CHBF-case-4-8	1	-	1	-	-	1	-	-	-	-	13	-	-	-	-
SXBF-case-1-8	1	-	-	-	1	1	-	-	-	-	12	-	-	-	-
SXBF-case-2-8	-	-	-	-	-	4	-	-	-	-	15	-	-	-	-
SXBF-case-3-8	1	-	-	-	1	3	-	-	-	-	14	-	-	-	-
SXBF-case-4-8	-	1	-	-	1	-	-	-	-	-	8	-	-	-	-
VBF-case-1-8	-	1	-	-	1	2	-	-	-	-	13	-	-	-	-
VBF-case-2-8	1	-	-	1	-	-	-	-	-	-	14	-	-	-	-
VBF-case-3-8	-	1	-	-	-	-	-	-	-	-	14	2	-	-	-
VBF-case-4-8	1	-	-	1	1	6	-	-	-	-	6	-	-	-	-

Figure 4.5 illustrates the distribution of the inelastic deformation and corresponding performance level according to FEMA356 for 2-storey frames with different brace configuration and distribution. Appendix B illustrates all BRBFs for different storey height, brace configurations, and distribution.

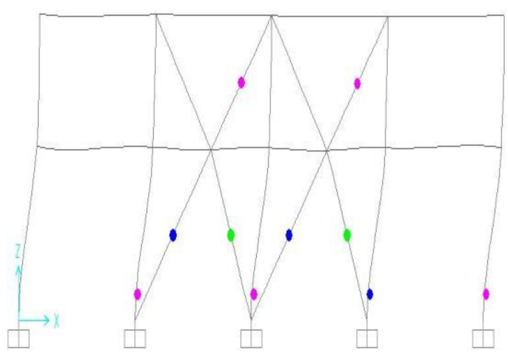
For 2 storey frames, it was observed that chevron and diagonal brace configurations were the most appropriate and safe case because most of the plastification were concentrated in the brace elements and few beams undergone plastic deformations with IO limit state. However, in the case of SXBF and VBF, some of the columns entered inelastic range of deformation which might be reported as unsafe mechanism.



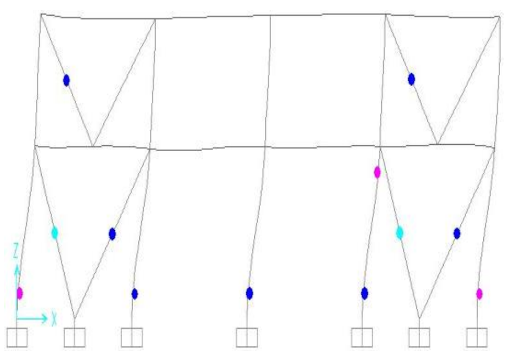
CHBF-case-2-2



DBF-case-2-2



SXBF-case-1-2



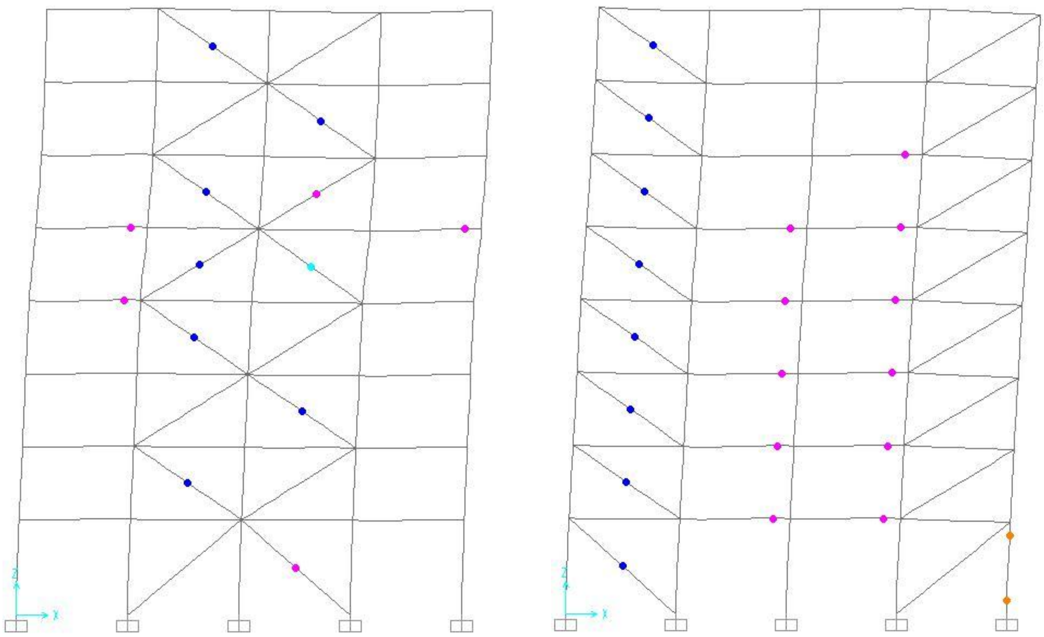
VBF-case4-2

Figure 4.5 Plastic hinge formation for 2-storey BRBFs with different configuration

The plastification of 4-storey CHBF, DBF, and SXBF with 2nd and 3rd distribution could be stated as a suitable configuration because the columns were in the elastic range. However, for the rest of the other frame cases, it was observed that many column members were reached inelastic range of deformations. For 6 and 8 storey frames with chevron and diagonal braces that utilized 1st, 2nd and 3rd distribution are more proper compared to the other configurations because the performance of the structural elements lays between IO-LS limit states, whilst a limit state between CP-D could be observed for the other configurations.

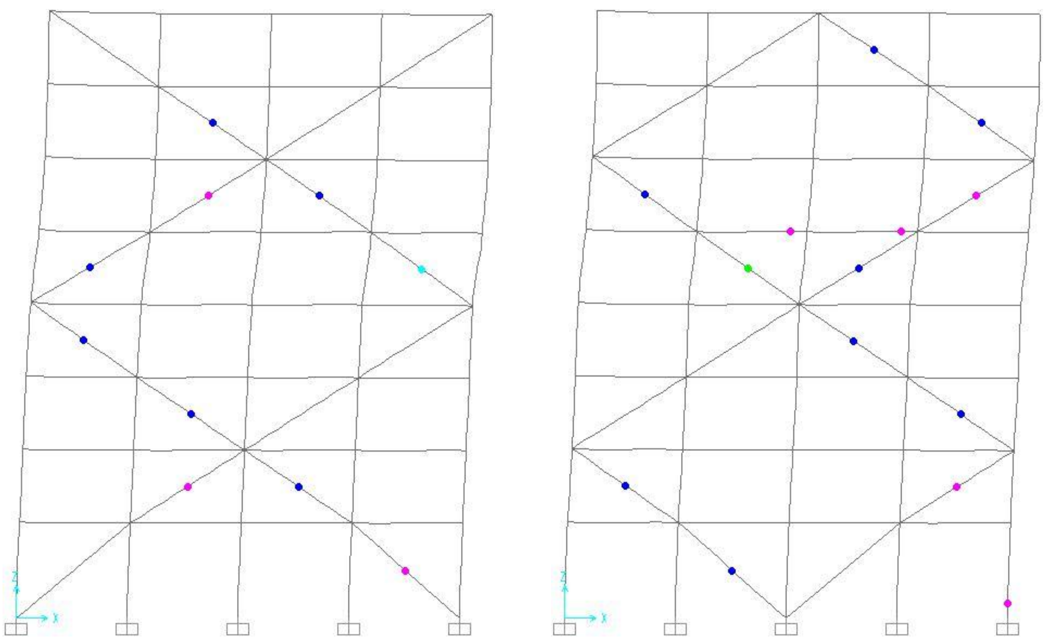
Regardless of the building height, it was more effective to resistance the lateral displacement when the braces were inserted near the mid-span than in the end span. For example, comparing the structural performance of 1st distribution (where the brace set at the mid-span) and 4th distribution (where the brace set at the end span) in the 8- storey frame that utilized diagonal brace configuration represented in Figure 4.6 was worth noted that the column members remained in the elastic range of deformation when the 1st distribution was used. However, it was evident that an exterior column reached D level of plastification when the 4th distribution used.

Furthermore, it was observed that 2nd and 3rd distribution (that distributed the braces over the bays and height of the structure) were in general demonstrates similar behavior as 1st distribution and they exhibited better performance compared to 4th distribution as represented in Figure 4.7. Results for chevron, split-X, and V-brace configurations showed the same tendency. These facts indicated that the location of the brace might be adjusted properly to make the stiffness distributing equalization of the whole structure.



DBF-case-1-8 DBF-case-4-8

Figure 4.6 Plastic hinge formation for 8-storey BRBFs with DBF configuration

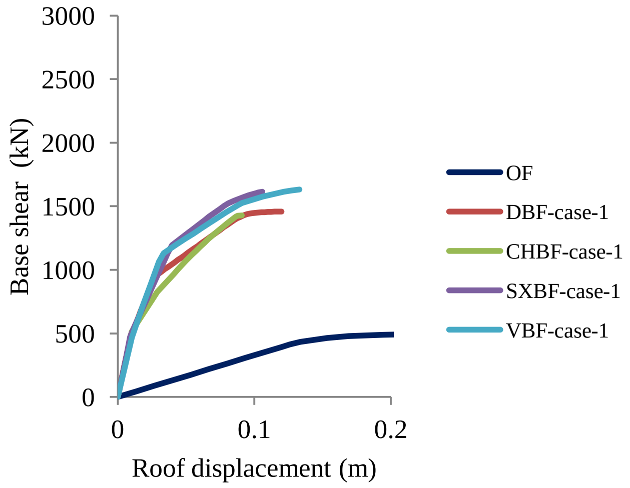


DBF-case-2-8 DBF-case-3-8

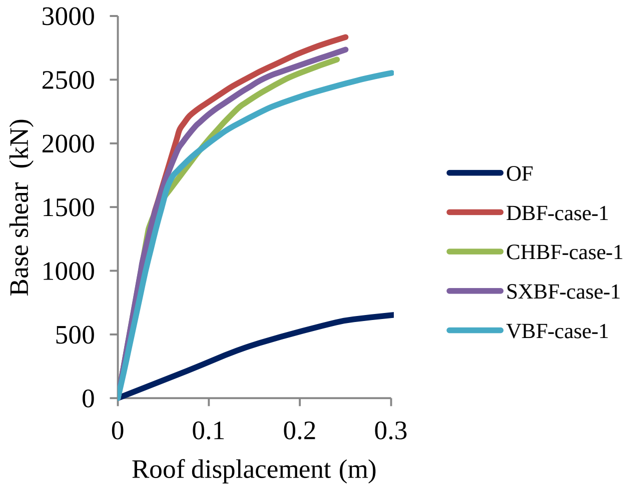
Figure 4.7 Plastic hinge formation for 8-storey BRBFs with DBF configuration

4.4 Capacity curves

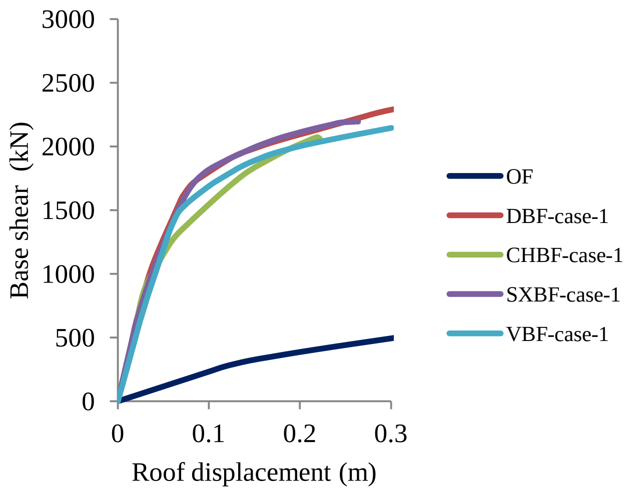
From the nonlinear static analysis in order to evaluate the stiffness and strength of OFs and frames with buckling restrained braces capacity curves were obtained as shown in Figures 4.8. Appendix C illustrates the capacity curves for BRBFs and OFs.



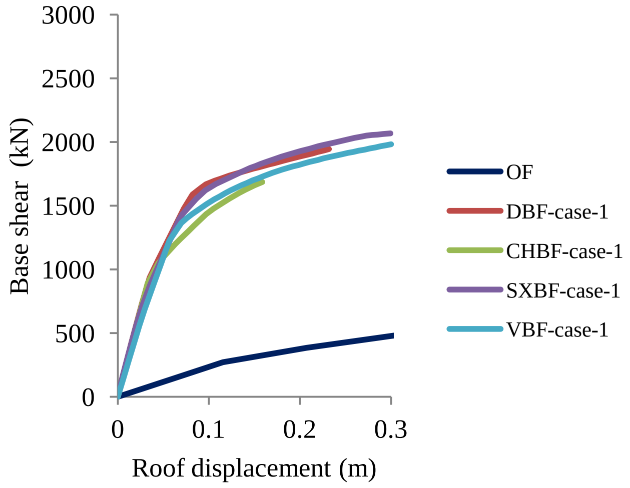
a)



b)



c)



d)

Figure 4.8 Capacity curves for OFs and BBRFs with different brace configurations: a) 2-storey, b) 4-storey, c) 6-storey, and d) 8-storey

In these curves, it is possible to identify several important parameters in the seismic response of the analyzed structures, namely the yielding displacement and the stiffness variation with the increase of the load. This representation still supplies information about the nonlinear behavior of the structure.

Depending up on the design parameters, the performance of the unbraced structures was considerably lower than that of frames retrofitted with BRBs. For the same top displacement of each of the four unbraced frame corresponds a lower base shear in comparison to the braced structural configuration. In general, these curves showed similar features for unbraced frames. They are initially linear but start to deviate from linearity as the columns and beams undergo inelastic actions. When the buildings were pushed well into the inelastic range, the curves became linear again but with a smaller slope. These curves could be approximated by a bi-linear relationship. However, for braced frames, it was observed that, in general, the pushover curves were tri-linear. The first change of the slope was induced by brace yielding and the second change was due to the yielding of the frame members. Therefore, the length of the second slope was the delay between the yielding of the brace and frame members.

Due to the fact that the same amount of steel for the braces was used, the pushover responses in the elastic range for all configurations were almost the same, as it can be seen from Figure 4.8 or appendix C, However, after the yielding of the braces, they started to deviate and experience a different stiffness drop. Also, it was noted that the capacity curves showed no decrease in the load carrying capacity of the buildings, suggesting good structural behavior.

As represented in Table 4.2, compared to the bare frame for the 2 storey buildings, the retrofitted cases with V-braces were the most effective because on average they increased the capacity by factor of 3.2. However, for 4 and 6 storey frames, the retrofitted cases with diagonal braces had provided the highest strength factor of 3.85 and 3.42, respectively. Whilst, the capacity of 8 storey frame was greatly enhanced through the addition of split-X brace, because on average they provided the highest strength factor of 3.02. This means that the number of stories (dynamic characteristics of structures) determines which system performs better. Also, it was observed that in general the 4th distribution (although stiffer than unbraced frames) did not proved high performance as high resistance as the other distribution.

Table 4.2 Initial stiffness and lateral strength capacity of ordinary frames and retrofitted cases

Cases	Initial stiffness (kN/mm)	Ultimate base shear ratio (kN)	Stiffness ratio	Strength ratio
OF-2	3235	514	1.0	1.0
DBF-case-1-2	48990	1459	15.1	2.8
DBF-case-2-2	45978	1459	14.2	2.8
DBF-case-3-2	48095	1459	14.9	2.8
DBF-case-4-2	38605	1450	11.9	2.8
CHBF-case-1-2	48889	1428	15.1	2.8
CHBF-case-2-2	45747	1405	14.1	2.7
CHBF-case-3-2	45988	1405	14.2	2.7
CHBF-case-4-2	47918	1318	14.8	2.6
SXBF-case-1-2	50482	1613	15.6	3.1
SXBF-case-2-2	41566	1410	12.8	2.7
SXBF-case-3-2	38955	1437	12.0	2.8
SXBF-case-4-2	44873	1496	13.9	2.9
VBF-case-1-2	44236	1631	13.7	3.2
VBF-case-2-2	45408	1648	14.0	3.2
VBF-case-3-2	47945	1644	14.8	3.2
VBF-case-4-2	39753	1628	12.3	3.2
OF-4	2816	724	1.0	1.0
DBF-case-1-4	37745	2833	13.4	3.9
DBF-case-2-4	43497	2920	15.4	4.0
DBF-case-3-4	39263	2878	13.9	4.0
DBF-case-4-4	24769	2529	8.8	3.5
CHBF-case-1-4	38281	2658	13.6	3.7
CHBF-case-2-4	38560	2697	13.7	3.7
CHBF-case-3-4	37183	2685	13.2	3.7
CHBF-case-4-4	32179	2615	11.4	3.6
SXBF-case-1-4	38559	2735	13.7	3.8
SXBF-case-2-4	40123	2702	14.2	3.7
SXBF-case-3-4	37529	2688	13.3	3.7
SXBF-case-4-4	29942	2637	10.6	3.6
VBF-case-1-4	31132	2551	11.1	3.5
VBF-case-2-4	35720	2909	12.7	4.0
VBF-case-3-4	35080	2878	12.5	4.0
VBF-case-4-4	22831	2225	8.1	3.1
OF-6	2317	647	1.0	1.0
DBF-case-1-6	30789	2423	13.3	3.7
DBF-case-2-6	35879	2459	15.5	3.8
DBF-case-3-6	33049	2384	14.3	3.7
DBF-case-4-6	18577	1618	8.0	2.5

Table 4.2 Continued

CHBF-case-1-6	30265	2060	13.1	3.2
CHBF-case-2-6	31561	2106	13.6	3.3
CHBF-case-3-6	30055	2055	13.0	3.2
CHBF-case-4-6	24498	1852	10.6	2.9
SXBF-case-1-6	30749	2195	13.3	3.4
SXBF-case-2-6	33225	2125	14.3	3.3
SXBF-case-3-6	30774	2071	13.3	3.2
SXBF-case-4-6	23068	1903	10.0	2.9
VBF-case-1-6	25348	2144	10.9	3.3
VBF-case-2-6	28162	2224	12.2	3.4
VBF-case-3-6	27697	2276	12.0	3.5
VBF-case-4-6	17820	1670	7.7	2.6
OF-8	2326	652	1.0	1.0
DBF-case-1-8	29203	1944	12.6	3.0
DBF-case-2-8	34536	1944	14.9	3.0
DBF-case-3-8	30602	1792	13.2	2.7
DBF-case-4-8	16693	1433	7.2	2.2
CHBF-case-1-8	28108	1686	12.1	2.6
CHBF-case-2-8	29193	1813	12.6	2.8
CHBF-case-3-8	28667	2010	12.3	3.1
CHBF-case-4-8	21641	1570	9.3	2.4
SXBF-case-1-8	29464	2066	12.7	3.2
SXBF-case-2-8	30859	2098	13.3	3.2
SXBF-case-3-8	28518	2005	12.3	3.1
SXBF-case-4-8	20451	1684	8.8	2.6
VBF-case-1-8	22997	1982	9.9	3.0
VBF-case-2-8	26248	2109	11.3	3.2
VBF-case-3-8	25727	2038	11.1	3.1
VBF-case-4-8	15878	1410	6.8	2.2

4.5 Deflected shapes

Figure 4.9 exhibited the deflected shapes for ordinary frames (OFs), and Figures 4.10-4.13 illustrate the deflected shapes for BRBs. They were computed from the nonlinear analysis at the instance corresponding to target displacement. It was point out that BRBs considerably decreased the value of maximum roof displacement and corresponding storey displacement compared to bare frames. Also, it was noticed that the maximum storey displacement curves were close to linear line and the maximum inter-storey drifts were relatively uniform along the height of the structure and there was not concentration of large deformation in one storey or without abrupt changes in the drift pattern with respect to the level of deformation as desired.

For various frame heights, the effect of using different configuration and distribution on the deflected shape response seemed to be negligible and that was due to the enhanced nonlinear behavior of BRBs which exhibits considerable yielding capacity in both compression and tension similarly.

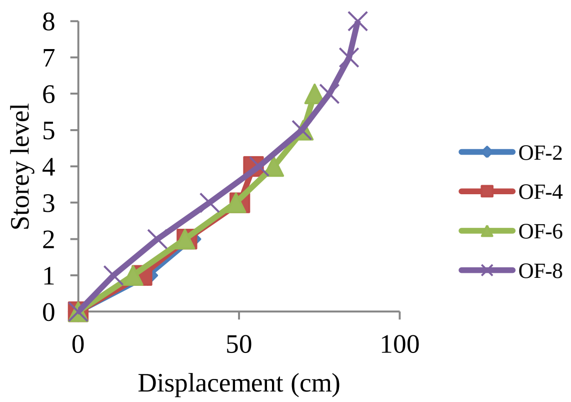


Figure 4.9 Deflected shape of different OFs

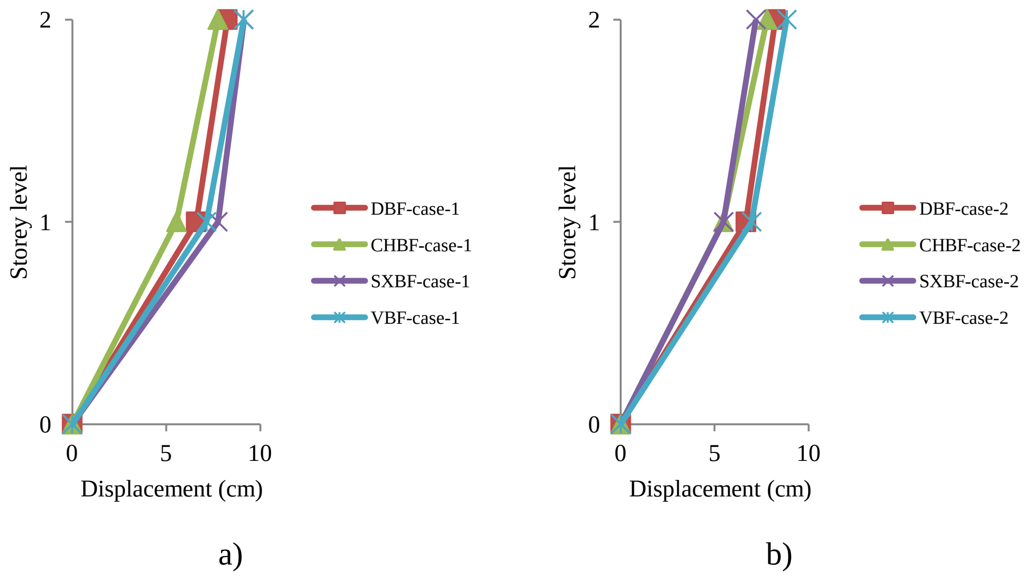
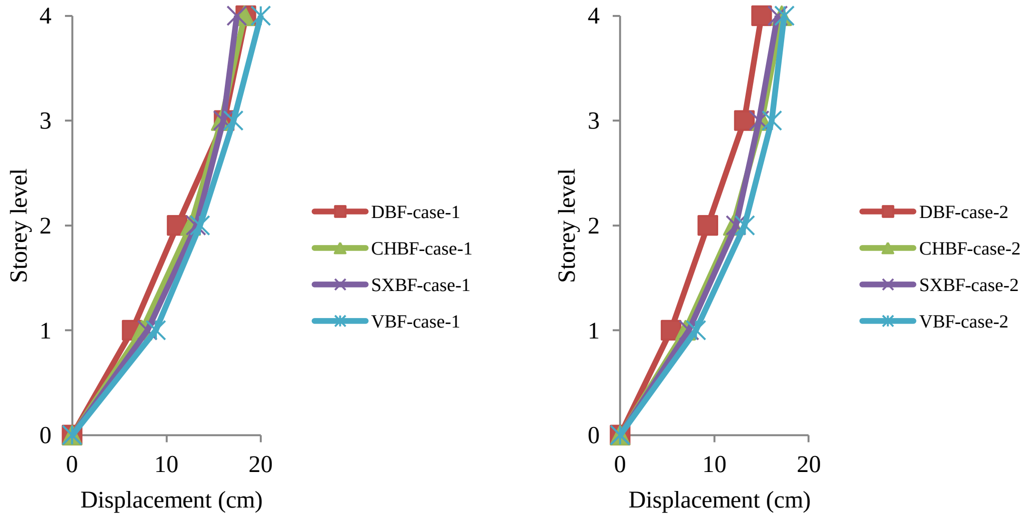
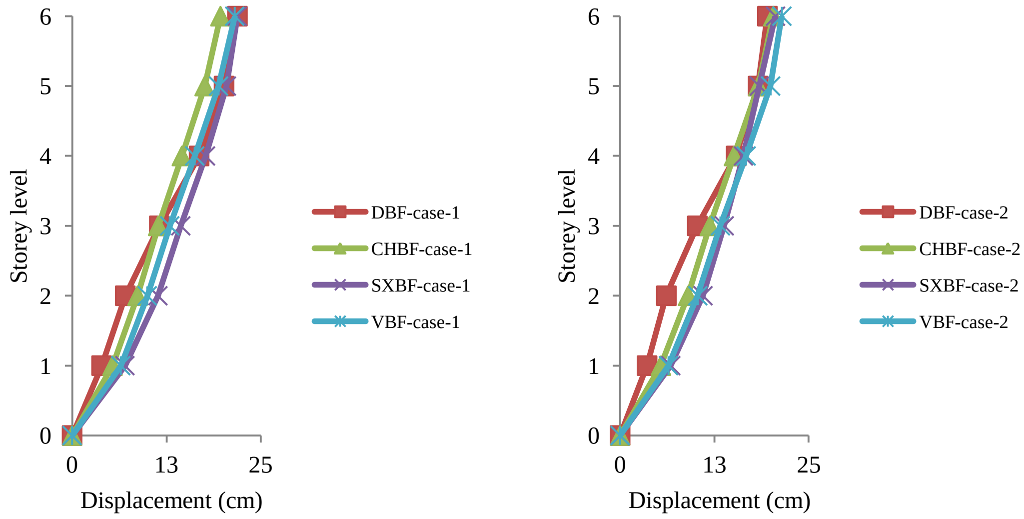


Figure 4.10 Deflected shape of 2-storey BRBFs with: a) case-1 and b) case-2



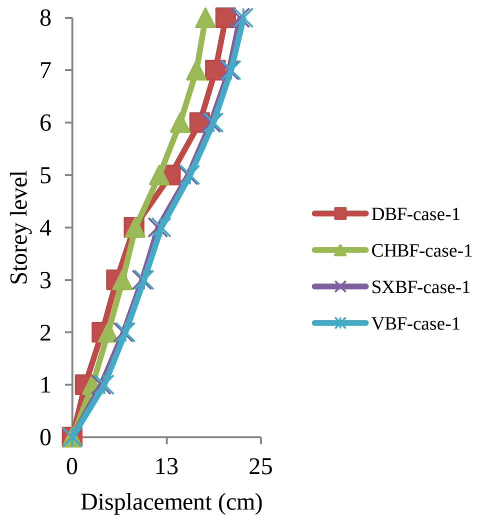
a) b)

Figure 4.11 Deflected shape of 4-storey BRBFs with: a) case-1 and b) case-2

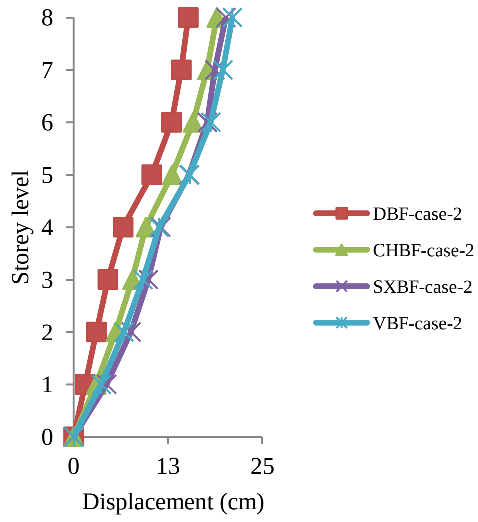


a) b)

Figure 4.12 Deflected shape of 6-storey BRBFs with: a) case-1 and b) case-2



a)



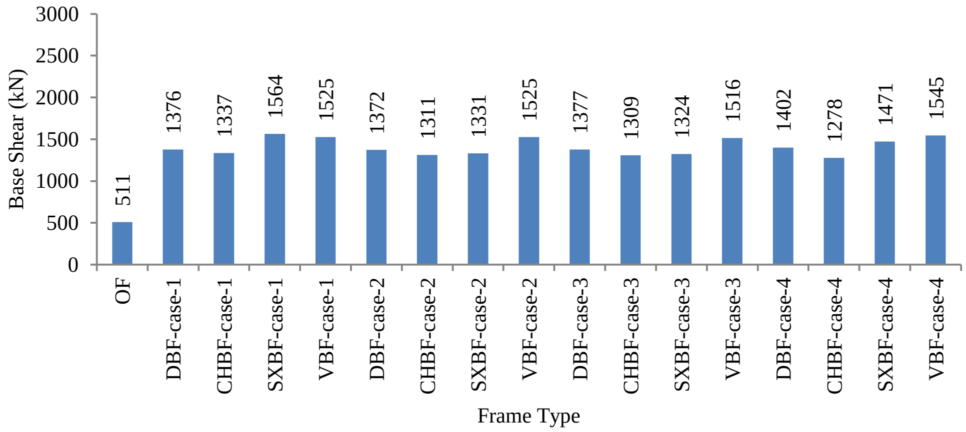
b)

Figure 4.13 Deflected shape of 8-storey BRBFs with: a) case-1 and b) case-2

4.6 Variation of base shear

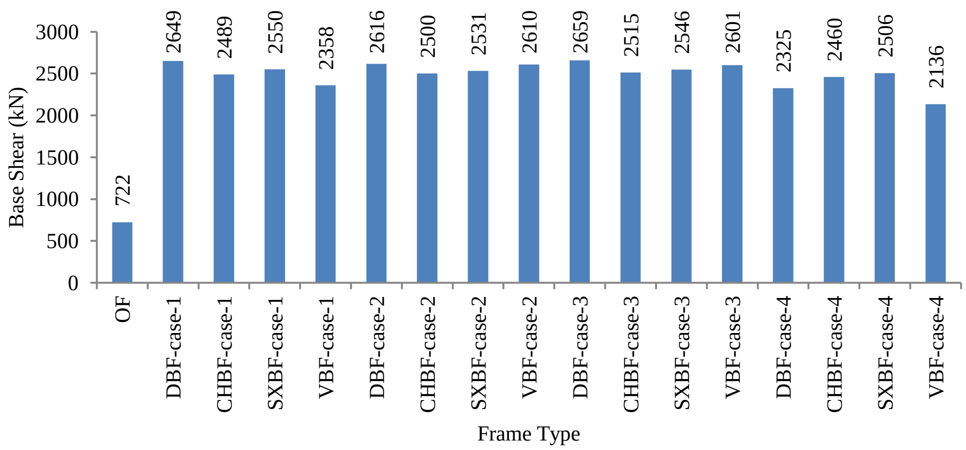
Variation of the base shears for various frame cases corresponding to OFs and BRBFs are shown in Figure 4.14. The base shear of the bare frame is the lowest, but corresponding bending moment in the frame member may be very large. The presence of bracings may increase the total base shear but the frame members were not influenced so much by this increment because the shear carried and hence the bending moments in the frames are reduced due to the shear resisted by the bracings.

For 6 and 8 storey frames with diagonal bracing with 4th distribution (case-4) had the lowest base shear compared to the other braced frame cases. However, frame cases of DBF-case-4-2 and DBF-case-4-4 had experienced larger base shear compared to i.e. CHBF-case-2-2, VBF-case-4-4, etc. This means that the variation of the base shear with respect to the type of the braces and distribution is irregular and that was due to the fact the total base shear depends on the combined lateral stiffness of bracings and the moment resisting frames.



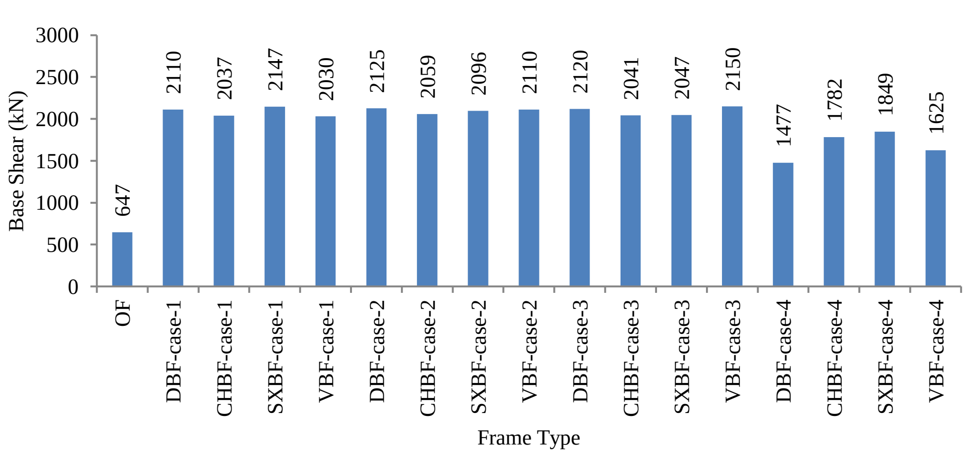
Base Shear (kN)
3000
2500
2000
1500
1000
500
0
511
1376
1337
1564
1525
1372
1311
1331
1525
1377
1309
1324
1516
1402
1278
1471
1545
OF
DBF-case-1
CHBF-case-1
SXBF-case-1
VBF-case-1
DBF-case-2
CHBF-case-2
SXBF-case-2
VBF-case-2
DBF-case-3
CHBF-case-3
SXBF-case-3
VBF-case-3
DBF-case-4
CHBF-case-4
SXBF-case-4
VBF-case-4
Frame Type

a)



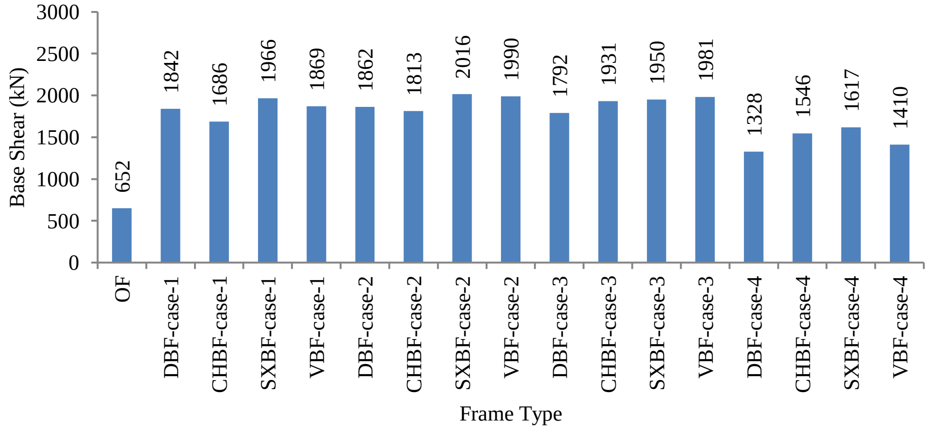
Base Shear (kN)
3000
2500
2000
1500
1000
500
0
722
2649
2489
2550
2358
2616
2500
2531
2610
2659
2515
2546
2601
2325
2460
2506
2136
OF
DBF-case-1
CHBF-case-1
SXBF-case-1
VBF-case-1
DBF-case-2
CHBF-case-2
SXBF-case-2
VBF-case-2
DBF-case-3
CHBF-case-3
SXBF-case-3
VBF-case-3
DBF-case-4
CHBF-case-4
SXBF-case-4
VBF-case-4
Frame Type

b)



Base Shear (kN)
3000
2500
2000
1500
1000
500
0
647
2110
2037
2147
2030
2125
2059
2096
2110
2120
2041
2047
2150
1477
1782
1849
1625
OF
DBF-case-1
CHBF-case-1
SXBF-case-1
VBF-case-1
DBF-case-2
CHBF-case-2
SXBF-case-2
VBF-case-2
DBF-case-3
CHBF-case-3
SXBF-case-3
VBF-case-3
DBF-case-4
CHBF-case-4
SXBF-case-4
VBF-case-4
Frame Type

c)



d)

Figure 4.14 Base shear for OFs and BRBFs: a) 2-storey, b) 4-storey, c) 6-storey, and d) 8-storey

CHAPTER 5

CONCLUSSION

The present analytical study aimed at retrofitting moment resisting frames (having insufficient lateral stiffness) with BRBs. For this purpose, four different concentrically braced frame configurations together with different brace distributions were analyzed to evaluate and compare their performances. The following conclusions were drawn from the results of this study:

1. Adding buckling restrained braces considerably reduced maximum inter-storey drift index, especially the use of chevron and diagonal configurations. It was observed that the maximum and minimum reduction of inter-storey index with respect to the bare frames was on average equal to 79.1% and 69.8%, respectively. Also, the use of BRBs significantly reduced the global damage index of the frames.

2. Regardless of the building height, the use of diagonal and chevron bracings provided an appropriate and safe plastification mechanism compared to the split-X brace and V-brace system.

3. For low rise frames, the use of V-braces significantly improved the stiffness of structure that provided highest capacity by factor of 3.2. However, for moderate rise frames, diagonal or split-X brace systems were provide to be more effective. This indicated that the number of stories had very effective on response characteristics of structure.

4. Irrespective of the brace configuration and building height, the use of BRBs led to uniform inter-storey drift along the height and that was belong to excellent nonlinear behavior of buckling restrained bracing system.

5. It was more effective to resist the lateral induced loads when the brace elements were inserted into the mid-spans than in the edge spans.

6. In general, it was evident that the frames with BRBs kept in the elastic range and plastification only occurred in the braces which might be changed easily after the damage. The BRBs dissipated more energy compare to other type of braces in the nonlinear range since they exhibited stable behavior and showed a spindle hysteresis behavior. As a result, the system provided a better behavior in the nonlinear range of deformations.

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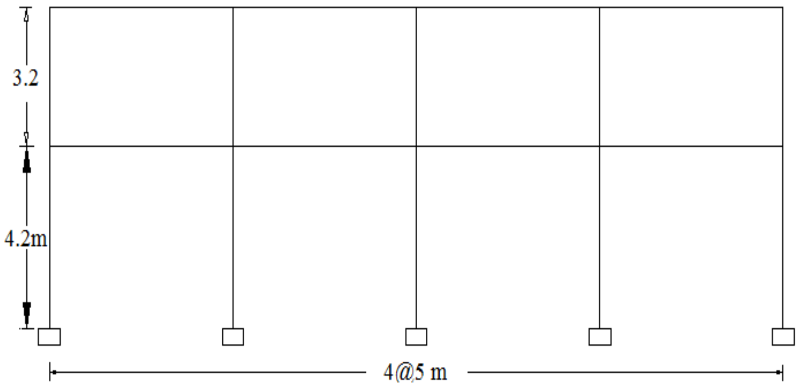
APPENDIX

Appendix A: OFs and BRBFs with different configuration and distribution



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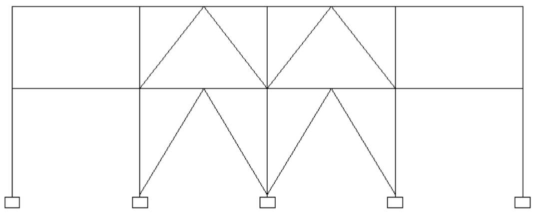
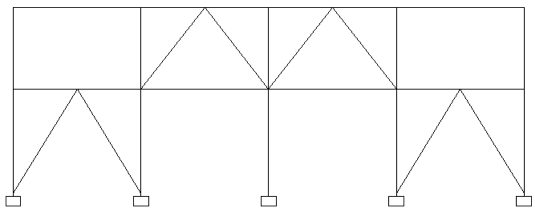


b)



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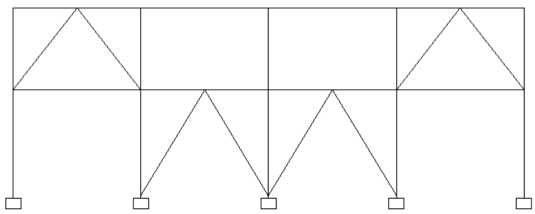

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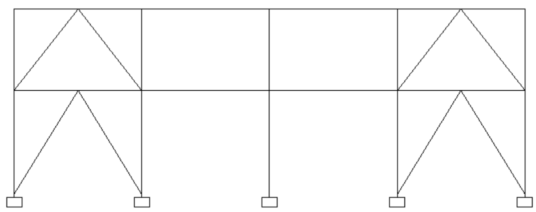


e)

Figure A-1 OF and BRBFs of two storey: a) OF, b) CHBF-case-1-2, c) CHBF-case2-2, d) CHBF-case-3-2, and e) CHBF-case-4-2

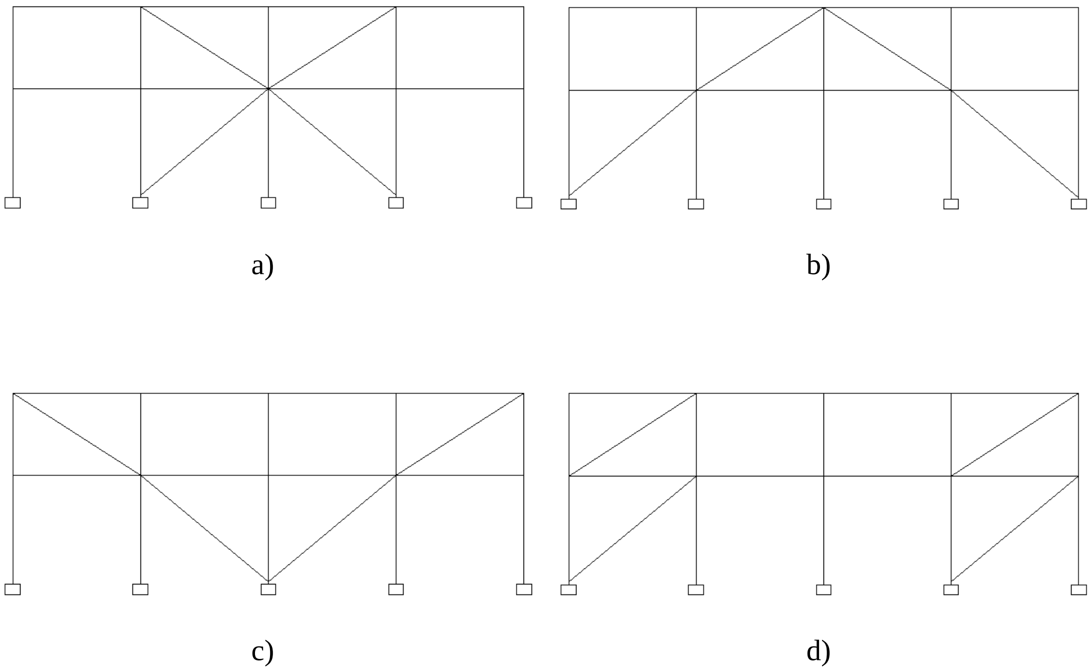


Figure A-2 BRBFs of two storey: a) DBF-case-1-2, b) DBF-case-2-2, c) DBF-case-3-2, and d) DBF-case-4-2

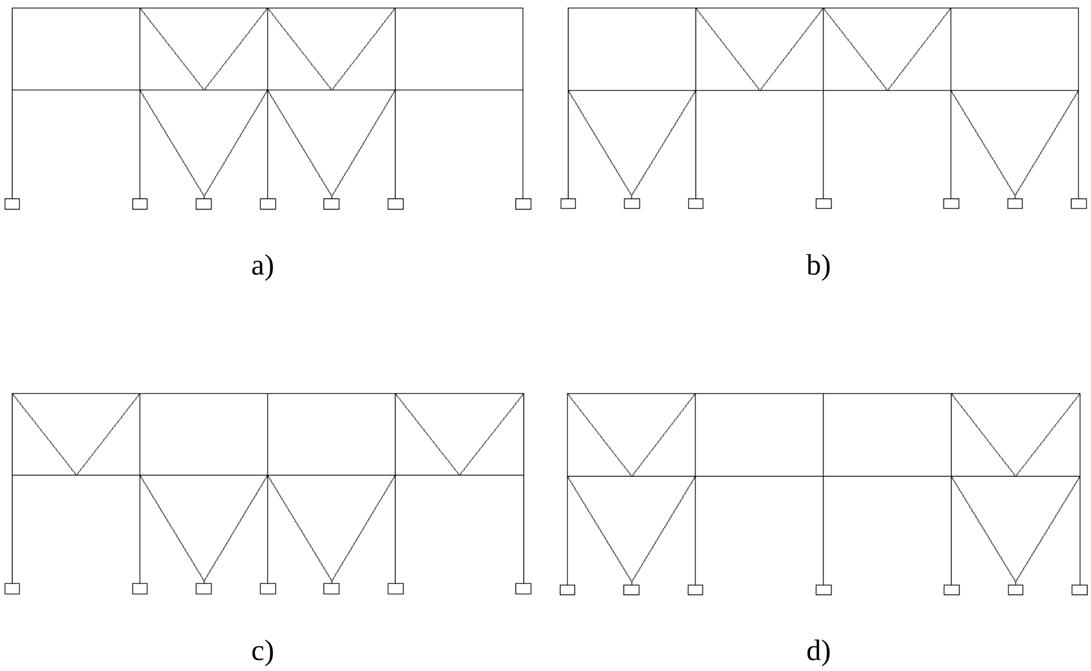


Figure A-3 BRBFs of two storey: a) SXBF-case-1-2, b) SXBF-case-2-2, c) SXBF-case-3-2, and d) SXBF-case-4-2

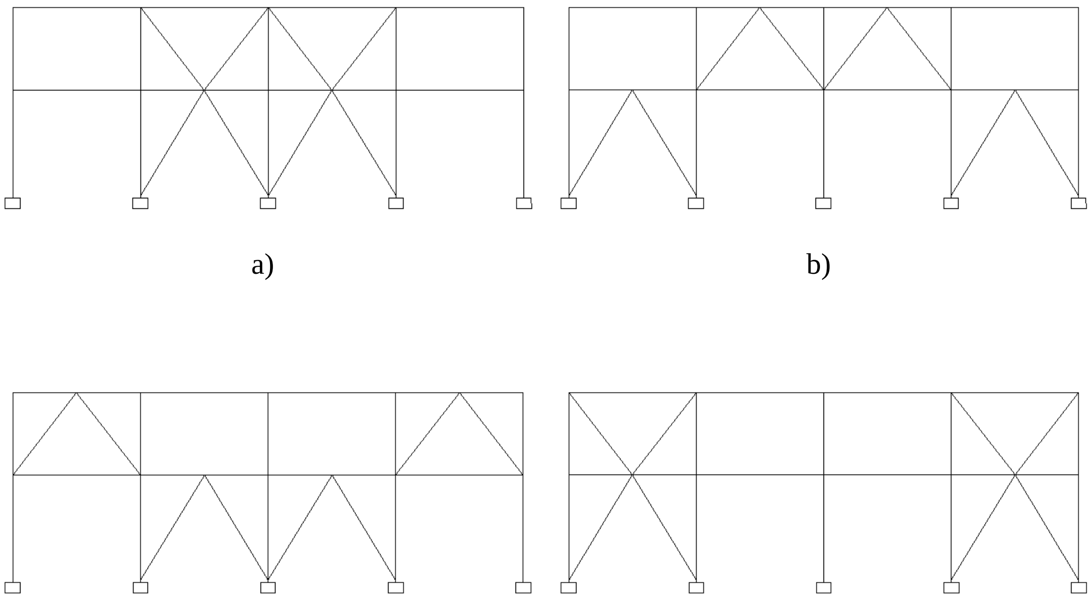


Figure A-4 BRBFs of two storey: a) VBF-case-1-2, b) VBF-case-2-2, c) VBF-case-3-2, and d) VBF-case-4-2

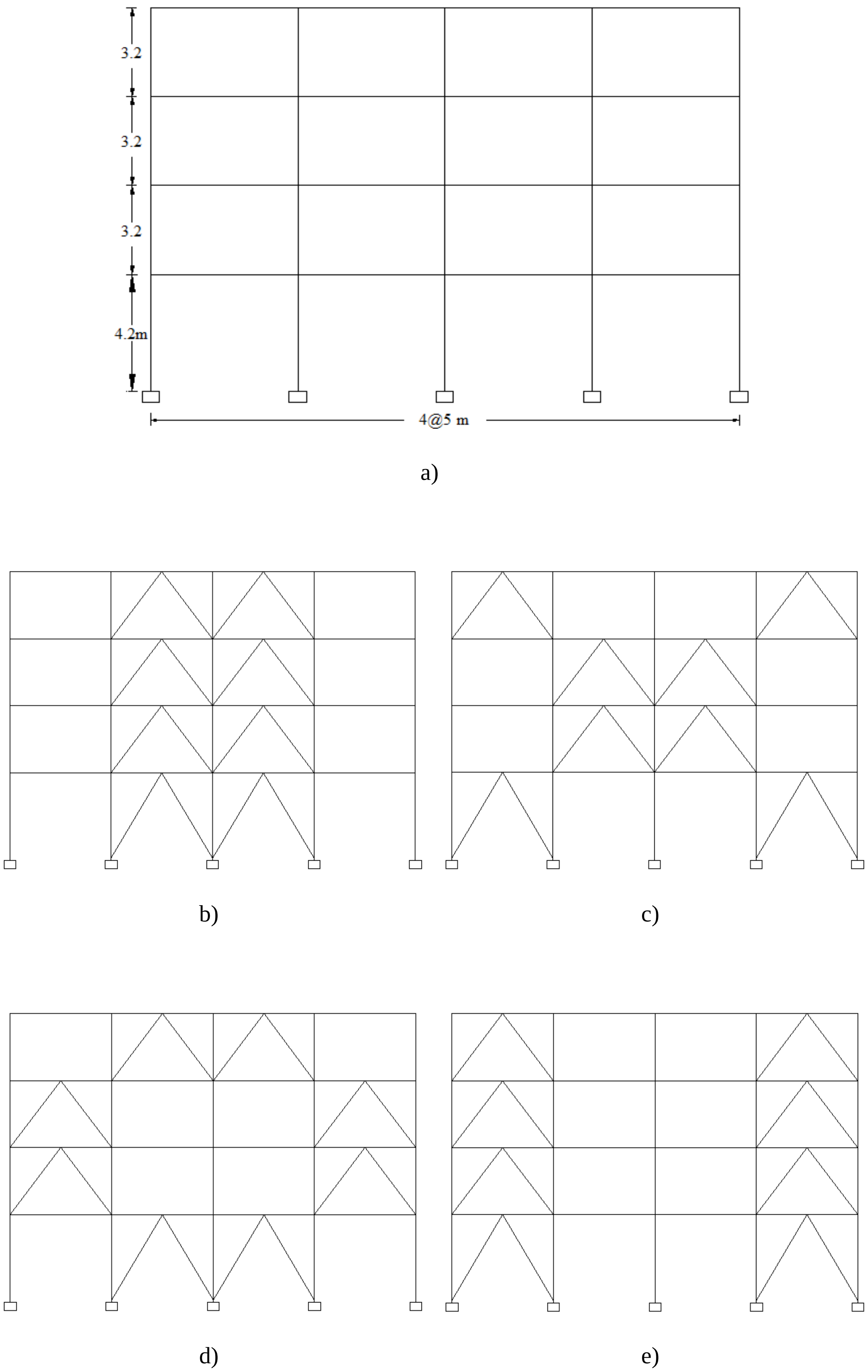


Figure A-5 OF and BRBFs of four storey: a) OF, b) CHBF-case-1-4, c) CHBF-case-2-4, d) CHBF-case-3-4, and e) CHBF-case-4-4

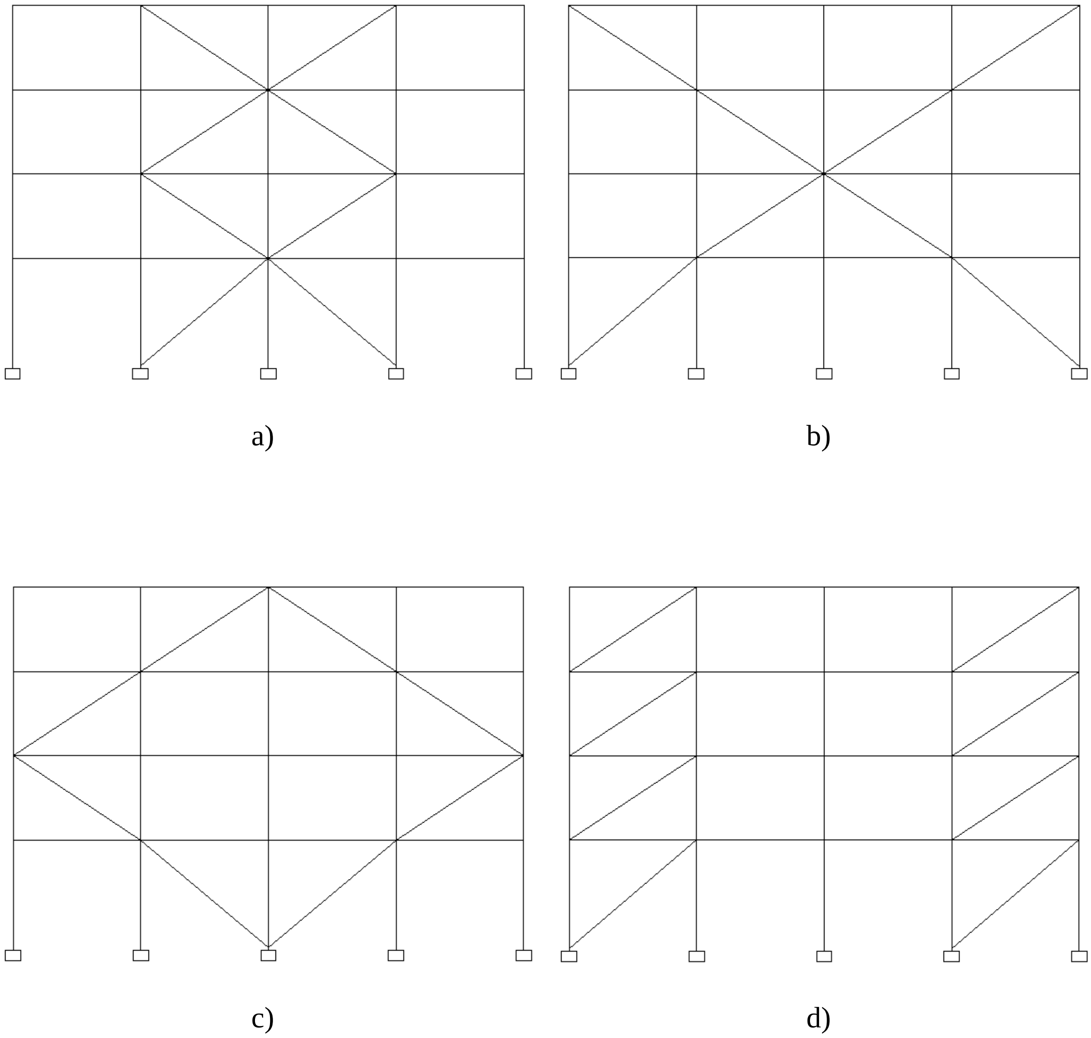
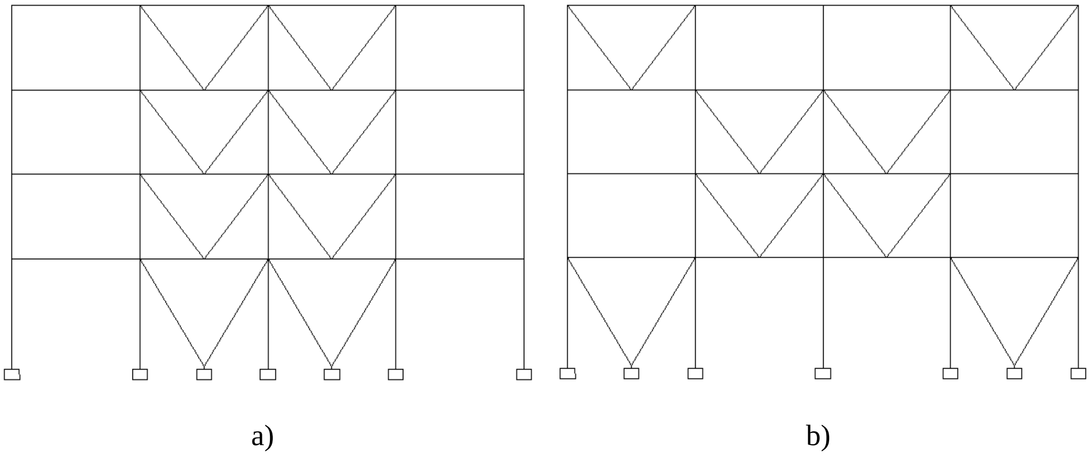
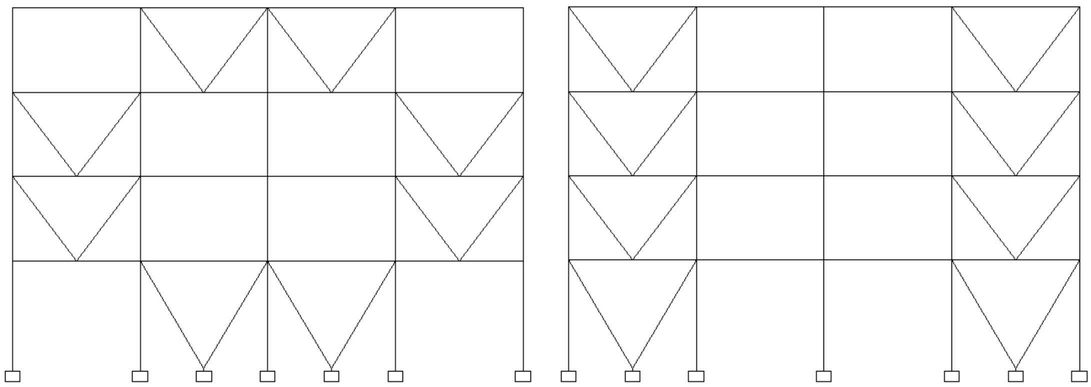


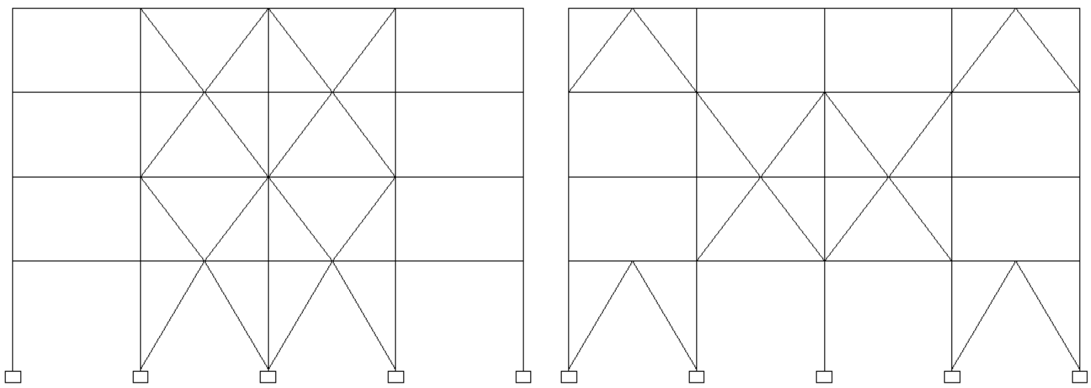
Figure A-6 BRBFs of four storey: a) DBF-case-1-4, b) DBF-case-2-4, c) DBF-case-3-4, and d) DBF-case-4-4



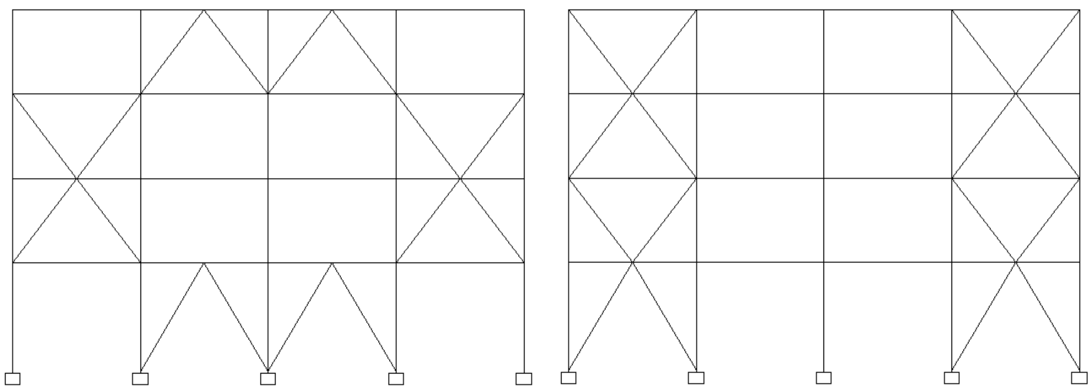


c) d)

Figure A-7 BRBFs of four storey: a) SXBF-case-1-4, b) SXBF-case-2-4, c) SXBF-case-3-4, and d) SXBF-case-4-4

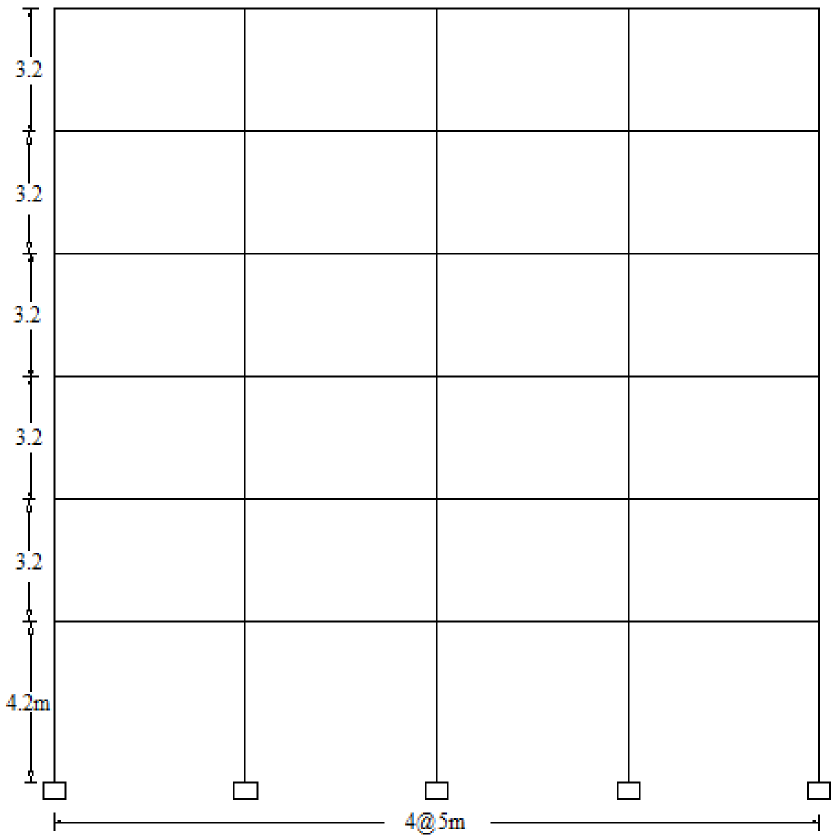


a) b)

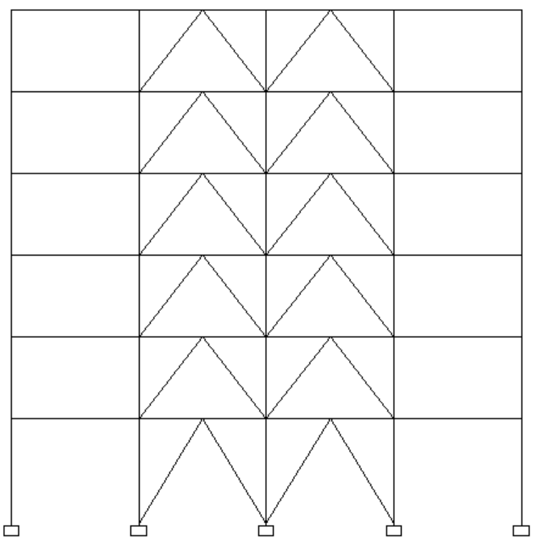


c) d)

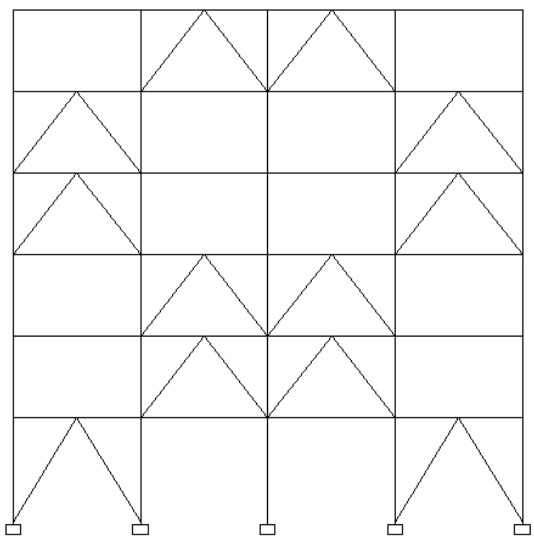
Figure A-8 BRBFs of four storey: a) VBF-case-1-4, b) VBF-case-2-4, c) VBF-case-3-4, and d) VBF-case-4-4



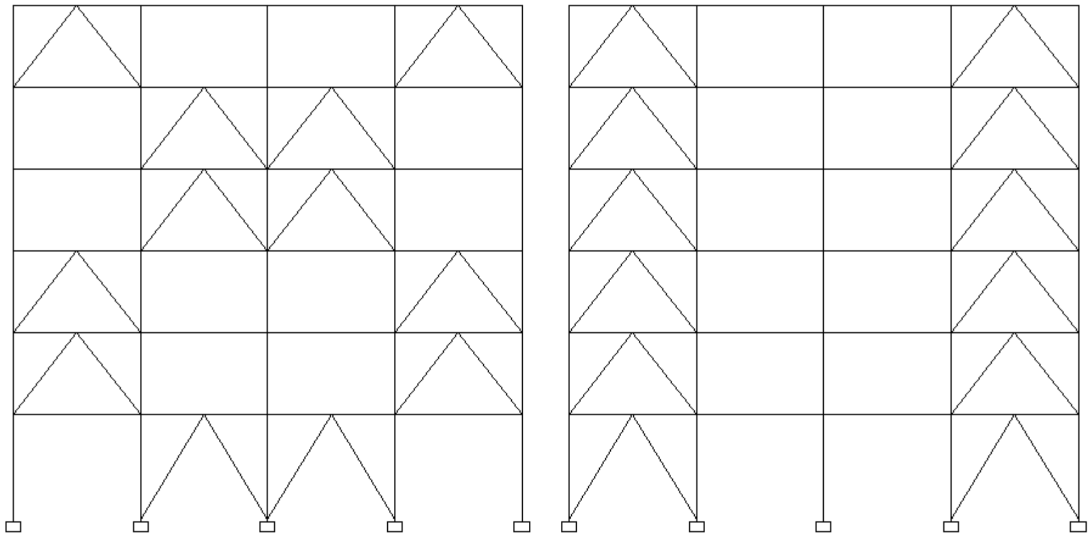
a)



b)

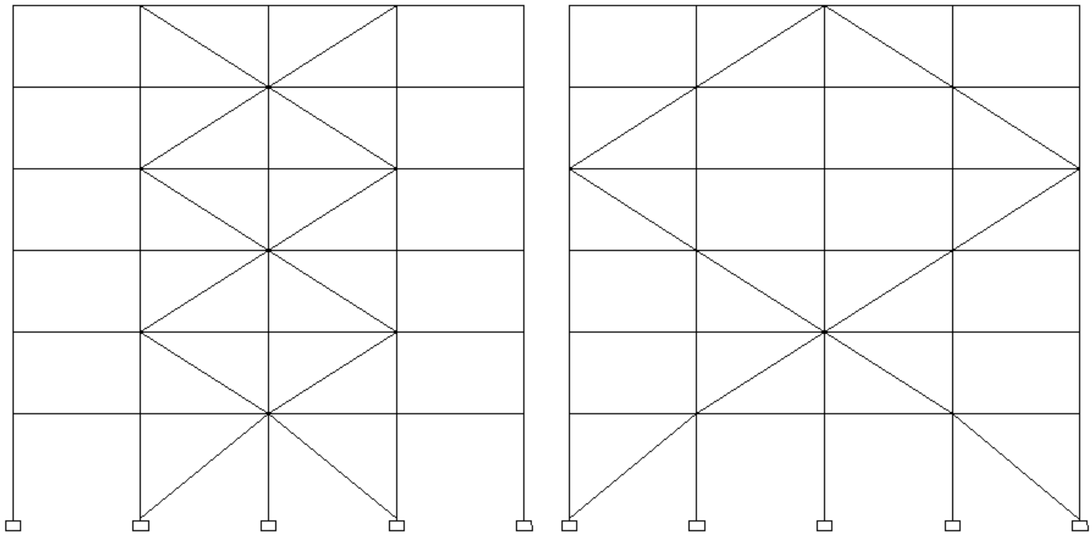


c)

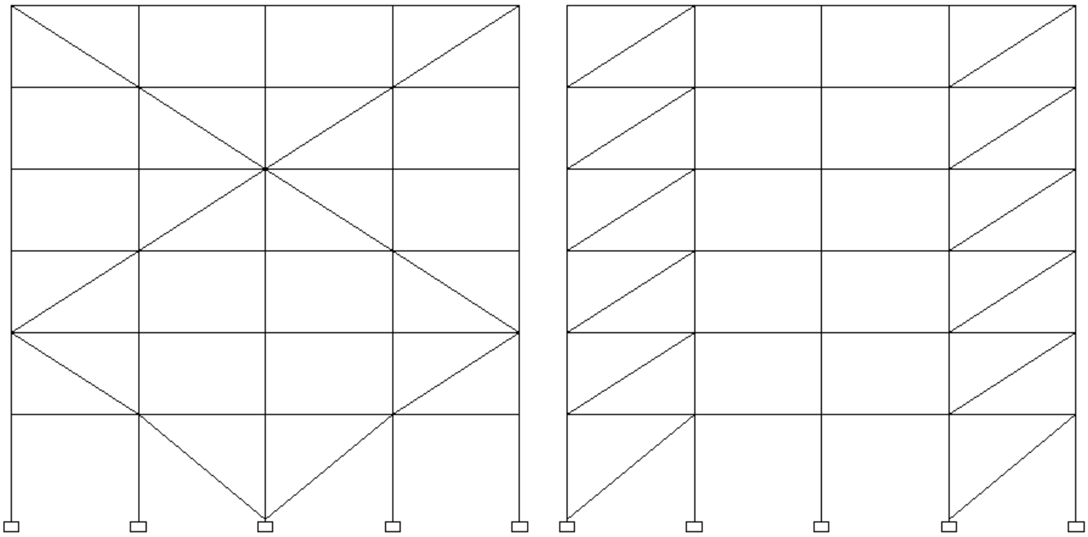


d) e)

Figure A-9 OF and BRBFs of six storey: a) OF, b) CHBF-case-1-6, c) CHBF-case-2-6, d) CHBF-case-3-6, and e) CHBF-case-4-6

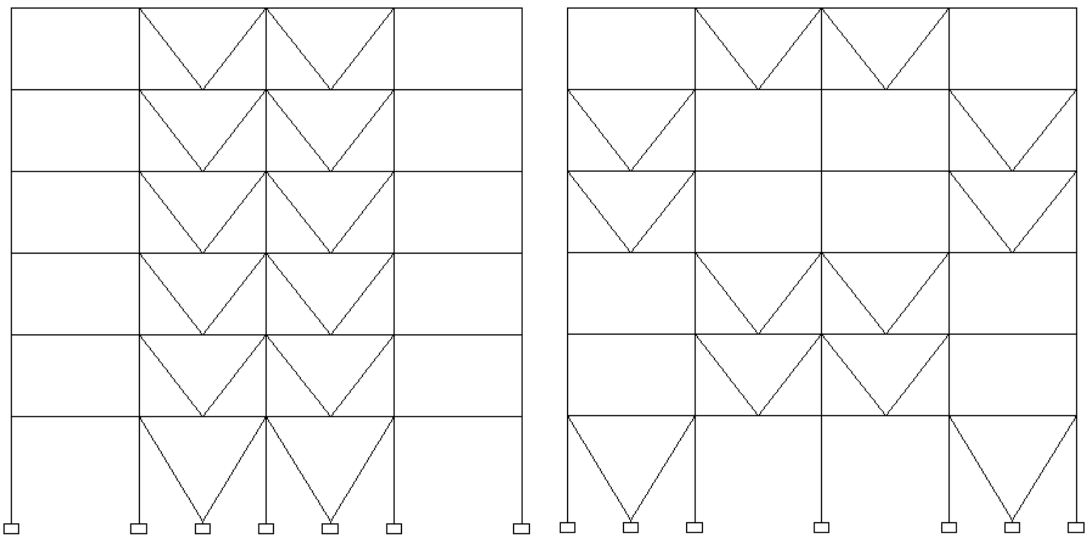


a) b)

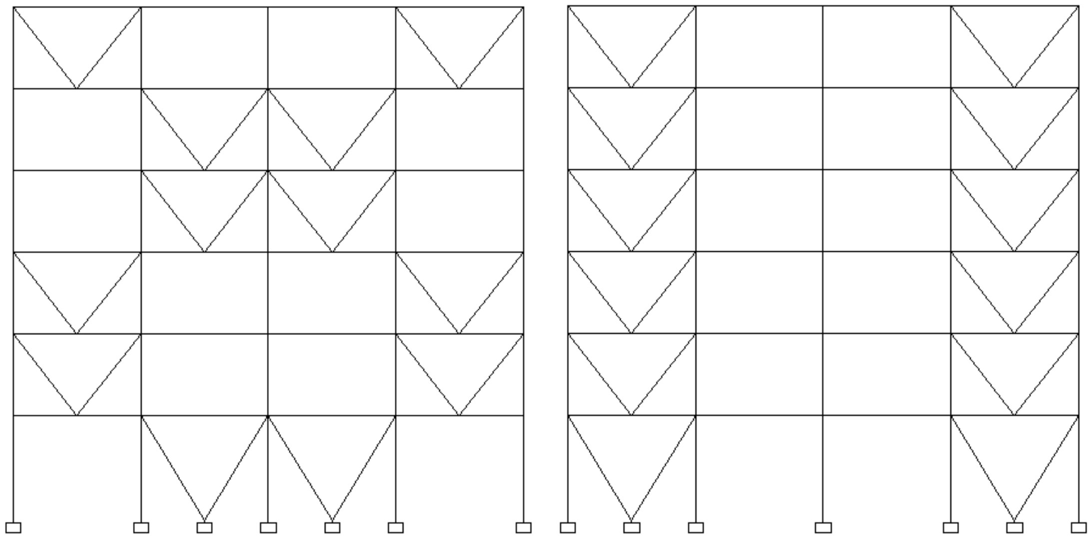


c) d)

Figure A-10 BRBFs of six storey: a) DBF-case-1-6, b) DBF-case-2-6, c) DBF-case-3-6, and d) DBF-case-4-6

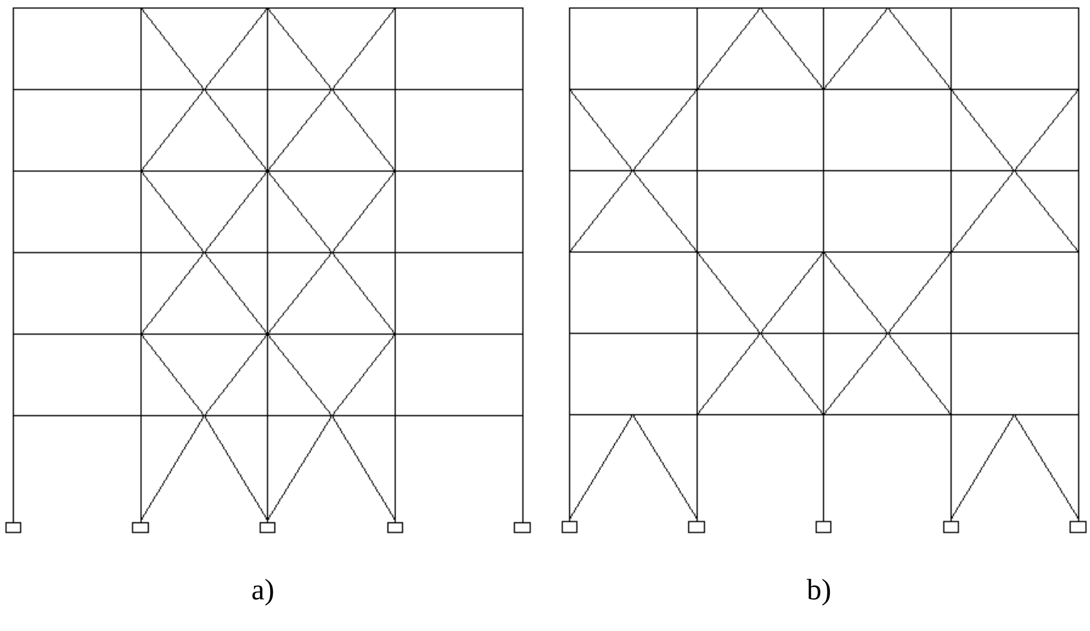


a) b)

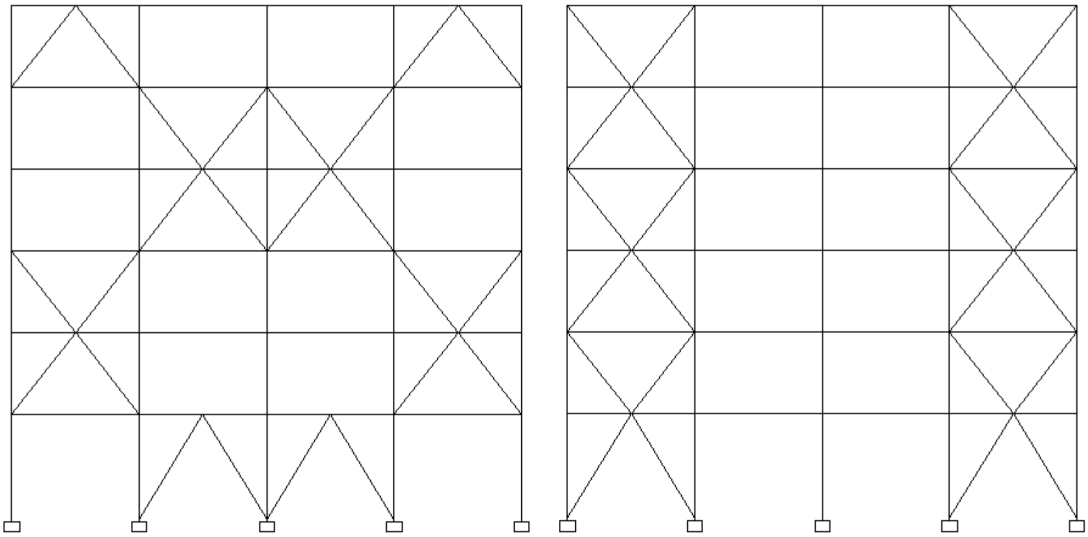


c) d)

Figure A-11 BRBFs of six storey: a) VBF-case-1-6, b) VBF-case-2-6, c) VBF-case-3-6, and d) VBF-case-4-6

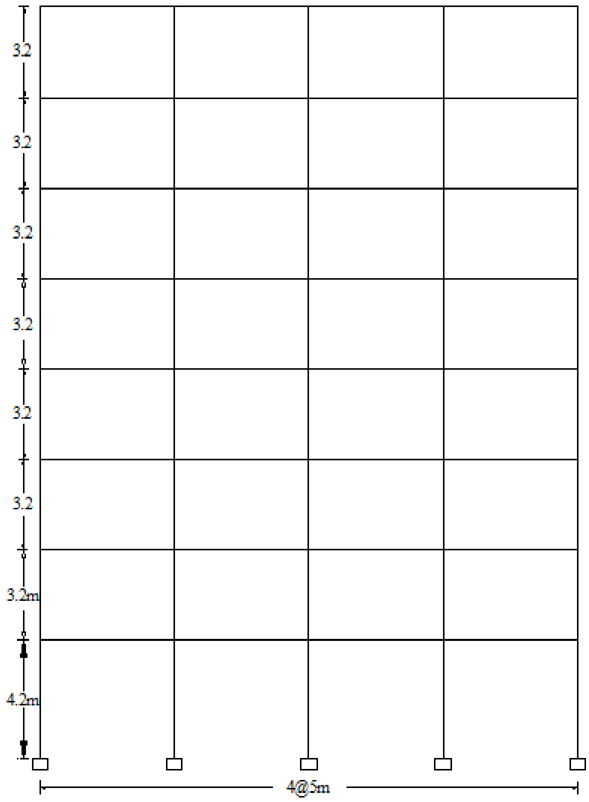


a) b)

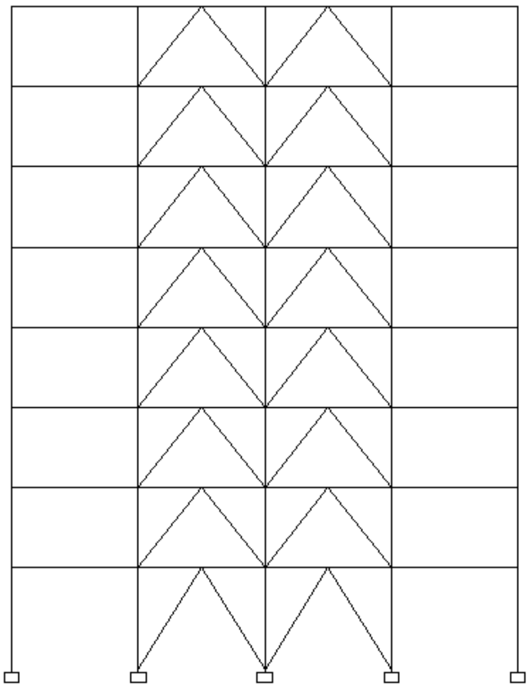


c) d)

Figure A-12 BRBFs of six storey: a) SXBF-case-1-6, b) SXBF-case-2-6, c) SXBF-case-3-6, and d) SXBF-case-4-6



a)



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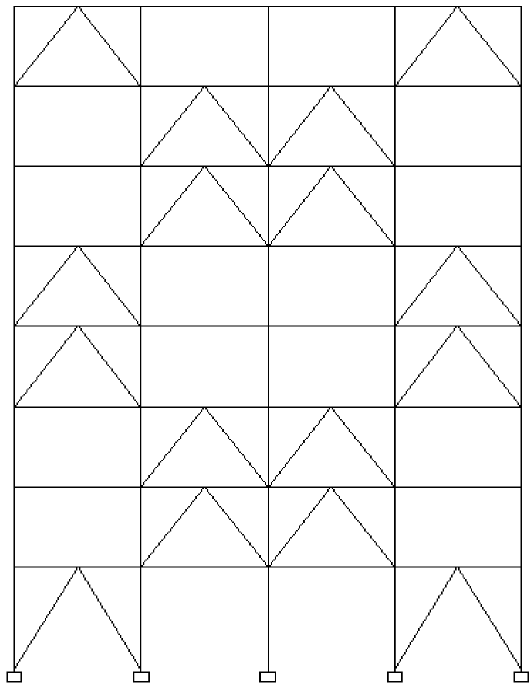

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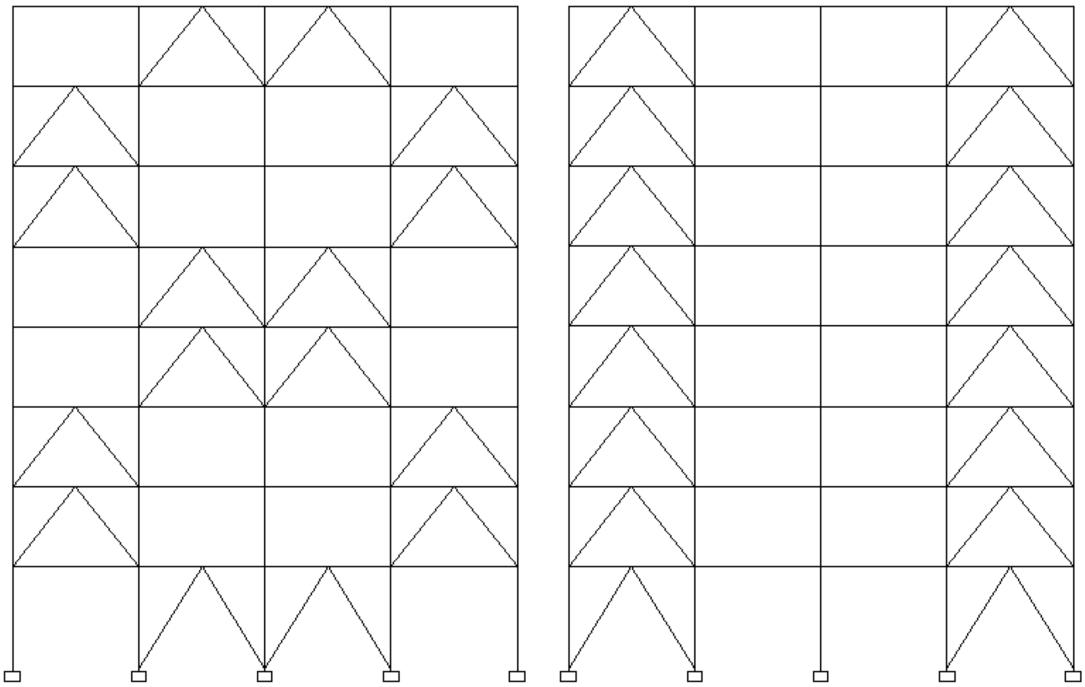
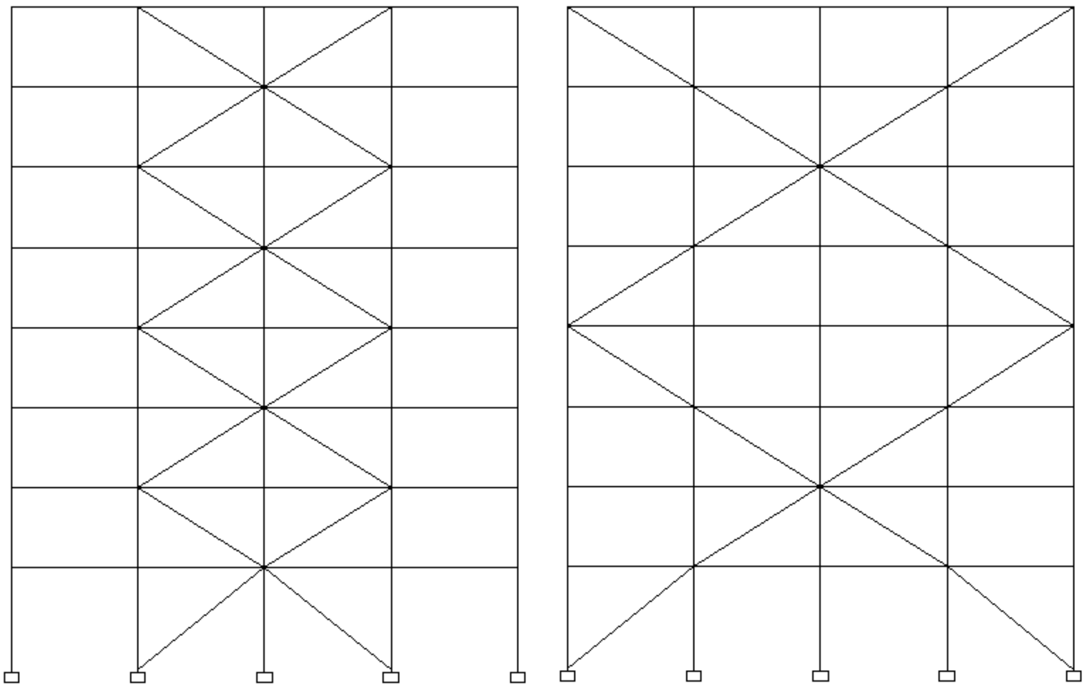
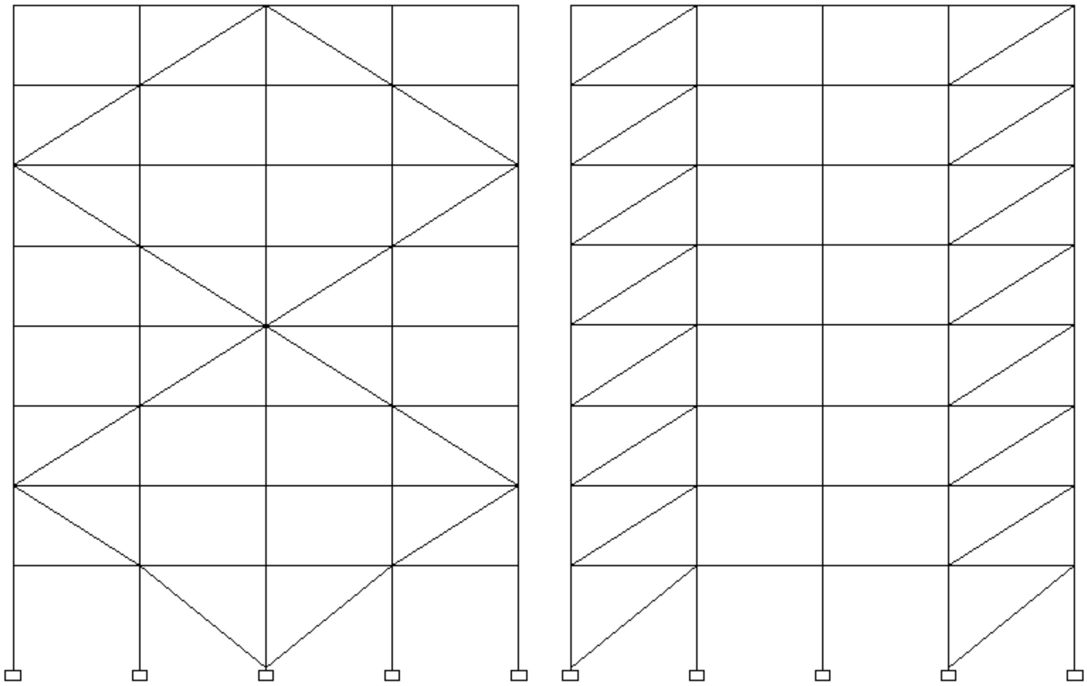


d) e)

Figure A-13 OF and BRBFs of eight storey: a) OF, b) CHBF-case-1-8, c) CHBF-case-2-8, d) CHBF-case-3-8, and e) CHBF-case-4-8

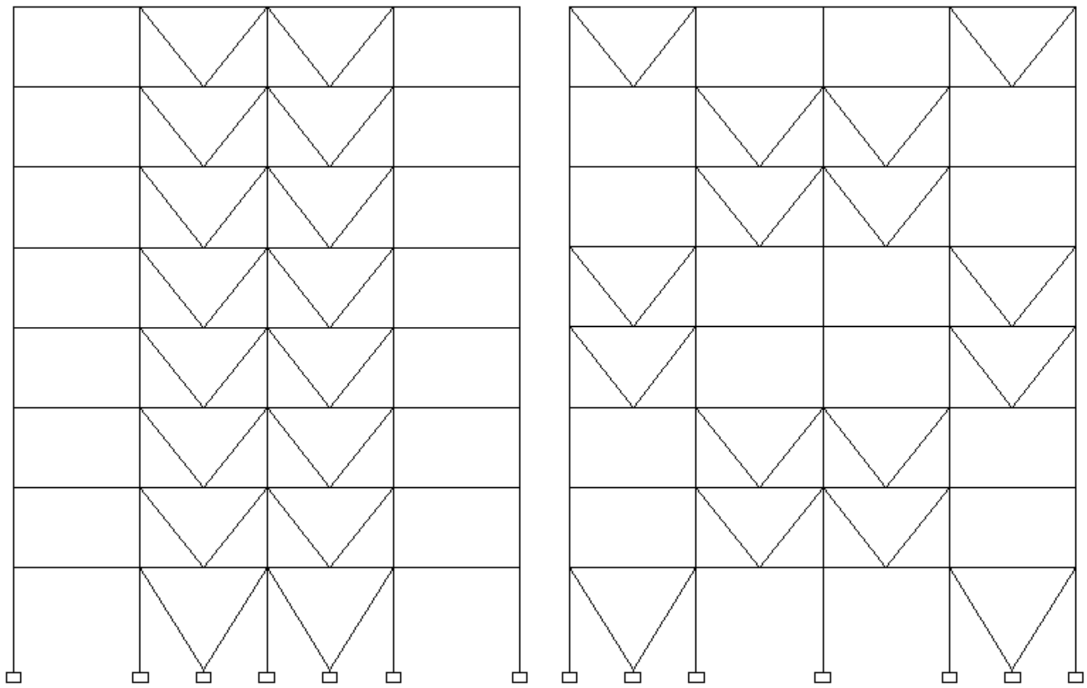


a) b)

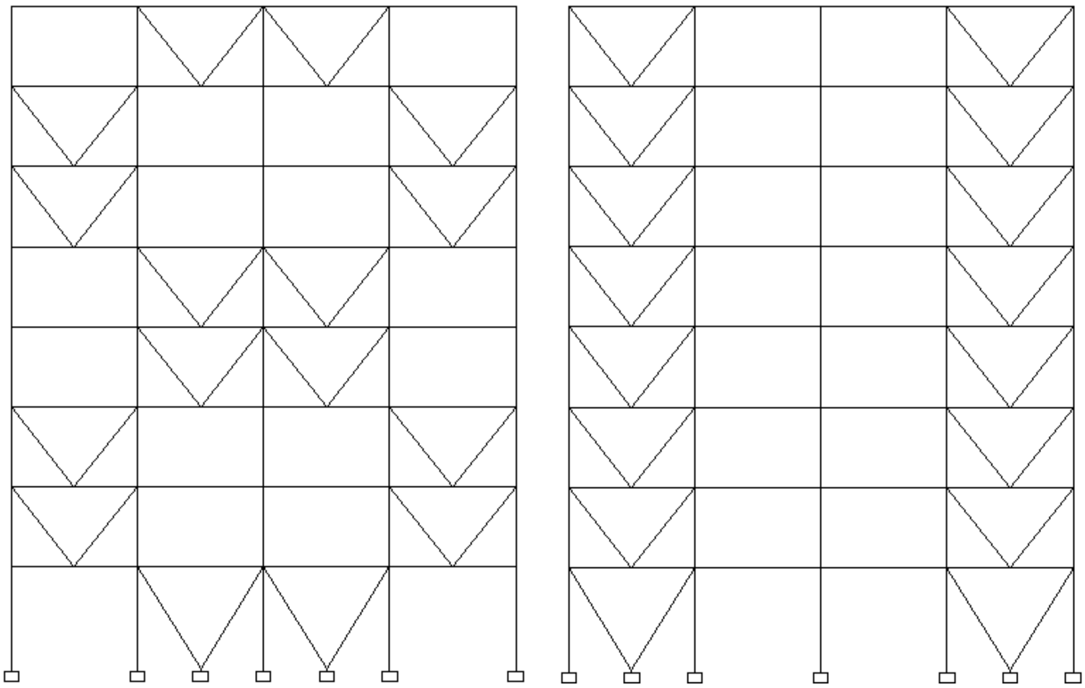


c) d)

Figure A-14 BRBFs of eight storey: a) DBF-case-1-8, b) DBF-case-2-8, c) DBF-case-3-8, and d) DBF-case-4-8

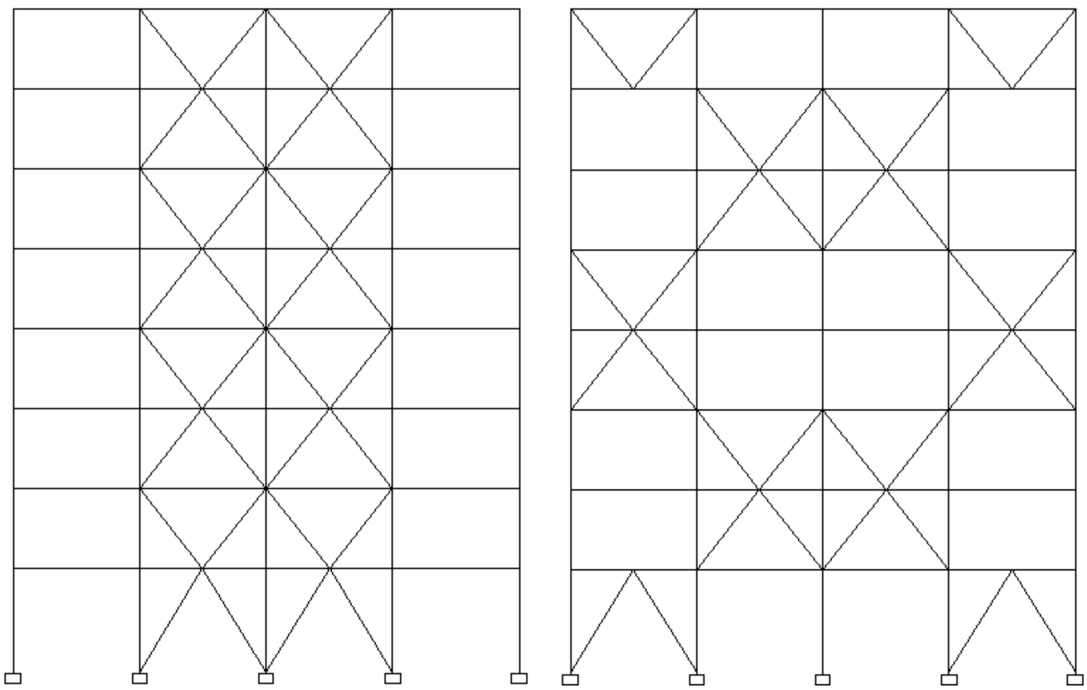


a) b)

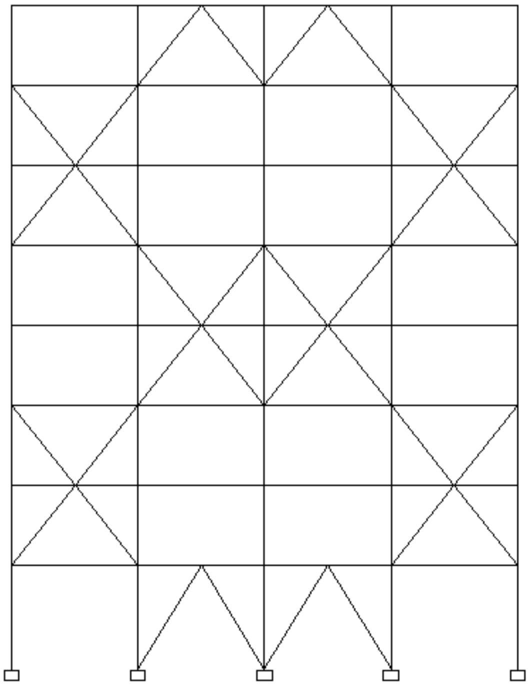


c) d)

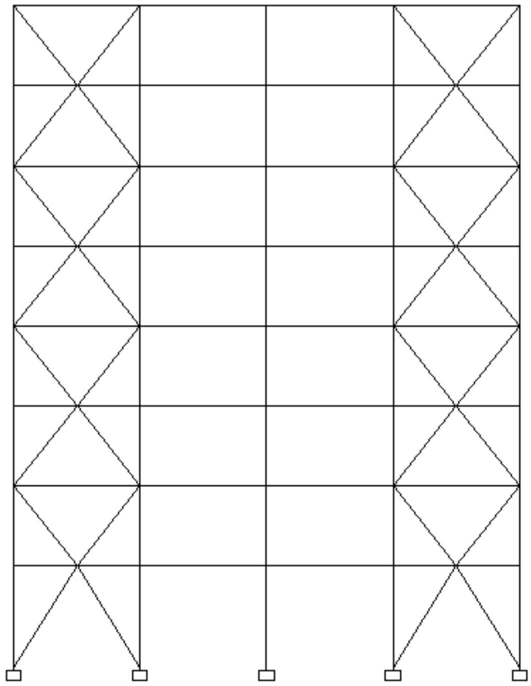
Figure A-15 BRBFs of eight storey: a) VBF-case-1-8, b) VBF-case-2-8, c) VBF-case-3-8, and d) VBF-case-4-8



a) b)



c)



d)

Figure A-16 BRBFs of eight storey: a) SXBF-case-1-8, b) SXBF-case-2-8, c) SXBF-case-3-8, and d) SXBF-case-4-8

Appendix B: Hinge location and performance level of BRBFs.

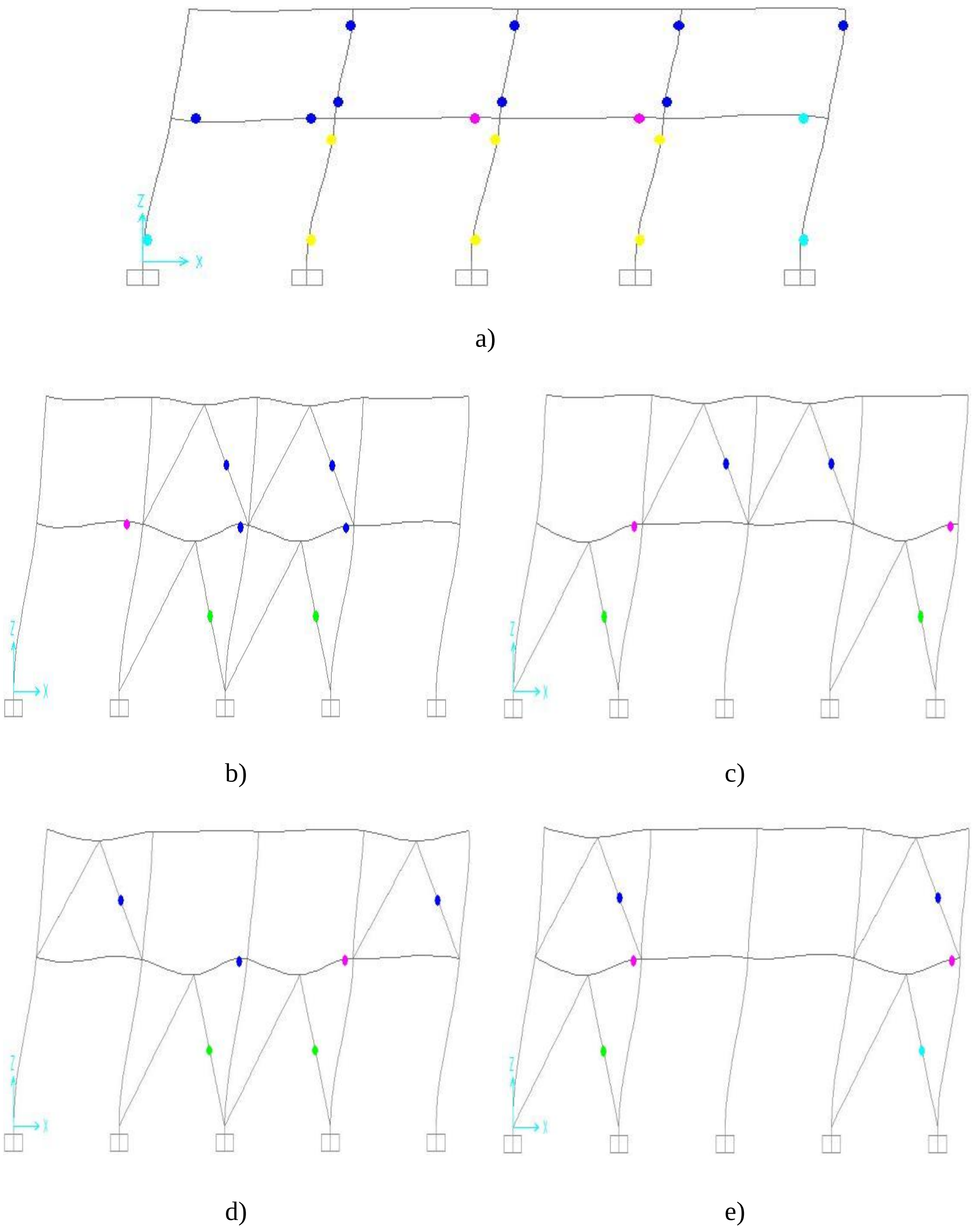
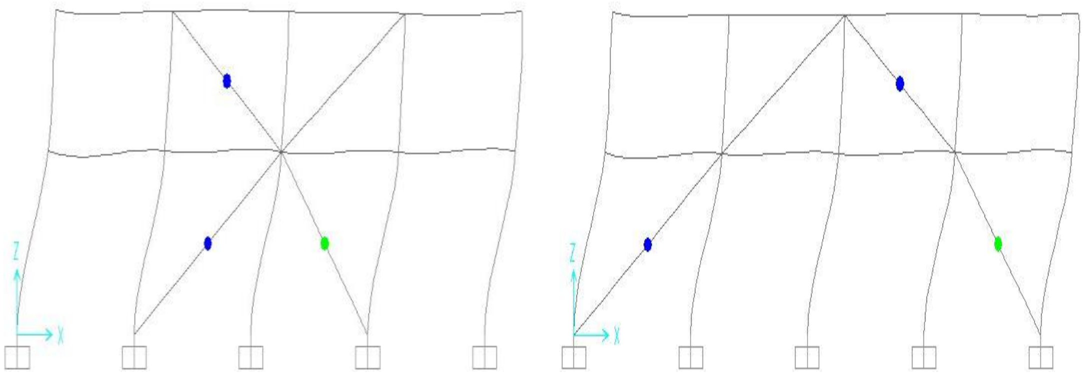
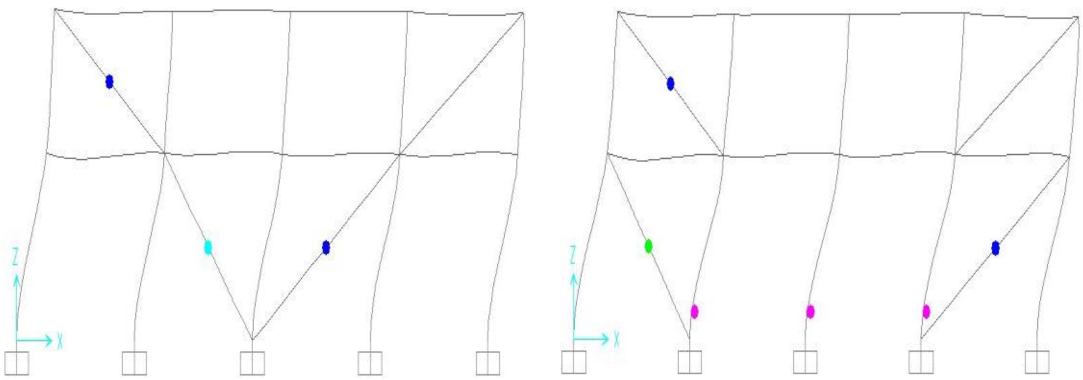


Figure B-1 OF and BRBFs of two storey: a) OF, b) CHBF-case-1-2, c) CHBF-case-2-2, d) CHBF-case-3-2, and e) CHBF-case-4-2

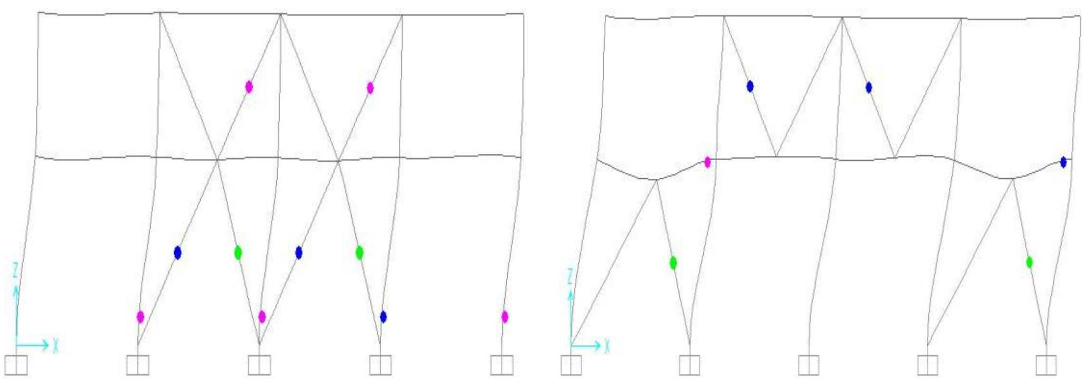


a) b)

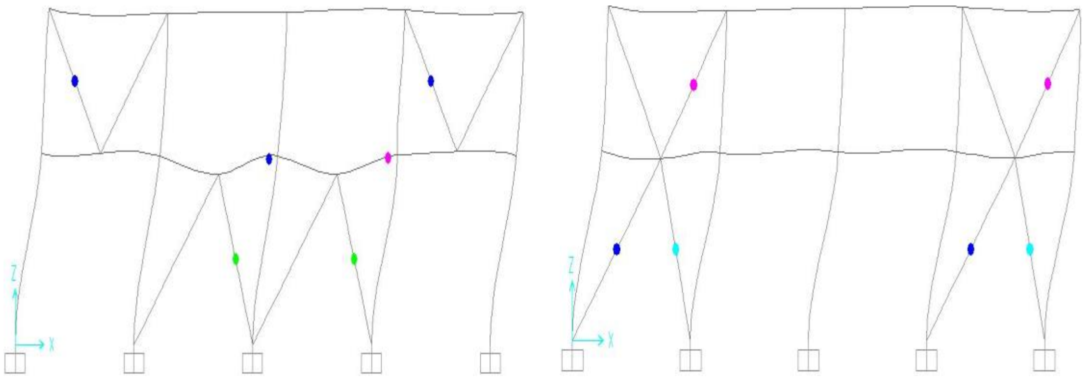


c) d)

Figure B-2 BRBFs of two storey: a) DBF-case-1-2, b) DBF-case-2-2, c) DBF-case-3-2, and d) DBF-case-4-2

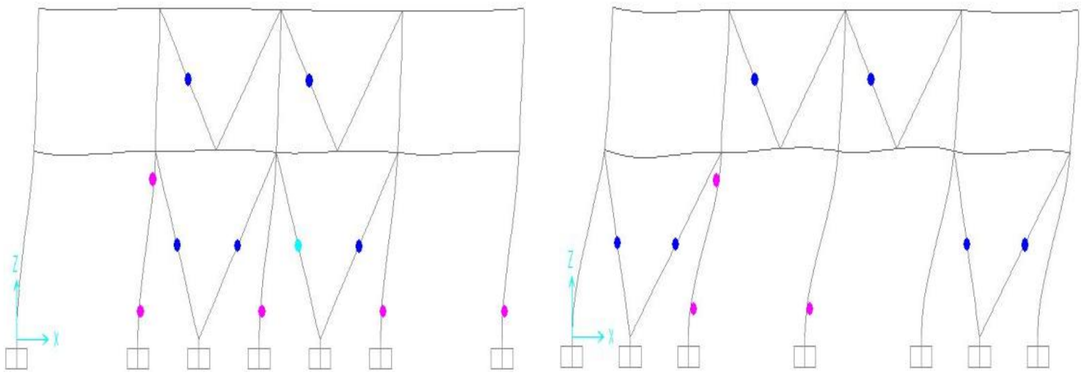


a) b)

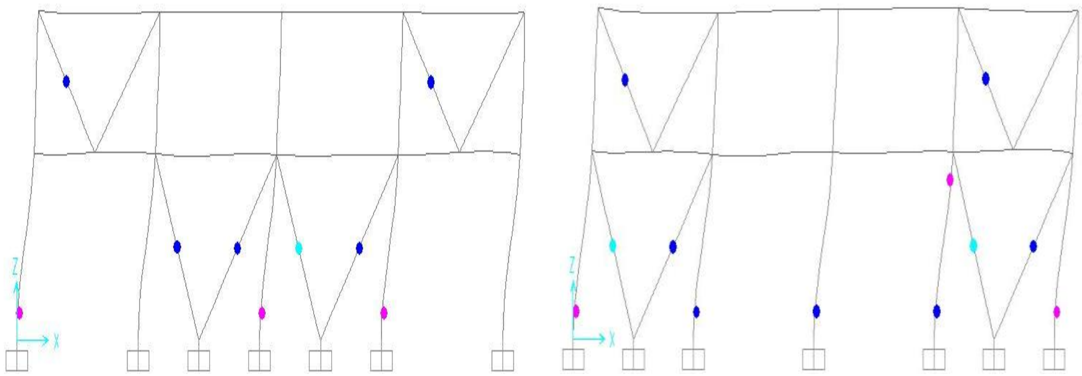


c) d)

Figure B-3 BRBFs of two storey: a) SXBF-case-1-2, b) SXBF-case-2-2, c) SXBF-case-3-2, and d) SXBF-case-4-2

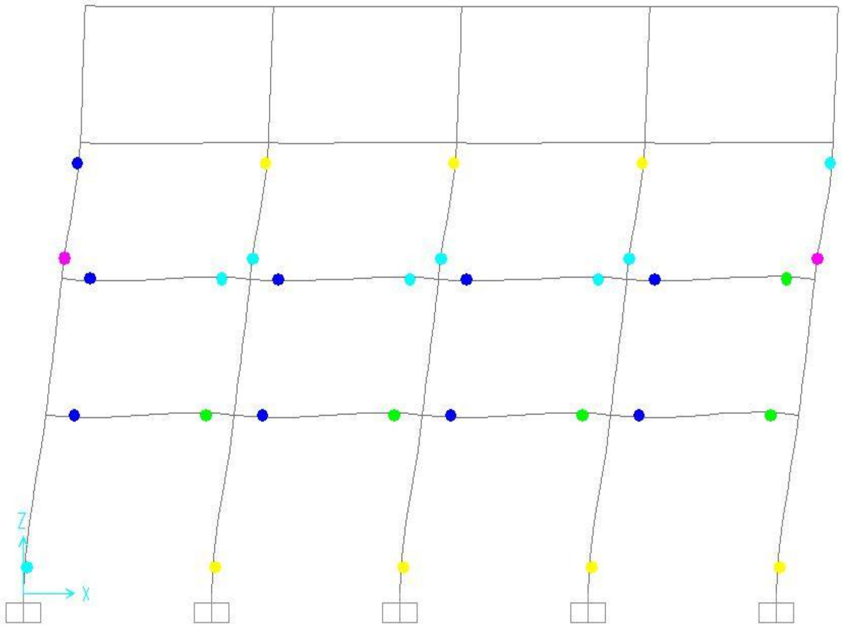


a) b)

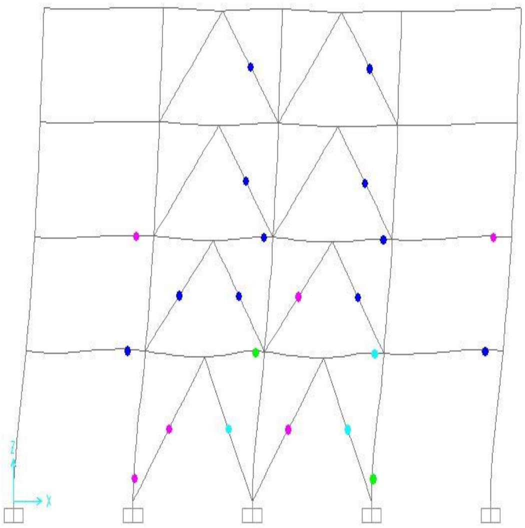


c) d)

Figure B-4 BRBFs of two storey: a) VBF-case-1-2, b) VBF-case-2-2, c) VBF-case-3-2, and d) VBF-case-4-2



a)



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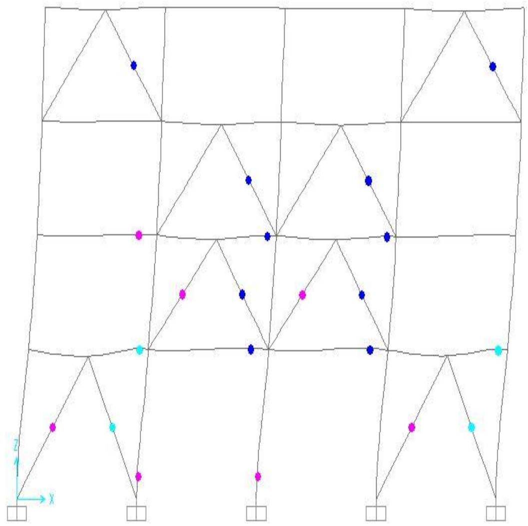

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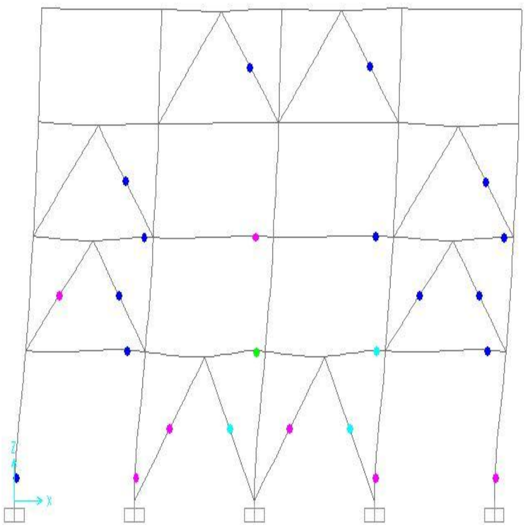
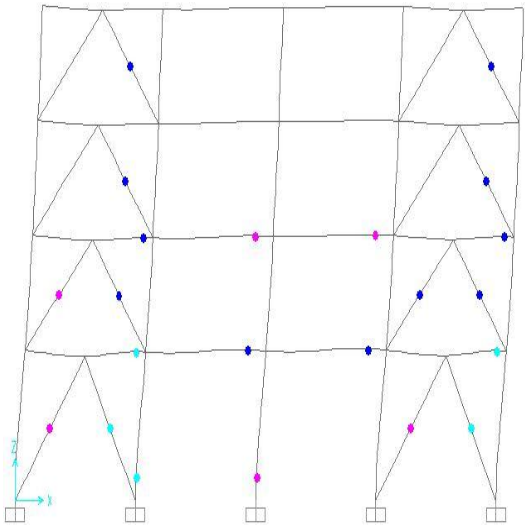
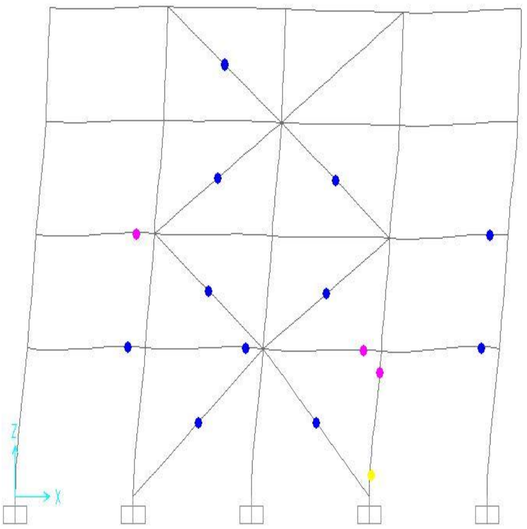


d)

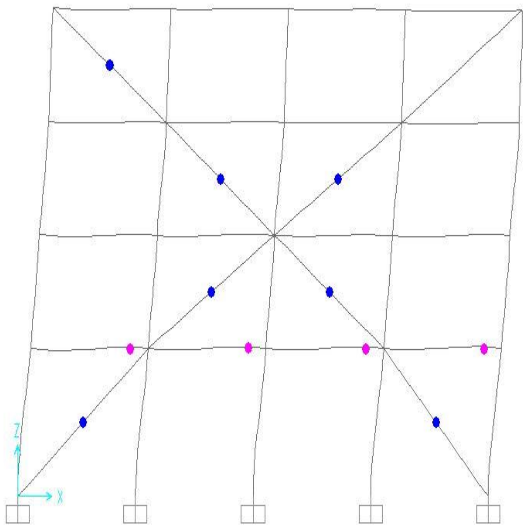


e)

Figure B-5 OF and BRBFs of four storey: a) OF, b) CHBF-case-1-4, c) CHBF-case-2-4, d) CHBF-case-3-4, and e) CHBF-case-4-4



a)



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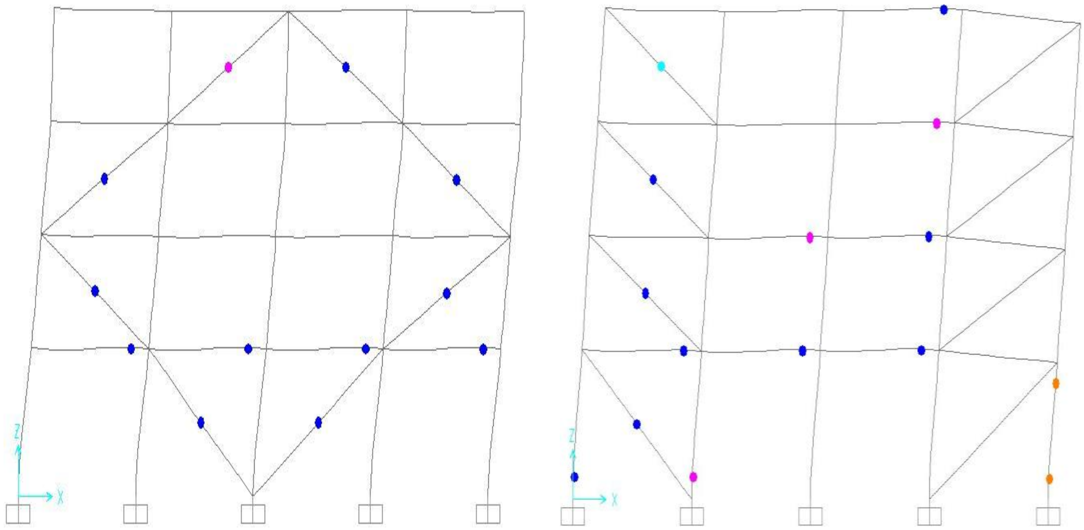
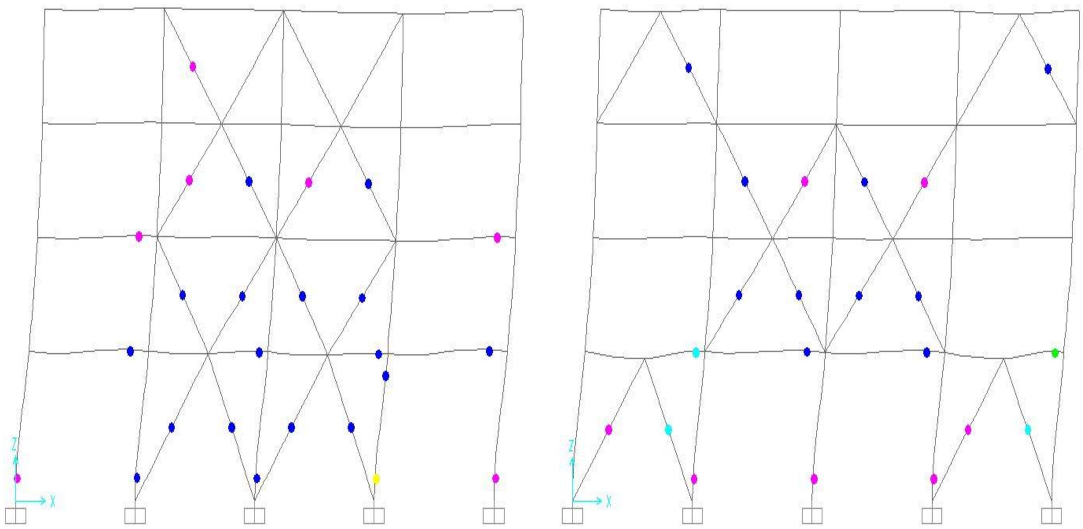
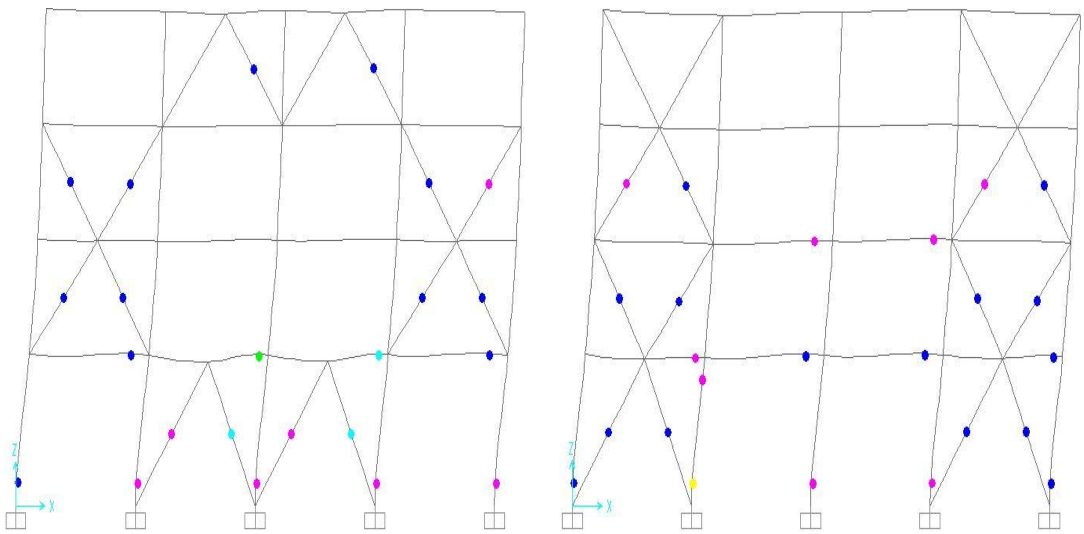


c) d)

Figure B-6 BRBFs of four storey: a) DBF-case-1-4, b) DBF-case-2-4, c) DBF-case-3-4, and d) DBF-case-4-4

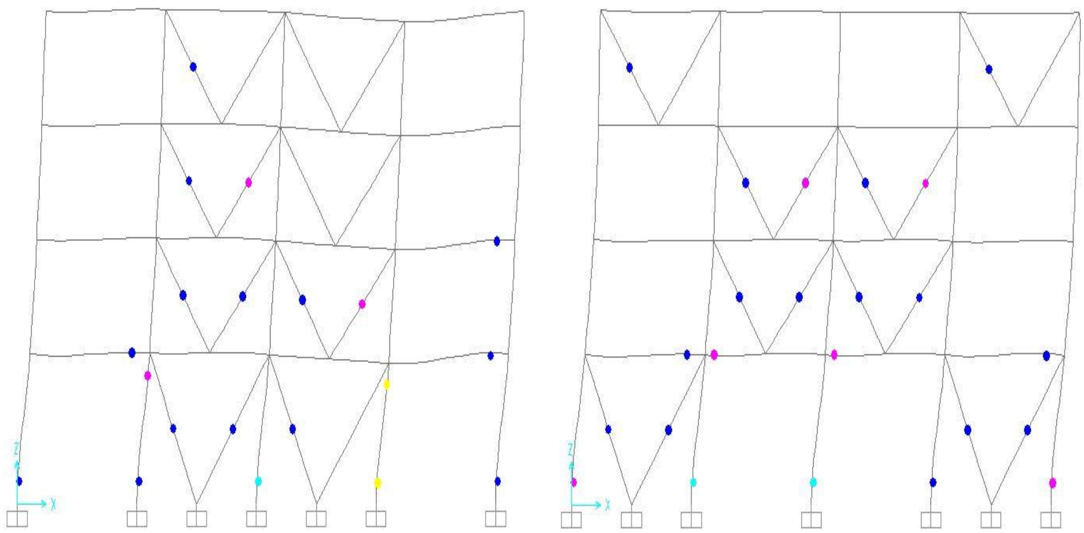


a) b)

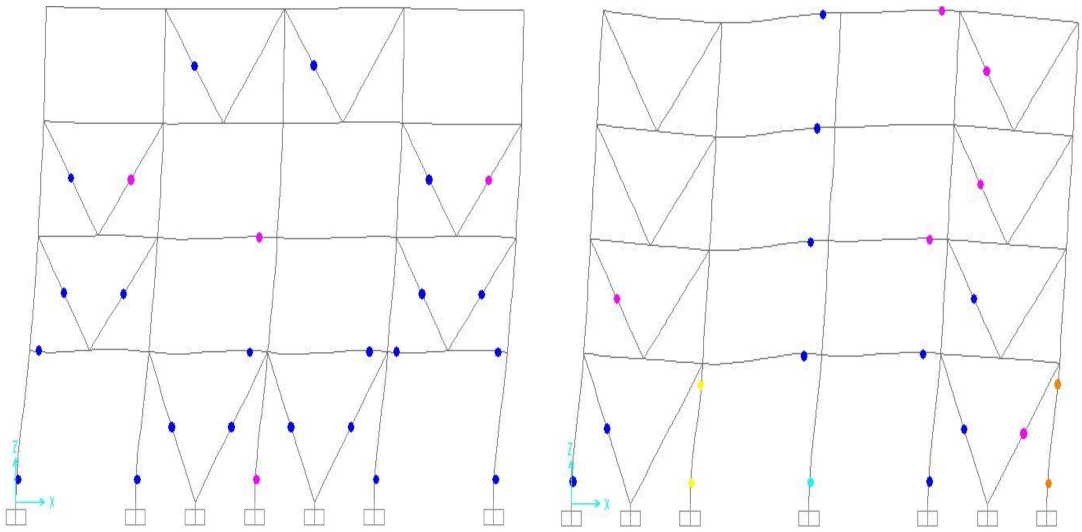


c) d)

Figure B-7 BRBFs of four storey: a) SXBF-case-1-4, b) SXBF-case-2-4, c) SXBF-case-3-4, and d) SXBF-case-4-4

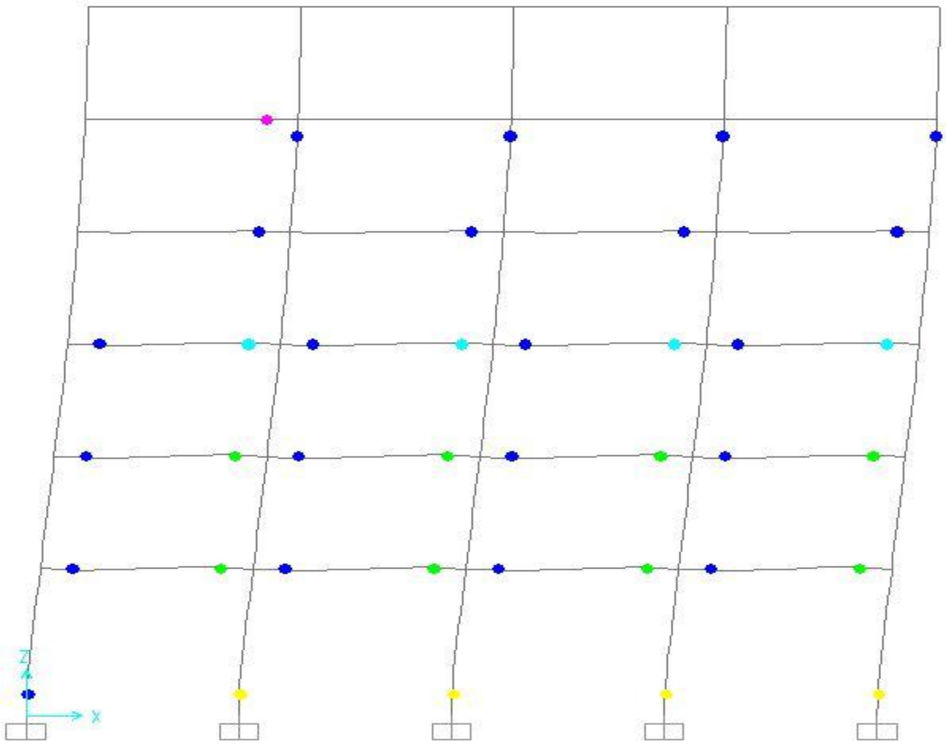


a) b)

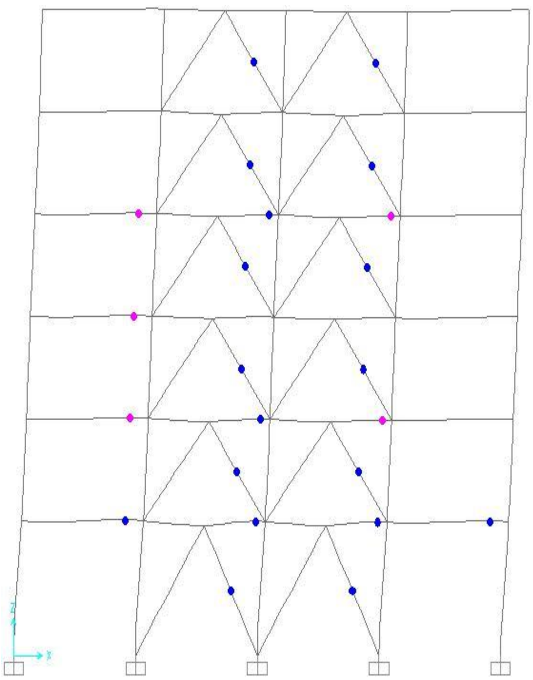


c) d)

Figure B-8 BRBFs of four storey a) VBF-case-1-4, b) VBF-case-2-4, c) VBF-case-3-4, and d) VBF-case-4-4



a)



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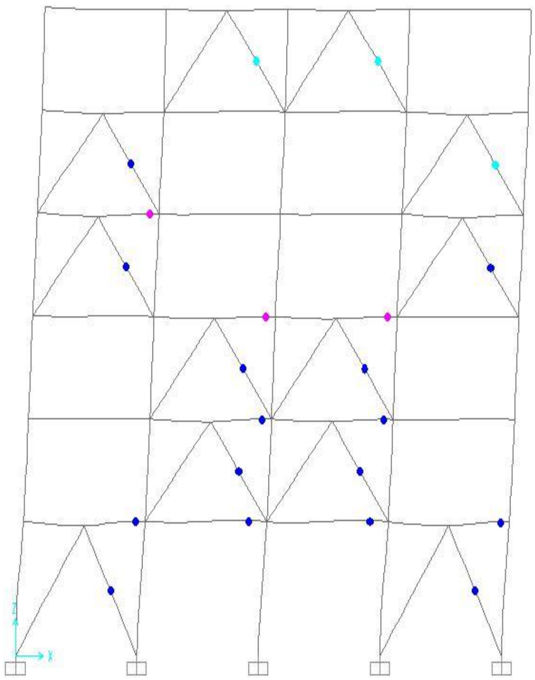

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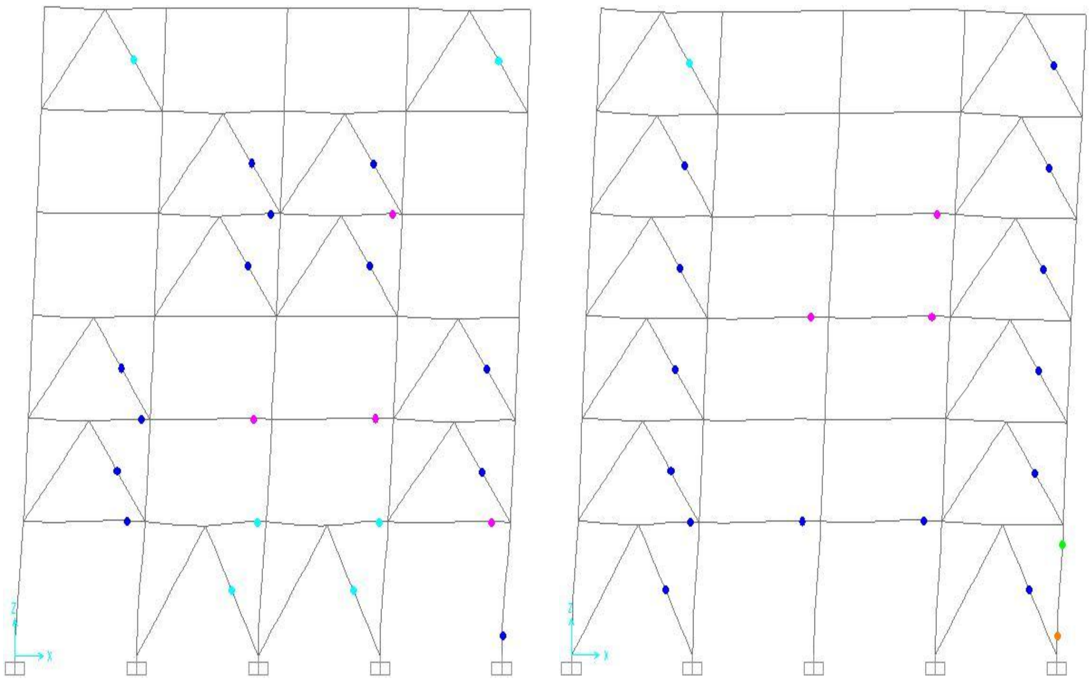
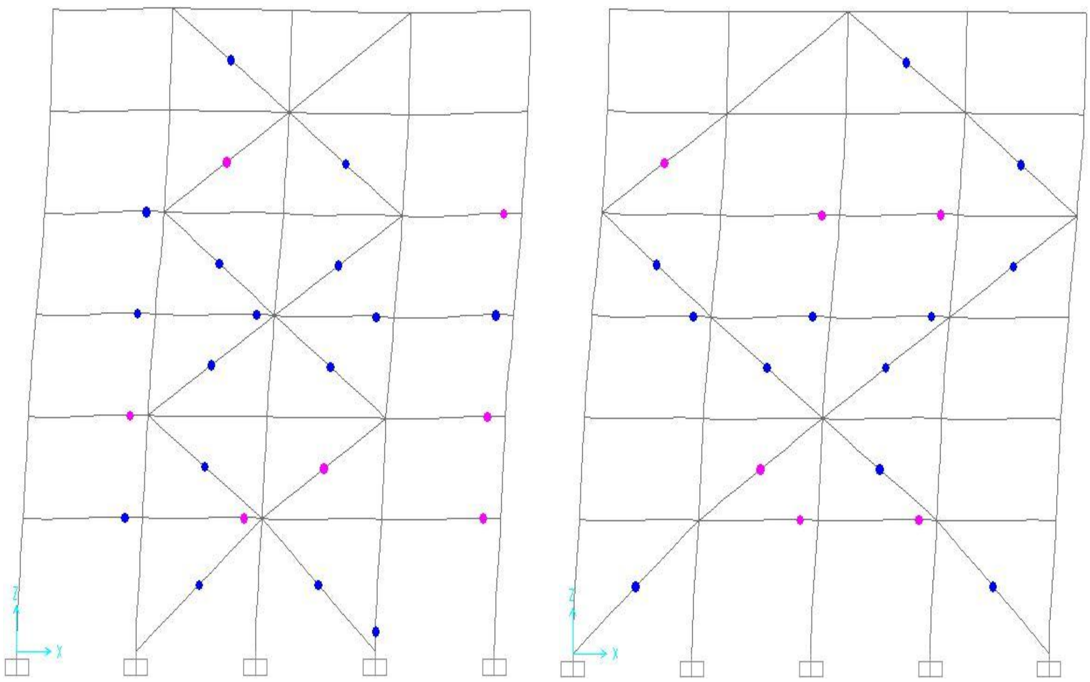
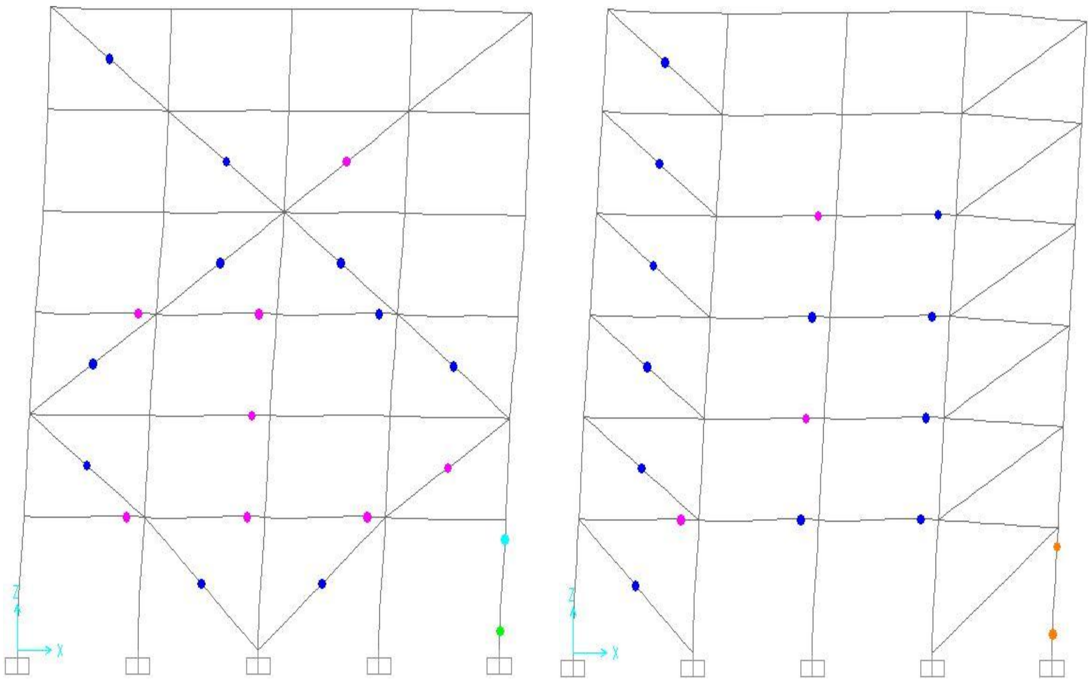


d) e)

Figure B-9 OF and BRBFs of six storey: a) OF, b) CHBF-case-1-6, c) CHBF-case-2-6, d) CHBF-case-3-6 and e) CHBF-case-4-6

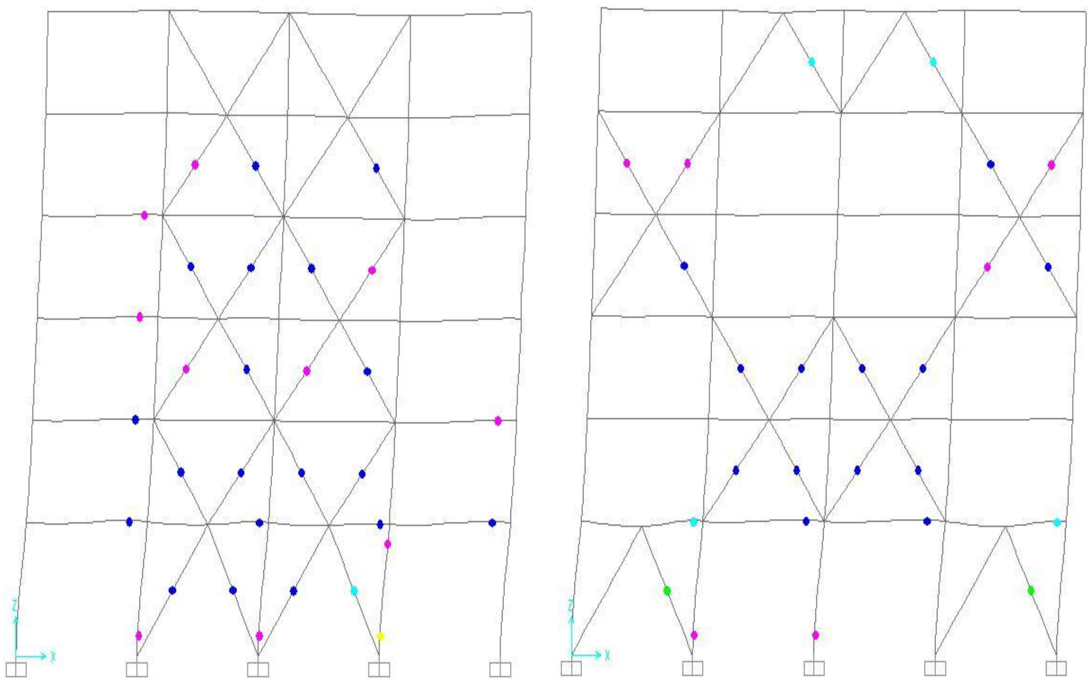


a) b)

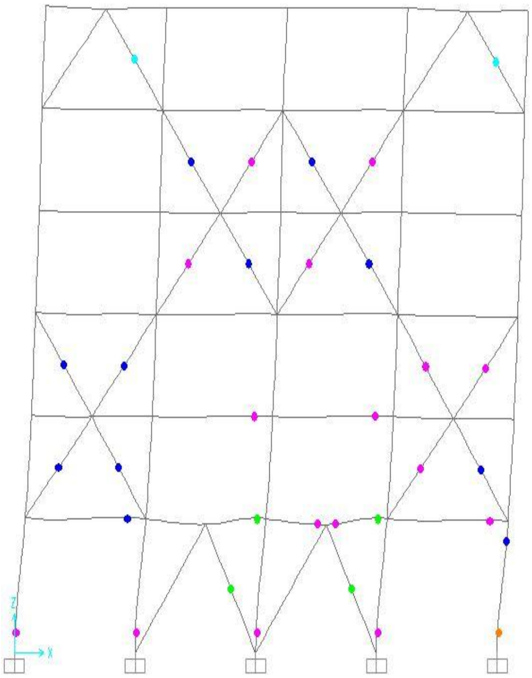


c) d)

Figure B-10 BRBFs of six storey: a) DBF-case-1-6, b) DBF-case-2-6, c) DBF-case-3-6, and d) DBF-case-4-6



a) b)



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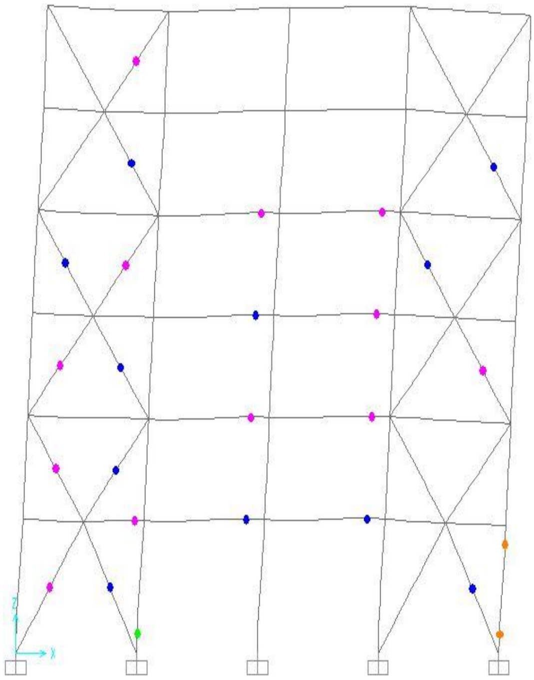
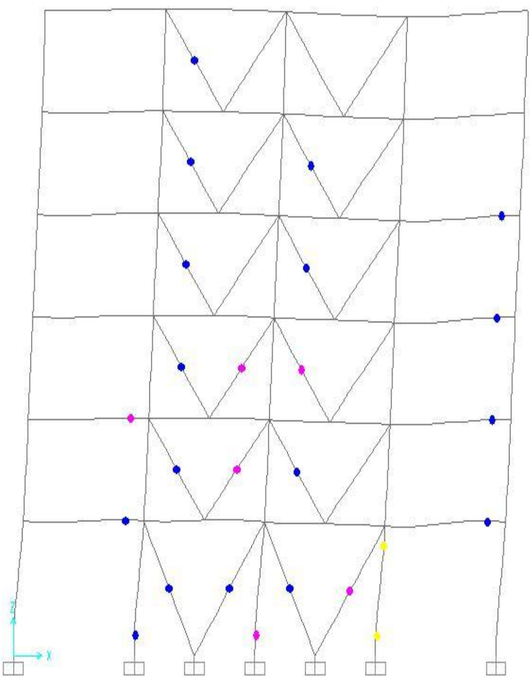


d)

Figure B-11 BRBFs of six storey: a) VBF-case-1-6, b) VBF-case-2-6, c) VBF-case-3-6, and d) VBF-case-4-6



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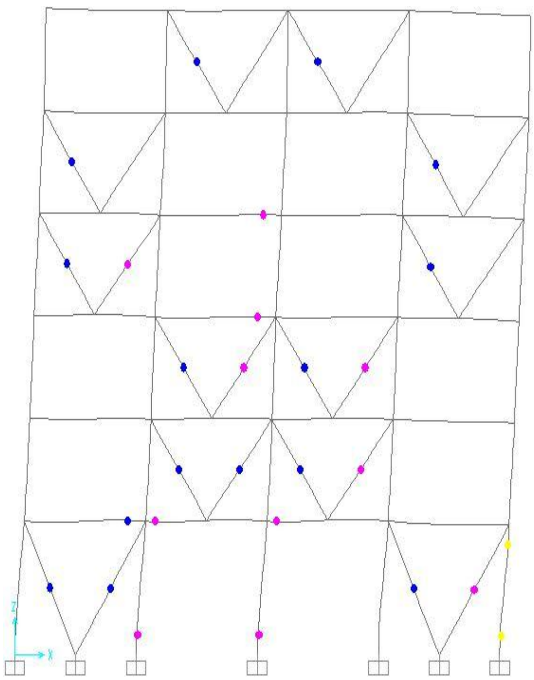

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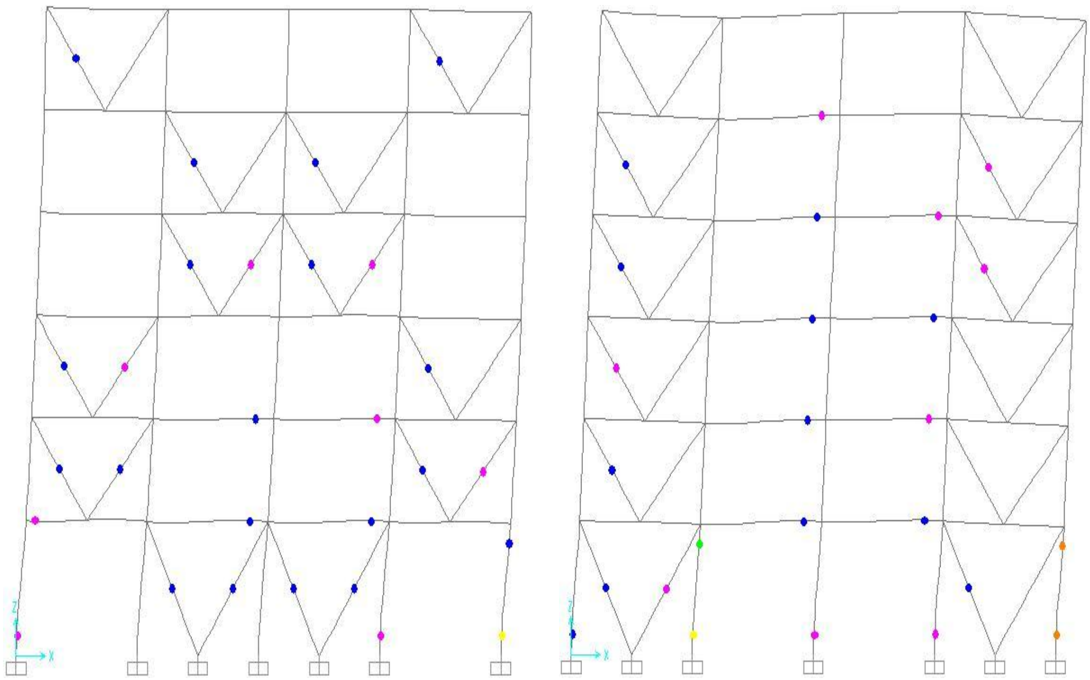
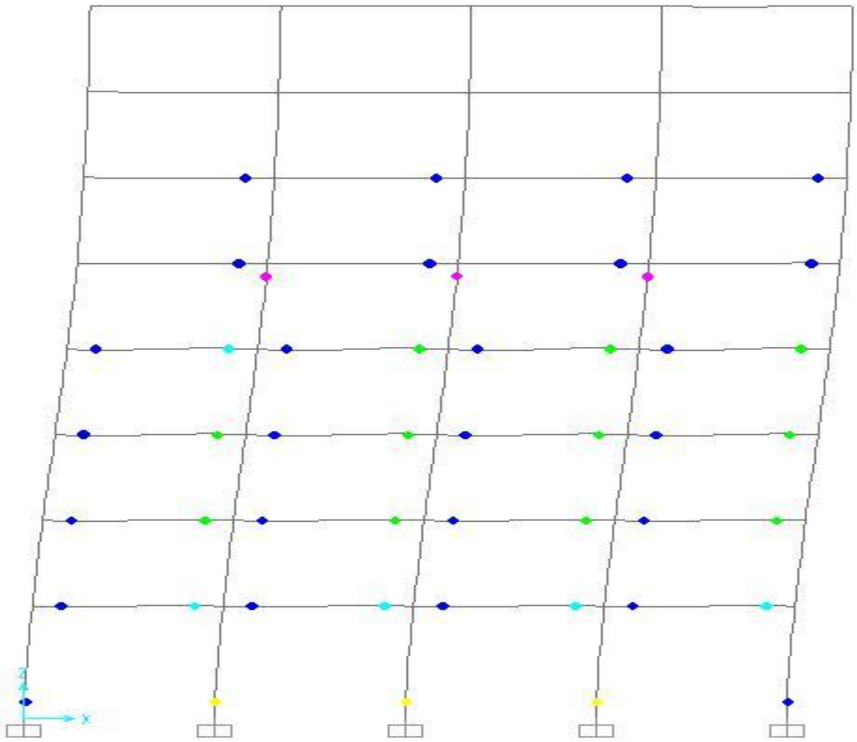


c) d)

Figure B-12 BRBFs of six storey: a) SXBF-case-1-6, b) SXBF-case-2-6, c) SXBF-case-3-6, and d) SXBF-case-4-6



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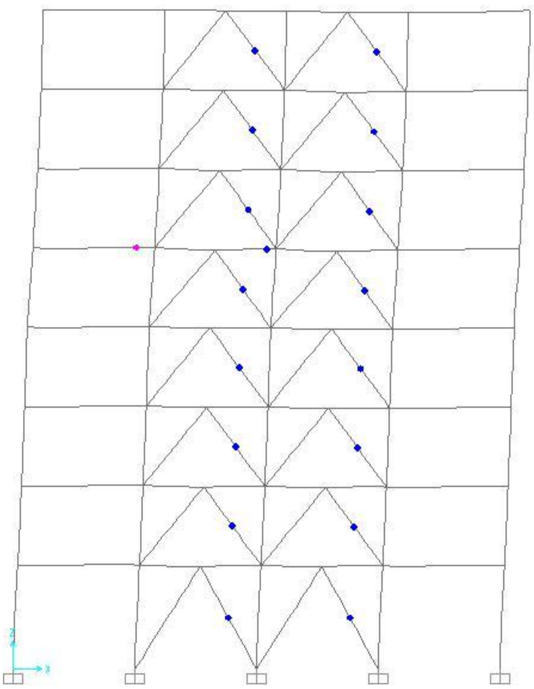

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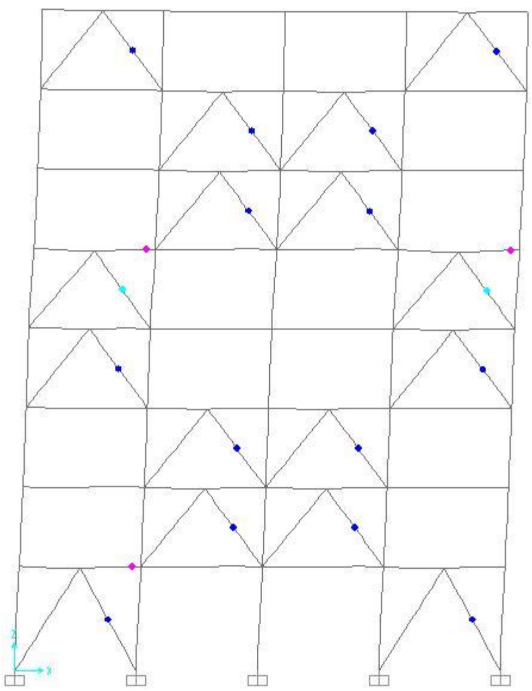

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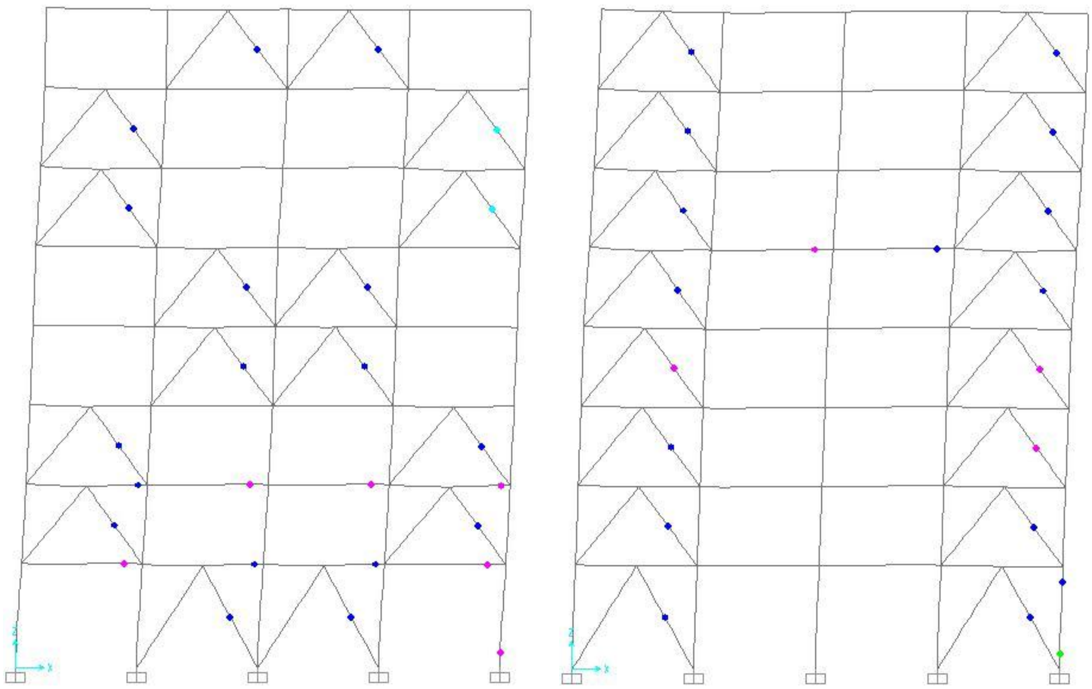
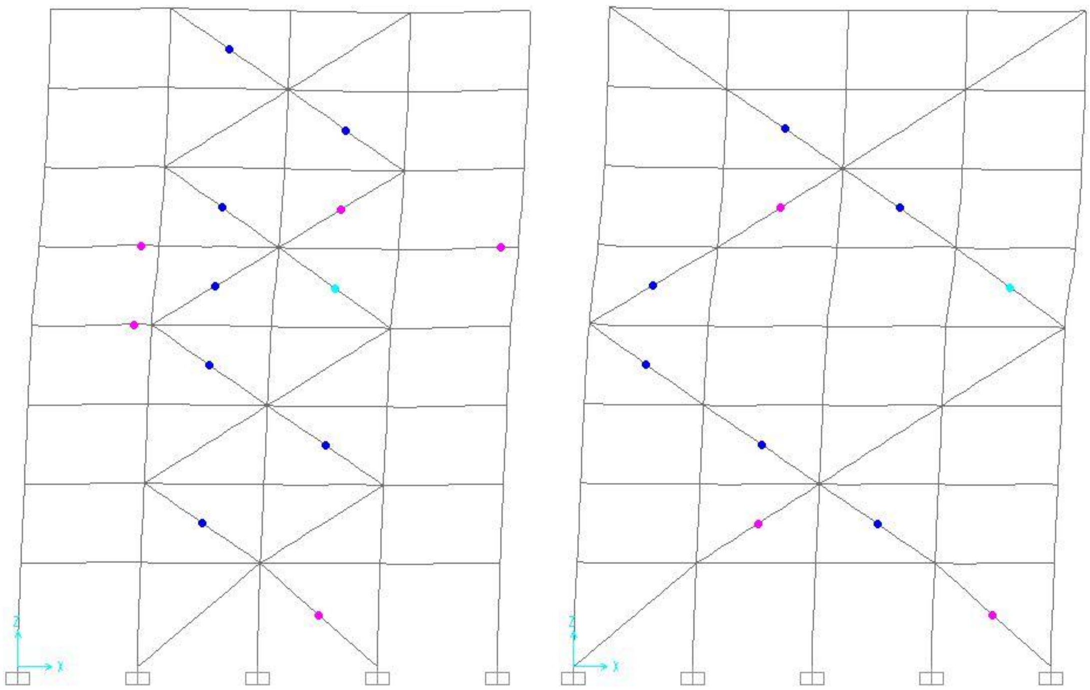
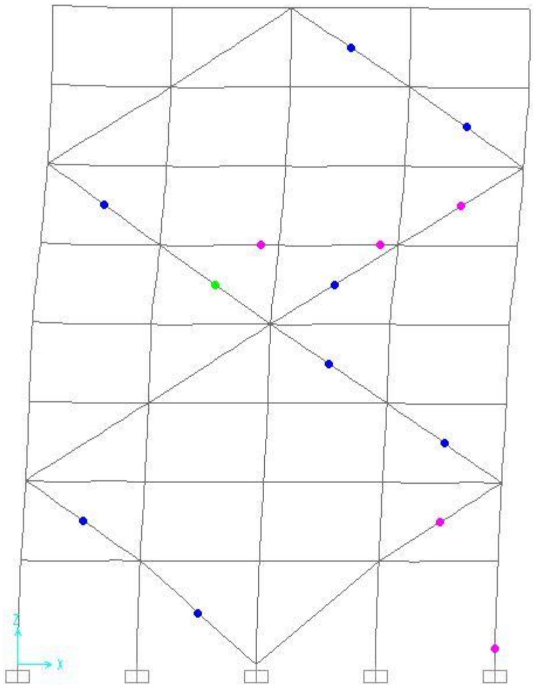


d) e)

Figure B-13 OF and BRBFs of eight storey: a) OF, b) CHBF-case-1-8, c) CHBF-case-2-8, d) CHBF-case-3-8, and e) CHBF-case-4-8



a) b)



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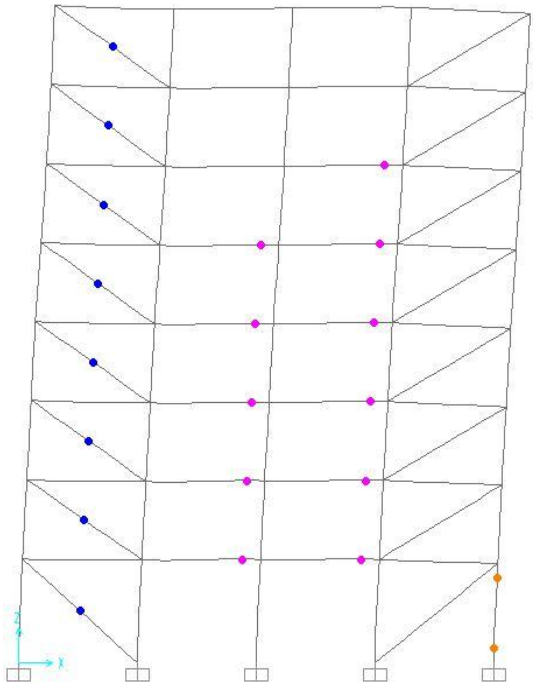
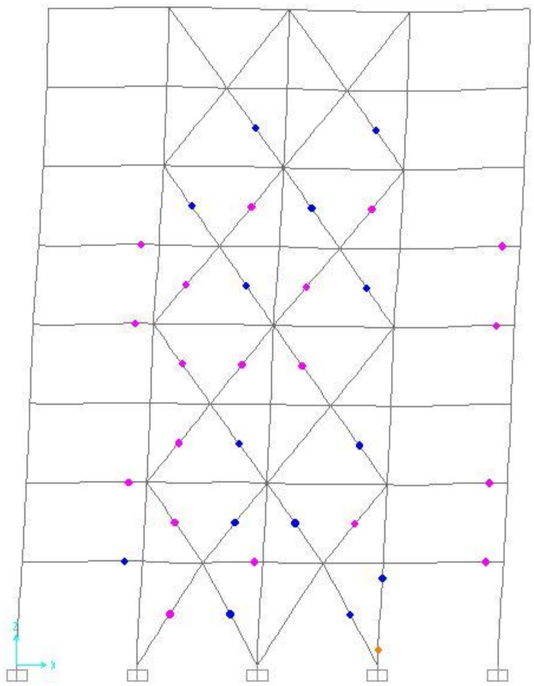
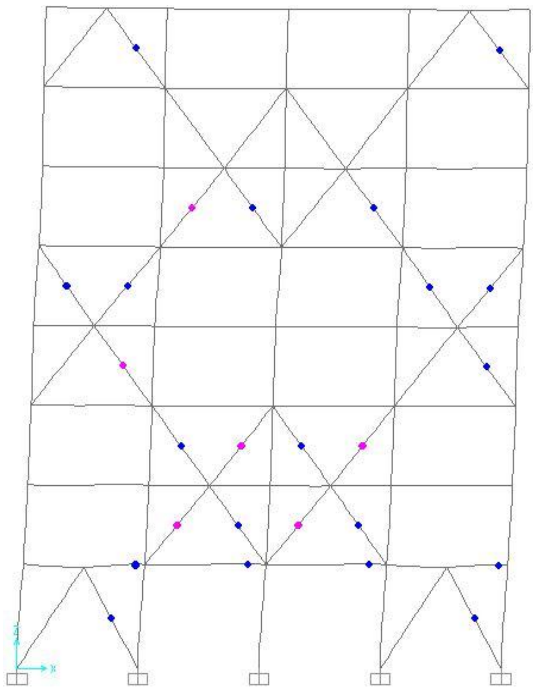


d)

Figure B-14 BRBFs of eight storey: a) DBF-case-1-8, b) DBF-case-2-8, c) DBF-case-3-8, and d) DBF-case-4-8



a)



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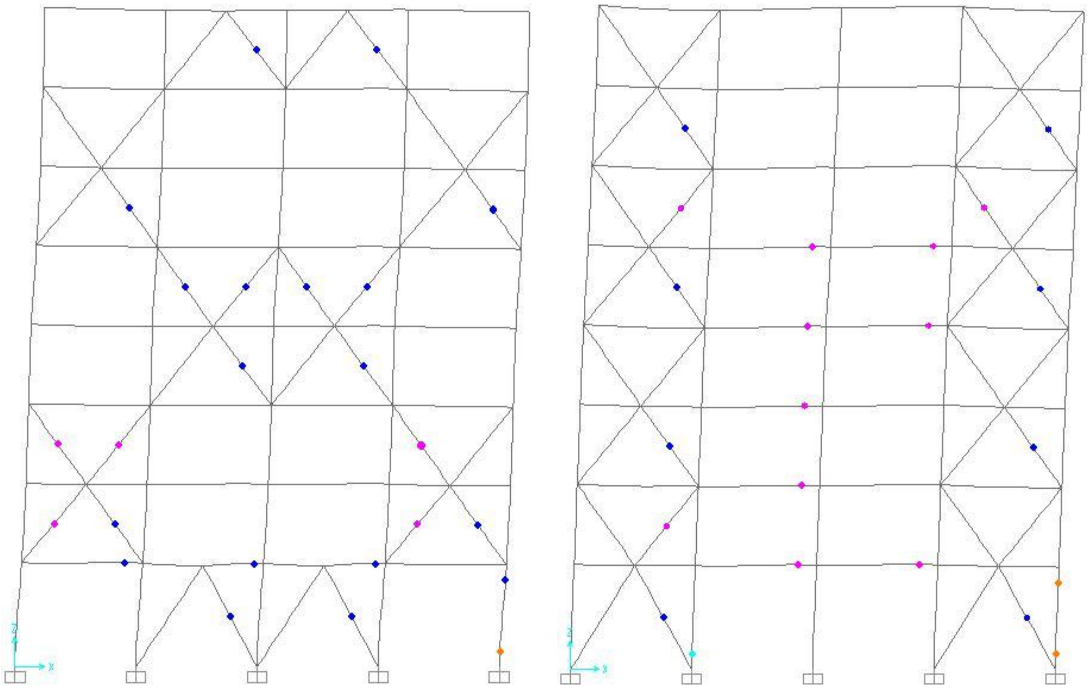
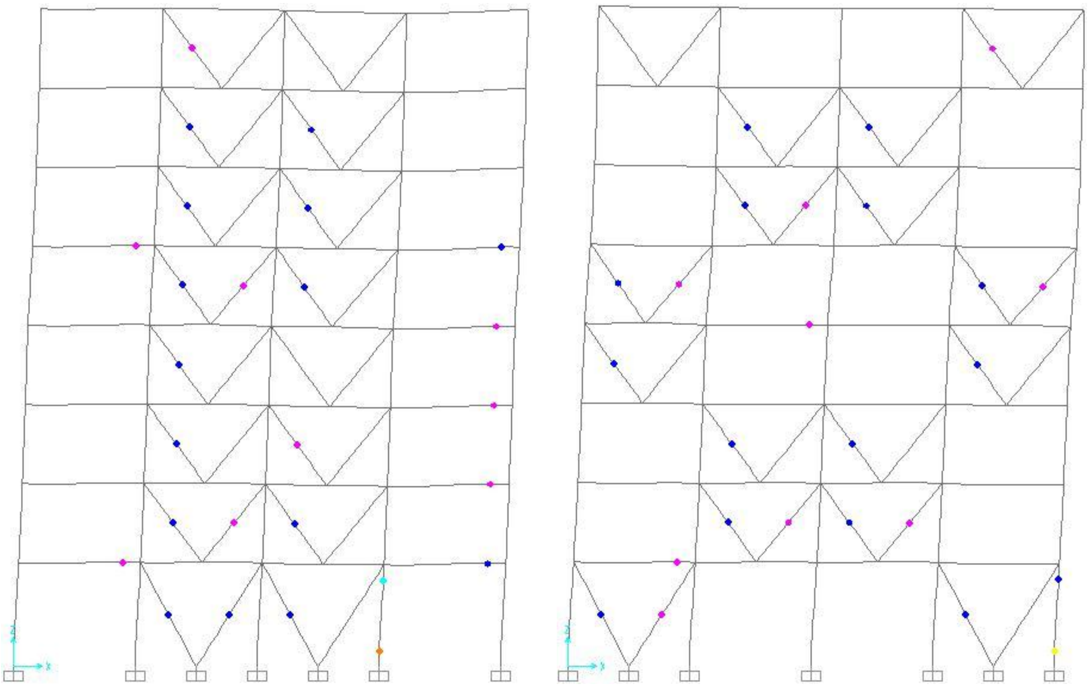
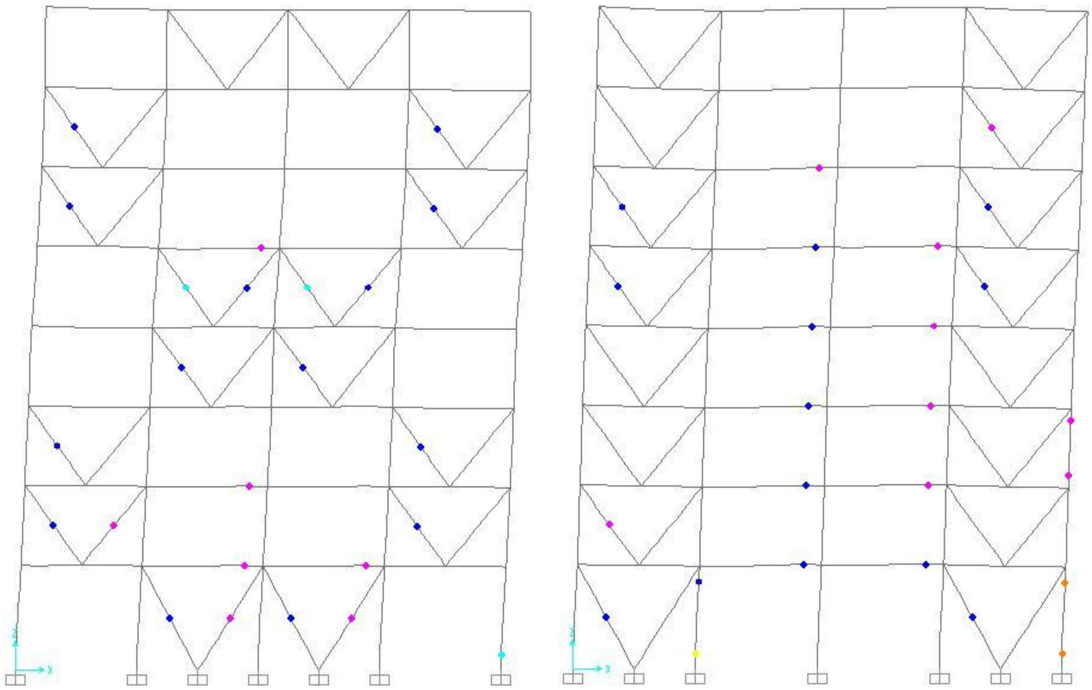


c) d)

Figure B-15 BRBFs of eight storey: a) VBF-case-1-8 b) VBF-case-2-8 c) VBF-case-3-8, and d) VBF-case-4-8



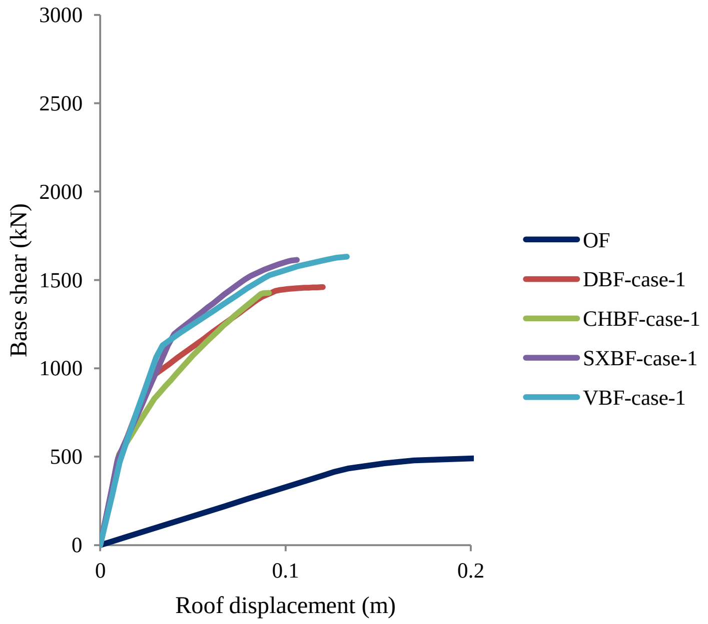
a) b)



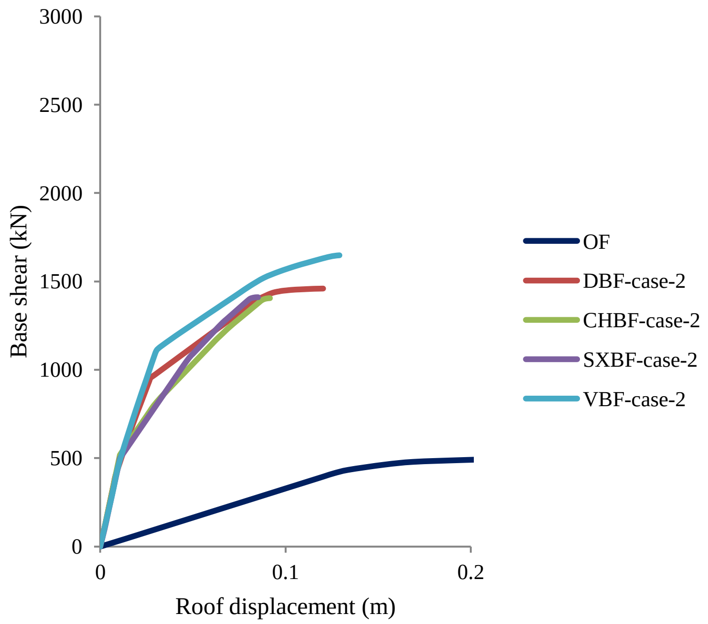
c) d)

Figure B-16 BRBFs of eight storey: a) SXBF-case-1-8, b) SXBF-case-2-8, c) SXBF-case-3-8, and d) SXBF-case-4-8

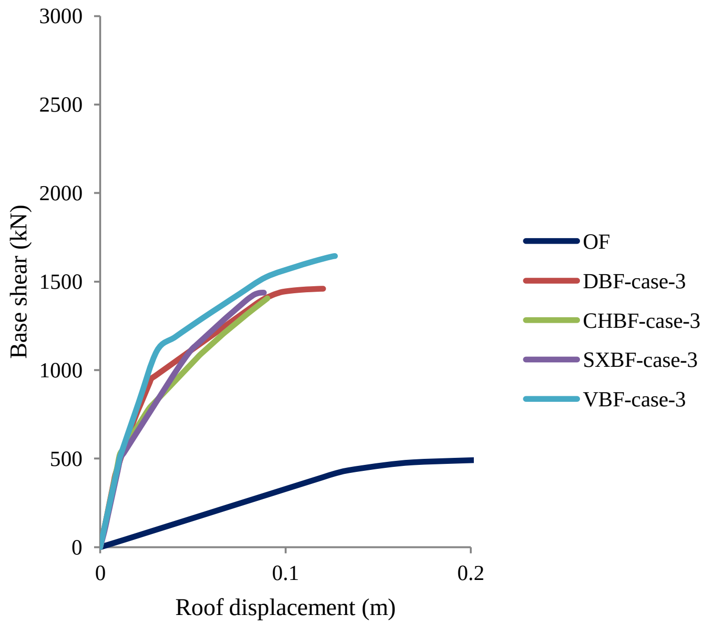
Appendix C: Pushover curves for BRBFs and OFs



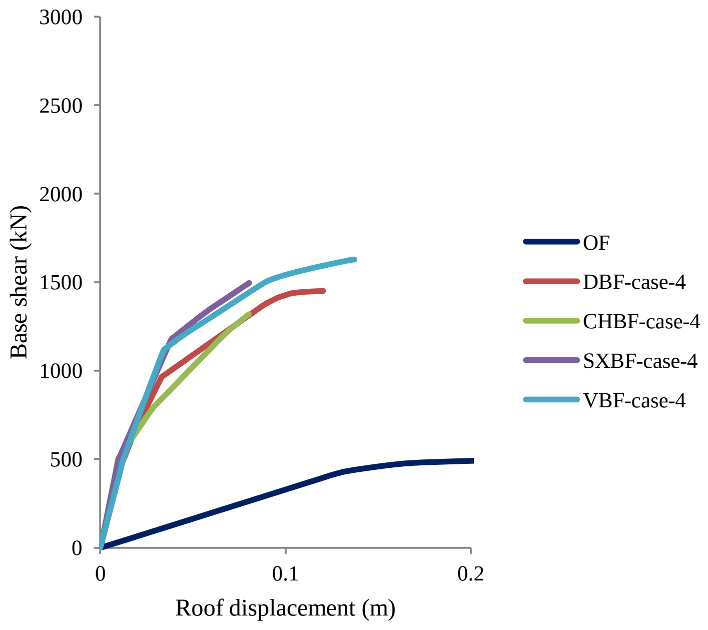
a)



b)

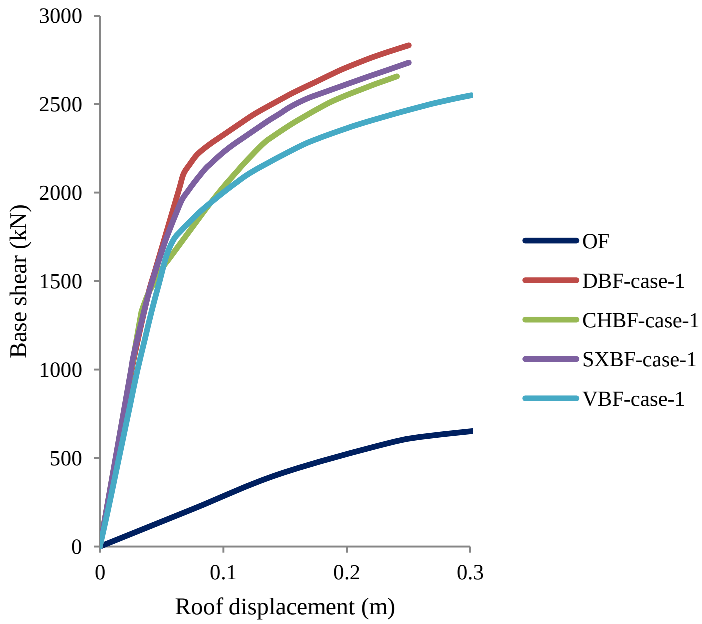


c)

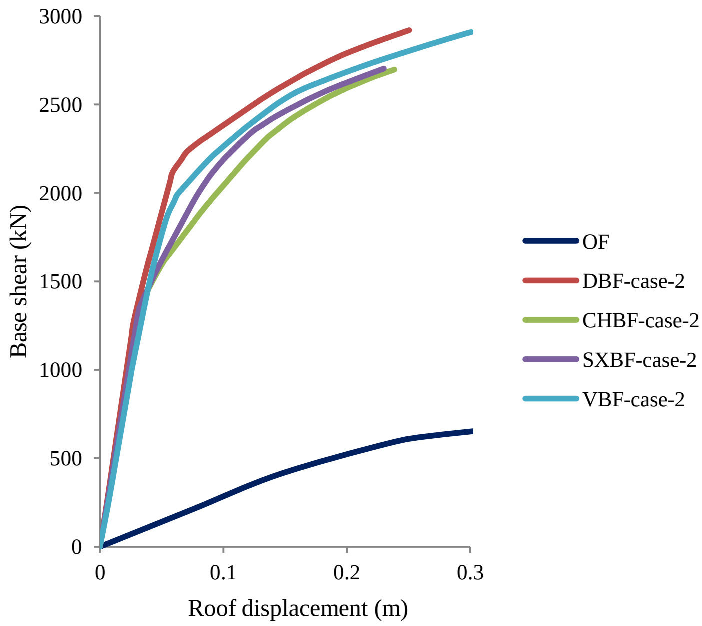


d)

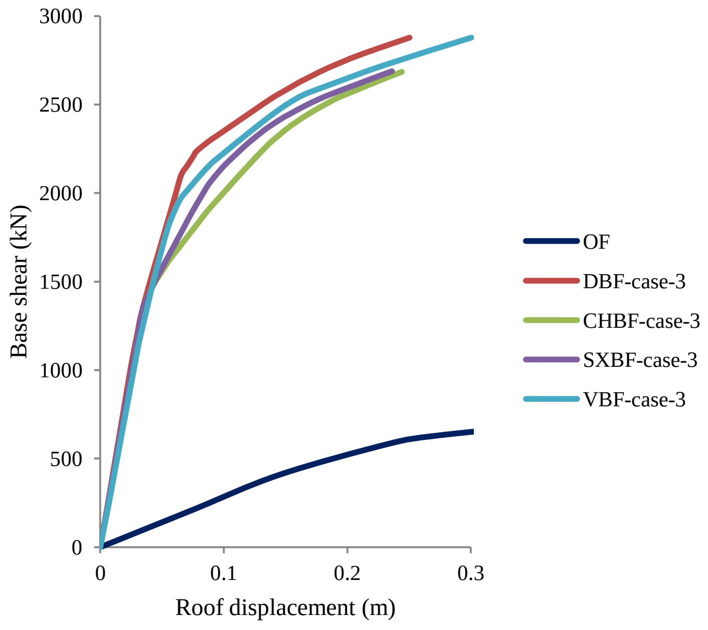
Figure C-1 Capacity curve for 2-storey OFs and BRBFs with different brace configuration and distribution: a) case-1, b) case-2, c) case-3, and d) case-4


3000
2500
2000
1500
1000
500
0
Base shear (kN)
0
0.1
0.2
0.3
Roof displacement (m)
OF
DBF-case-1
CHBF-case-1
SXBF-case-1
VBF-case-1

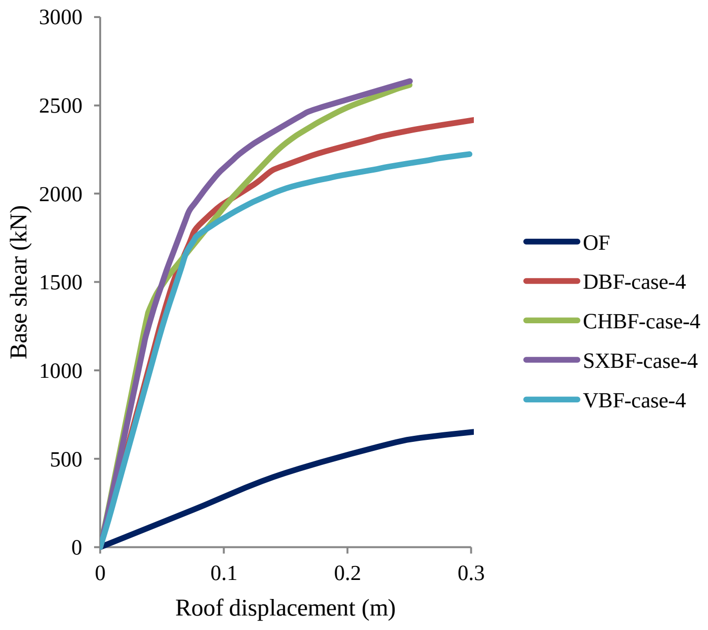
a)


3000
2500
2000
1500
1000
500
0
Base shear (kN)
0
0.1
0.2
0.3
Roof displacement (m)
OF
DBF-case-2
CHBF-case-2
SXBF-case-2
VBF-case-2

b)

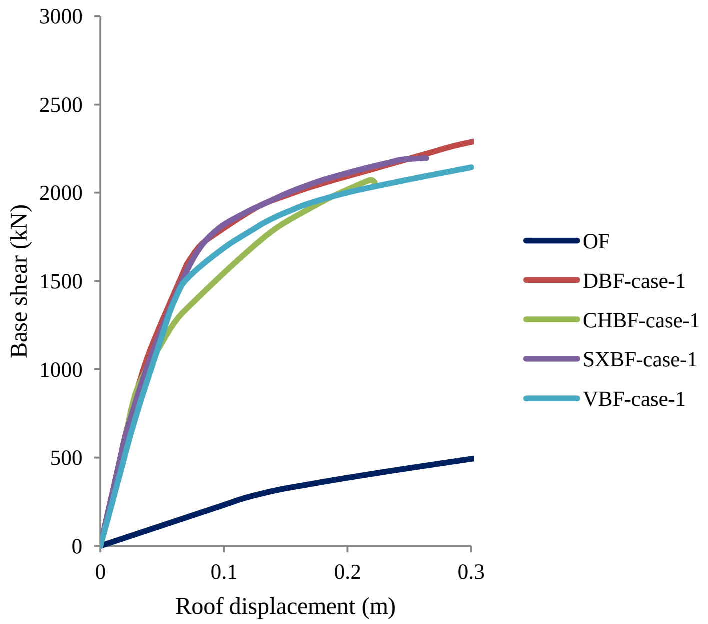


c)

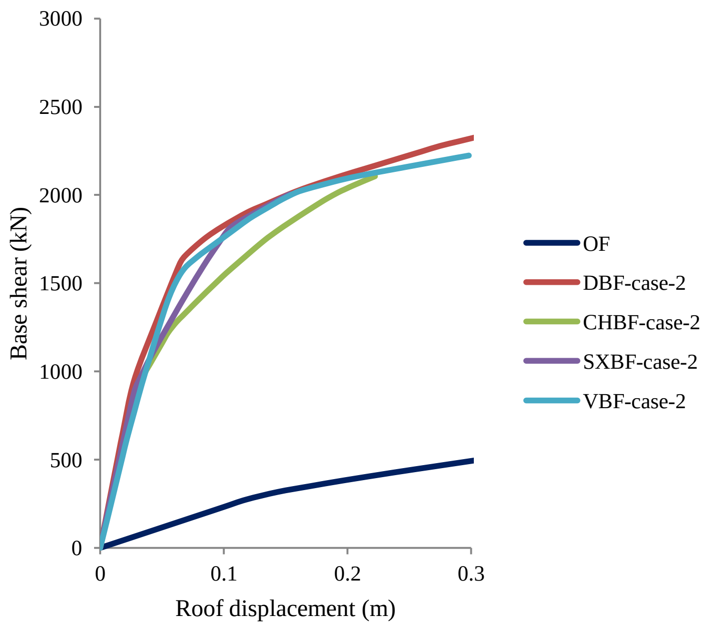


d)

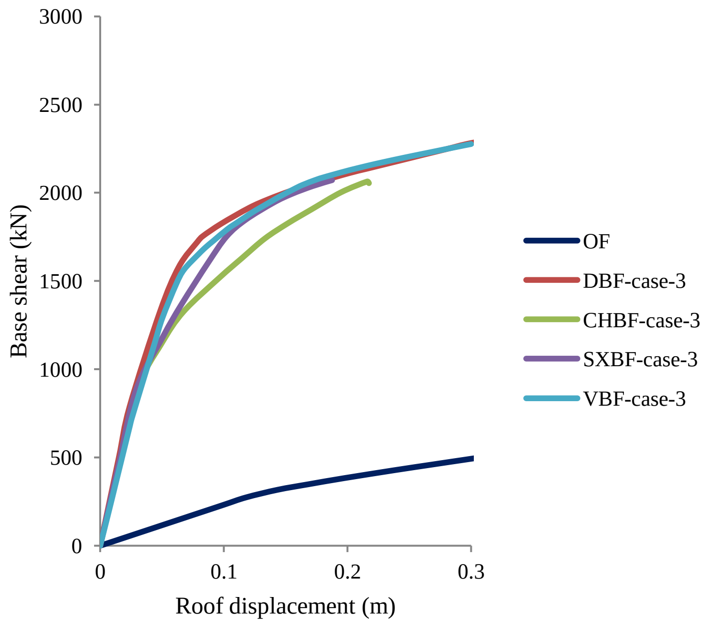
Figure C-2 Capacity curve for 4-storey OFs and BRBFs with different configuration and distribution: a) case-1, b) case-2, c) case-3, and d) case-4


3000
2500
2000
1500
1000
500
0
Base shear (kN)
0
0.1
0.2
0.3
Roof displacement (m)
OF
DBF-case-1
CHBF-case-1
SXBF-case-1
VBF-case-1

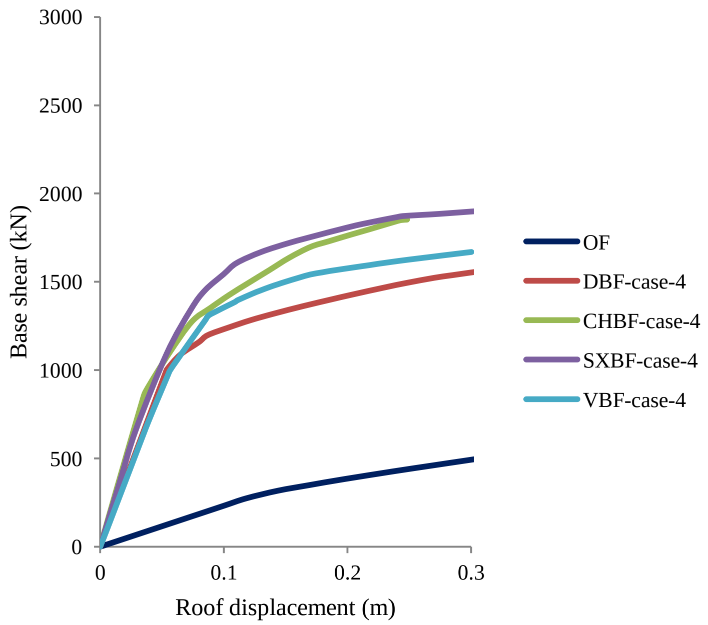
a)


3000
2500
2000
1500
1000
500
0
Base shear (kN)
0
0.1
0.2
0.3
Roof displacement (m)
OF
DBF-case-2
CHBF-case-2
SXBF-case-2
VBF-case-2

b)

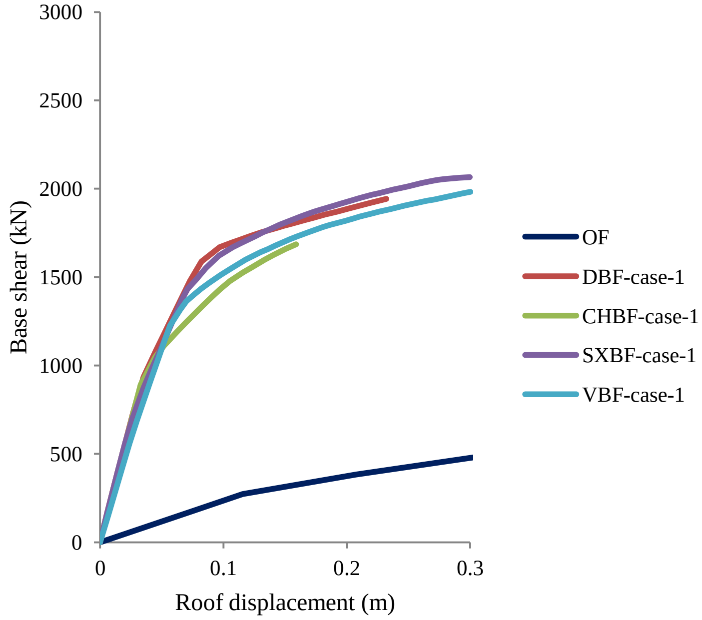


c)

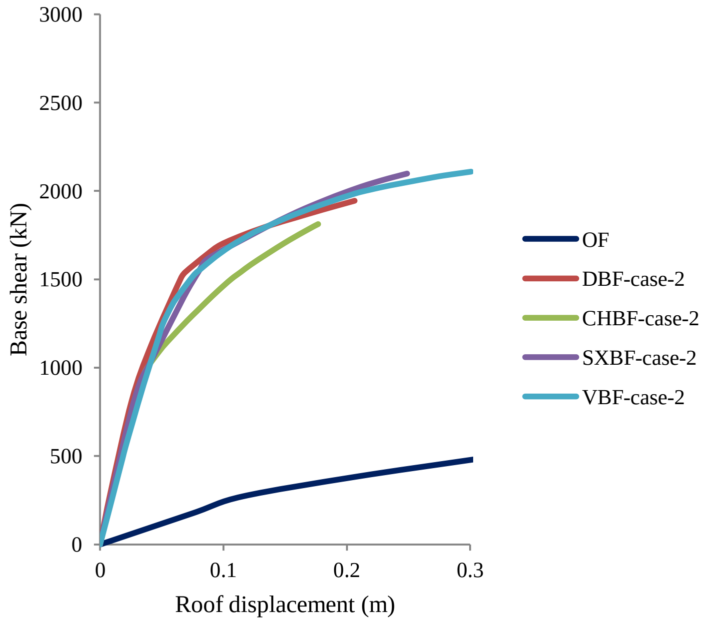


d)

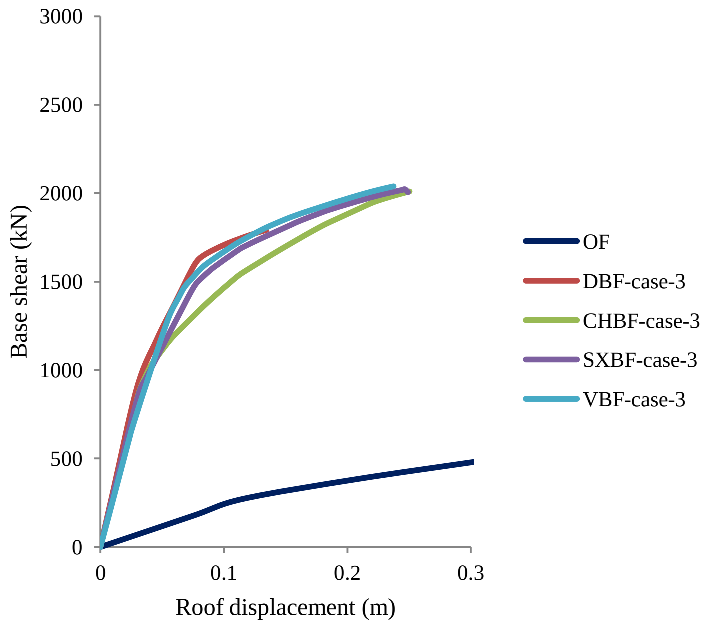
Figure C-3 Capacity curve for 6-storey OFs and BRBFs with different configuration and distribution: a) case-1, b) case-2, c) case-3, and d) case-4


3000
2500
2000
1500
1000
500
0
Base shear (kN)
0
0.1
0.2
0.3
Roof displacement (m)
OF
DBF-case-1
CHBF-case-1
SXBF-case-1
VBF-case-1

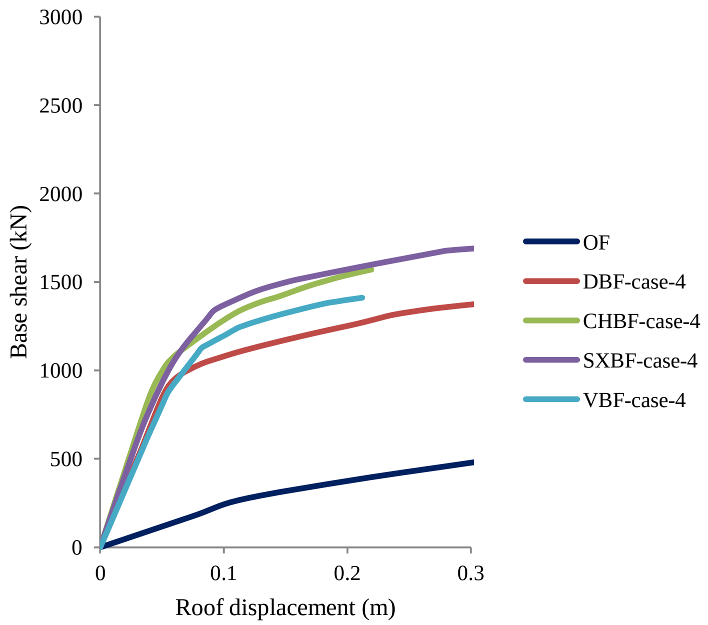
a)


3000
2500
2000
1500
1000
500
0
Base shear (kN)
0
0.1
0.2
0.3
Roof displacement (m)
OF
DBF-case-2
CHBF-case-2
SXBF-case-2
VBF-case-2

b)



c)



d)

Figure C-4 Capacity curve for 8-storey OFs and BRBFs with different configuration and distribution: a) case-1, b) case-2, c) case-3, and d) case-4

Appendix D: Deflected shape of the frames

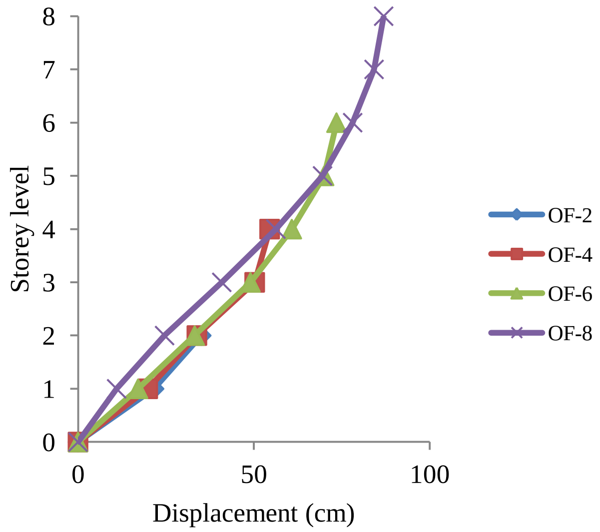
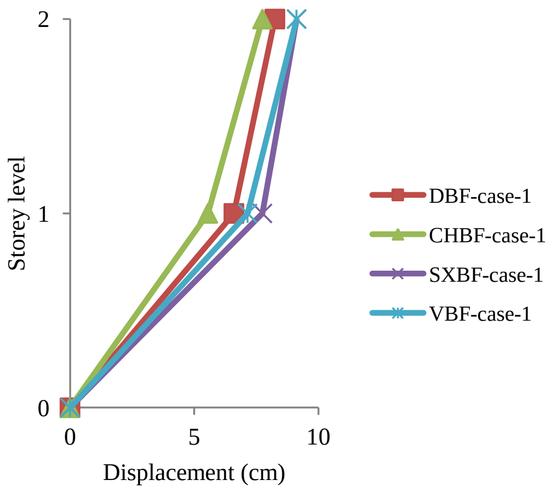
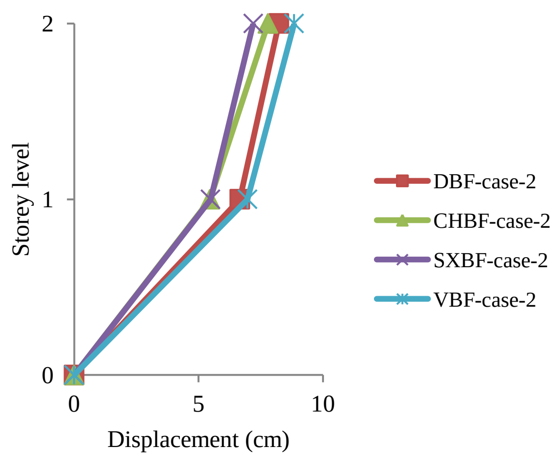


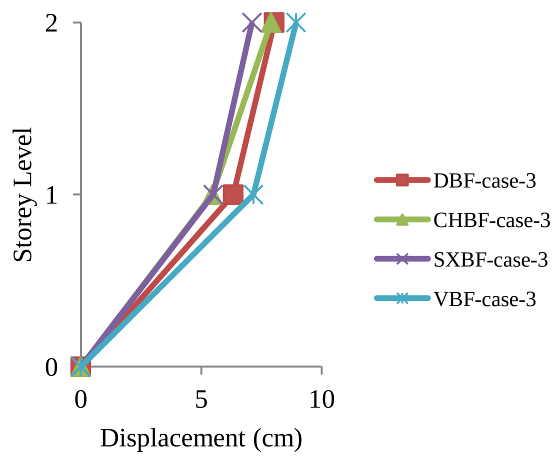
Figure D-1. Deflected shape of different OFs



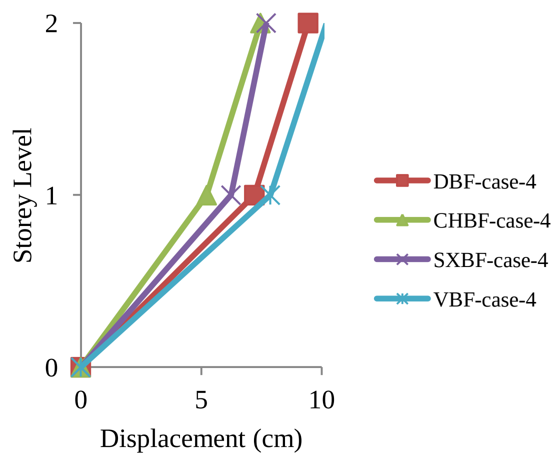
a)



b)

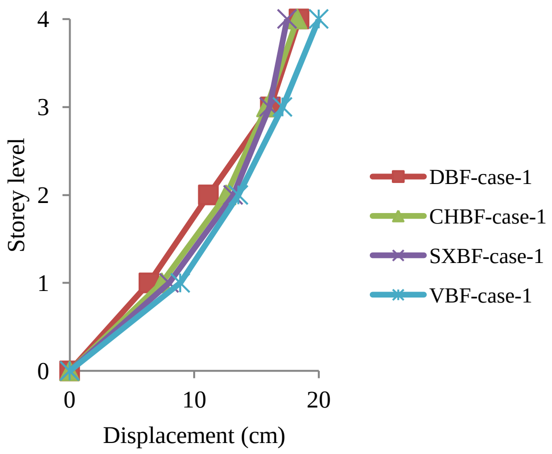


c)



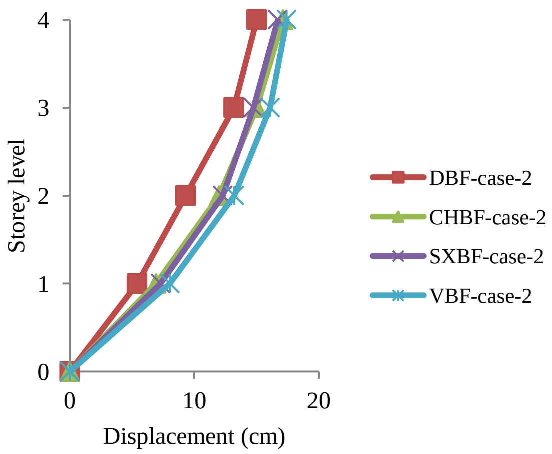
d)

Figure D-2 Deflected shape of 2-storey BRBFs with: a) case-1, b) case-2, c) case-3, and d) case-4



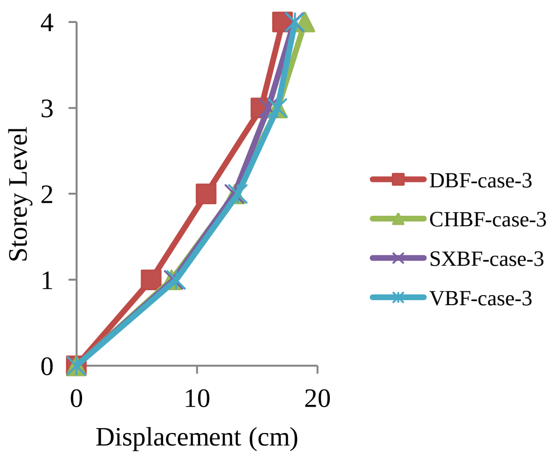
Storey level
Displacement (cm)
DBF-case-1
CHBF-case-1
SXBF-case-1
VBF-case-1

a)



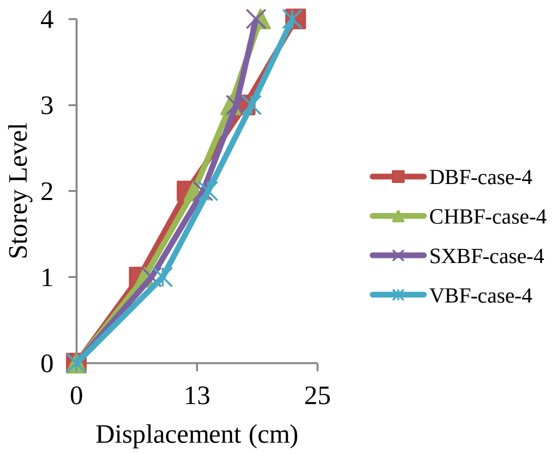
Storey level
Displacement (cm)
DBF-case-2
CHBF-case-2
SXBF-case-2
VBF-case-2

b)



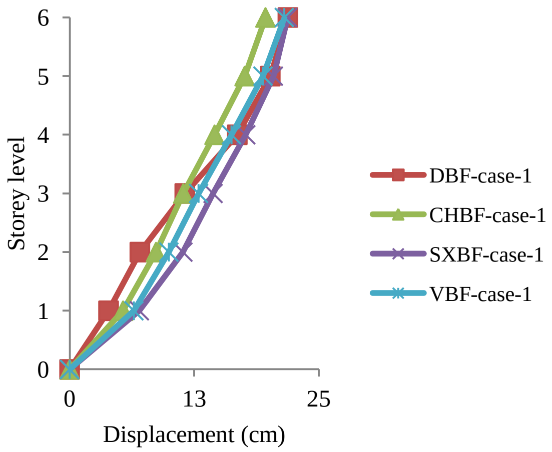
Storey Level
Displacement (cm)
DBF-case-3
CHBF-case-3
SXBF-case-3
VBF-case-3

c)

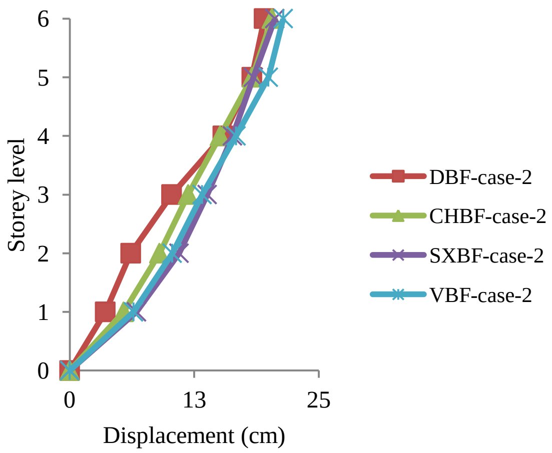


d)

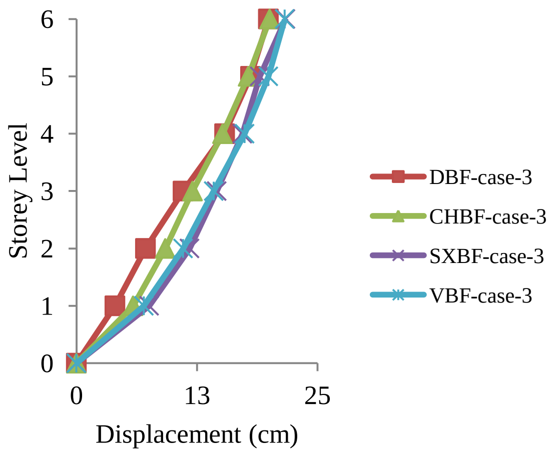
Figure D-3 Deflected shape of 4-storey BRBFs with: a) case-1, b) case-2, c) case-3, and d) case-4



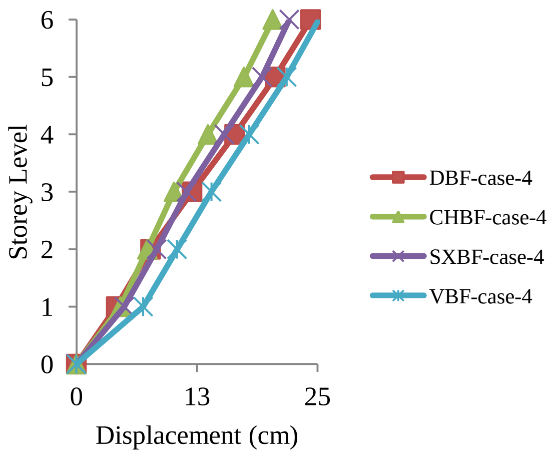
a)



b)

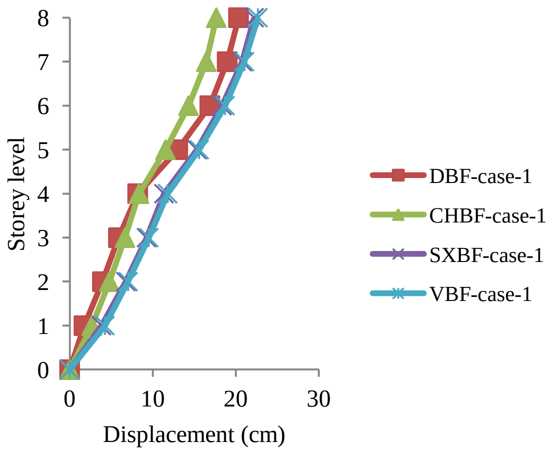


c)

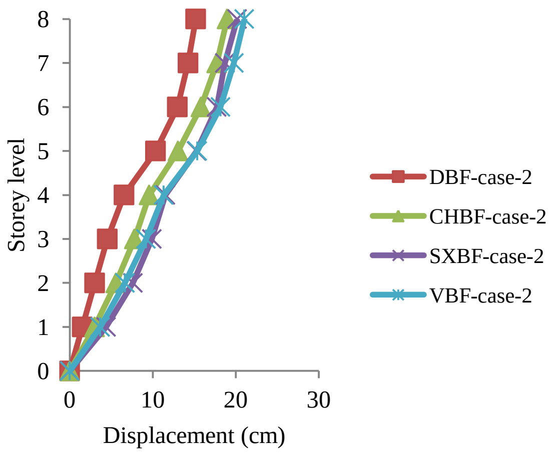


d)

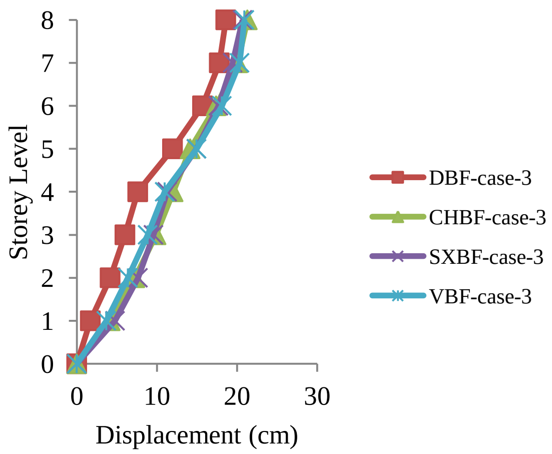
Figure D-4 Deflected shape of 6-storey BRBFs with: a) case-1, b) case-2, c) case-3, and d) case-4



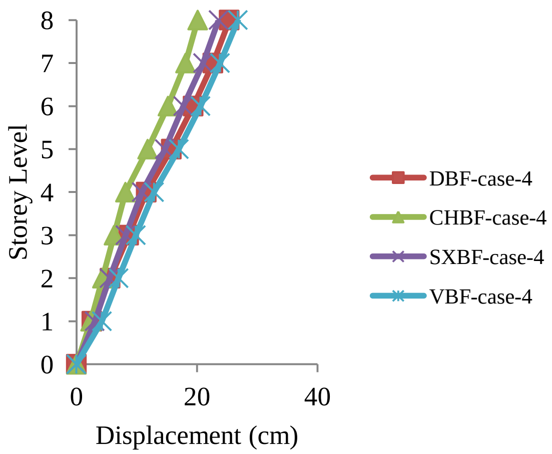
a)



b)



c)



d)

Figure D-5 Deflected shape of 8-storey BRBFs with: a) case-1, b) case-2, c) case-3, and d) case-4