UNIVERSITY OF GAZİANTEP
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EARTHQUAKE RESISTANCE OF HIGH RISE REINFORCED CONCRETE STRUCTURES WITH BUCKLING RESTRAINED BRACES

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IN
CIVIL ENGINEERING

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ALİ İHSAN ÖZÇETİN
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Earthquake Resistance of High Rise Reinforced Concrete Structures with Buckling Restrained Braces

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Ali İhsan ÖZÇETİN

ABSTRACT

Earthquake Resistance of High Rise Reinforced Concrete Structures with Buckling Restrained Braces

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Buckling restrained brace is relatively new type of structural bracing system and includes a steel core confined within a steel hollow section by a grout infill material. This bracing system utilizes the ductility of the steel more effectively than conventional one. In this study, the earthquake response of the high-rise reinforced concrete (RC) frame structures with buckling-restrained braces were investigated. For this, two RC structures having 9 and 18 stories were taken into account as a case study. The effect of using buckling-restrained braces to protect the structures against earthquakes was studied. As a bracing pattern, diagonal shape buckling-restrained braces were utilized over the height of the RC frames. The nonlinear dynamic time-history analysis was performed on the RC frame structures before and after inclusion of buckling-restrained braces. Different ground motion records, namely, 1999 Chi Chi, 1999, Hector Mine, and 1999 Düzce, were used in the analysis. It was concluded that the use of buckling-restrained braces over the frame height was found to have a significant effect on the seismic behavior of the structures. Buckling-restrained bracing cases exhibited lower displacement and more uniform inter-story drift demands compared to the cases without buckling restrained braces.

Keywords: Buckling restrained brace, Earthquake, Nonlinear analysis, Reinforced concrete structure, Seismic upgrading.

ÖZ

Burkulması Önlenmiş Çaprazlı Yüksek Katlı Betonarme Yapıların Depreme Karşı Dayanımı

ÖZÇETİN, Ali İhsan
İnşaat Mühendisliği Yüksek Lisans
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Burulması önlenmiş çapraz yapısal çapraz sisteminin oldukça yeni bir türü olup, çelik çekirdek eleman, çekirdek elemanı çevreleyen çelik tüp ve dolgu malzemesinden oluşmaktadır. Bu çapraz sistem çelik malzemenin süneklilığinden geleneksel sistemlere göre daha etkin olarak yararlanır. Bu çalışmada, burulması önlenmiş çapraz içeren çok katlı betonarme çerçeve yapıların deprem davranışı incelenmiştir. Bu amaçla, vaka çalışması olarak 9 ve 18 katlı betonarme yapılar irdelenmiştir. Burulması önlenmiş çaprazların deprem etkisine karşı yapıları korumaya yönelik etkisi araştırılmıştır. Çapraz modeli olarak, diyagonal tip burulması önlenmiş çapraz betonarme çerçeve yapının yüksekliği boyunca kullanılmıştır. Zaman tanım alanında doğrusal olmayan dinamik analizler burulması önlenmiş çaprazlı ve çaprazsız yapılarda gerçekleştirilmiştir. Yapının analizinde 1999 Chi Chi, 1999 Hector Mine ve 1999 Düzce deprem ivmesi kayıtlarından yararlanılmıştır. Sonuç olarak, burulması önlenmiş çaprazların yapı yüksekliği boyunca kullanıldığında yapının deprem performasına belirgin katkısı olduğu gözlemlenmiştir. Burulması önlenmiş çaprazların kullanıldığı durumlarda yapının daha düşük yerdeğiştirme ve daha düzenli göreli kat ötelenme dağılımına sahip olduğu tespit edilmiştir.

Anahtar kelimeler: Burulması önlenmiş çapraz, Deprem, Doğrusal olmayan analiz, Betonarme yapı, Deprem güçlendirmesi.

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LIST OF SYMBOLS/ABBREVIATIONS

ω_i	Natural frequency of the i^{th} mode
ω_j	Natural frequency of the j^{th} mode
Ω_0	Overstrength factor
AISC	American Institute of Steel Construction
AIJ	Architectural Institute of Japan
BF	Braced frame
BRB	Buckling restrained brace
BRBF	Buckling restrained braced frame
CBF	Concentrically braced frame
CHBF	Chevron braced frame
D	Roof displacement
DBF	Diagonal braced frame
E	Modulus of elasticity
EBF	Eccentrically braced frame
FEMA	Federal emergency management agency
h	Story height
H	Total height of the building

HSS	Hollow square section
L	Brace clear span
LVDT	Linear variable differential transformer
MRF	Moment resisting frame
NBCC	National Building Code of Canada
OCBF	Ordinary concentrically braced frame
OF	Original frame
P	Axial force
P_{cr}	Critical buckling load
P_e	Elastic buckling strength of the steel casing
P_y	Yield strength of the restrained yielding segment
R	Response modification factor
RC	Reinforced concrete
S_a	Response spectrum acceleration
SCBF	Special concentrically braced frame
SEAOC	Structural engineering association of California
T	Period
UB	Unbonded brace
UF	Un-braced frame
λ	Slenderness ratio
σ_y	Yield strength

β	Stiffness proportional damping coefficient
ζ	Damping ratio (%)

CHAPTER 1

INTRODUCTION

1.1 General

Earthquake action has been huge threat for the humanbeings from the primeval era until today. In the earthquakes up to now, not only many people lost their lives, but also it has had deep impacts to the countries economy. These unfavourable cases have motivated the structural engineers to spend much more efforts on such subjects. As a result, several latest developments in 20th century have enabled the structural engineers to approach one step closer to designing more efficient earthquake-resistant buildings.

Steel structures have been broadly used in seismic regions due to their proper mass-to-stiffness ratio, ductility, and thus improved energy absorption capacity. The typical steel frame configurations, i.e., moment resisting frame (MRF), concentrically braced frame (CBF), and eccentrically braced frame (EBF), show particular behavior with respect to ductility, stiffness and strength. Traditional lateral force resisting systems such as the moment resisting frame and concentrically braced frame that have been employed for many years at the construction sector. Steel moment-resisting frames have been regarded as an exceptional structural system for the regions prone to the earthquake. However, unexpected and widespread occurance of fracture at the beam to column connections during the 1994 Northridge, 1995 Hyogo-ken Nanbu, and other recent earthquakes have disturbed many structural enginneers logically due to rely on such system. Significant developments in design guidelines during late 1990s achieved some confidence. However, it is concluded that these systems involve intensive effort during the design and construction to manage ductile connection behavior. The study conducted on these systems showed that severe earthquakes could result in very large interstory drifts, thus damage occured at structural and nonstructural elements (FEMA, 2000; AISC, 2002).

Then, some engineers have utilized concentrically and eccentrically braced frames; substantial demand has observed in the use of such braced frames. Unfortunately, the steel braced frame has showed inferior seismic performance. Heavy damage was observed in the braced frames following 1985 Mexico (Osteraas and Krawinkler, 1989), 1989 Loma Prieta (Kim and Goel, 1992), 1994 Northridge (Tremblay et al., 1995; Krawinkler et al., 1996) and Hyogo-ken Nanbu (AIJ 1995; Hisatoku 1995; Tremblay et al., 1996) earthquakes. Frequent damage occured in the frames in which the braces were designed to resist tension only where the connections were weaker compared to the braces connected to them. Due to buckling of the compression members and material softening caused by Bauschinger effect, the hysteretic behavior of the braced frames was unreliable (Uriz and Mahin, 2004).

Inefficient stiffness of a moment resisting frame as well as the limited ductility of a conventional braced frame motivated structural engineeers to make much more effort to discover new type of system called as buckling restrained braced frame. In the recent years, the usage of buckling restrained braced frame considerably increases since the energy dissipation capacity of the structure and decreases demand for inelastic deformation in main structural members are significantly improved (Kim and Seo, 2004).

The buckling restrained brace (BRB) mainly includes a core segment that carries the whole brace axial load, and a restraining mechanism that inhibits the core segment from buckling in compression, enabling it to fully yield in both tension and compression. De-bonding material is placed between the core segment and the restraining mechanism to enable to deform independently. Also, de-bonding material reduces shear transfer between the core segment and restraining mechanism (Uang et al., 2004).

The main scope of this study is to compare the seismic performance of the high-rise reinforced concrete (RC) structures equipped with and without diagonal BRBs and to prove the effectiveness of BRBs on upgrading the performance of seismically deficient structures. In this manner, the nonlinear dynamic analysis was performed on the structures having 9 and 18 stories. In the dynamic analysis, 1999 Hector Mine, 1999 Chi Chi, and 1999 Düzce earthquake records were used. The performance of the existing and retrofitted RC structures were evaluated in terms of displacement time history, interstory drift ratio, etc.

1.2 Outline of the thesis

Chapter 1-Introduction: The first chapter of the thesis includes the aim and scope of the study.

Chapter 2-Literature review: This chapter covers the historical background on the practical application, prior researches on the buckling restrained braces. Also, a brief explaination is made on the braced frames with different arrangaments, buckling phenomenon, and components of BRBs.

Chapter 3-Methodology: This chapter focusses on the description of frames that were analysed. In this regard, the frame properties and analysis parameters are explained. As well, the type of analysis method is described.

Chapter 4-Results and discussion: This chapter presents and discusses the results obtained from the nonlinear dynamic analysis for evaluating the structural performance of each frame system considered in this study in terms of displacement time history, interstory drift ratio, etc.

Chapter 5-Conclusions: This chapter provides the most appropriate conclusions of this work with a few remarkable aspects for future research.

CHAPTER 2

LITERATURE REVIEW

2.1 General Background

Lateral deformations on structural buildings have been great concern for the engineers. Conventional frames experience high levels of lateral deformation during the earthquake action. If this deformation is excessive, structural and nonstructural damages are obvious. To overcome such deformations, several types of elements and devices have been utilized in the frames. Diagonal members, called braces, have been used as additional structural members that increase stiffness and energy dissipation capacity and limit interstory drift so as to defend the structure against the damage. Conventional braces are constituted a single steel element, with a varying cross sections, that is designed to carry both compressive and tensile loads (Escudero, 2003).

2.2 Braced Frames

Bracing is a very effective method that is utilized to increase the global stiffness and strength of the composite frames (Di Sarno and Elnashai, 2002). Braced frames are advantageous because the lateral loads are resisted mainly by brace axial capacity with little or no bending in the member. Braced frame members are sized to ensure sufficient stiffness to satisfy code drift requirements and enough member strength to resist both the compressive and tensile axial forces. The total behavior of the brace under repeated stress is related to its hysteretic behavior. Brace behavior is mainly depending on unwanted member buckling phenomenon. Thus, during cyclic loading, the brace performs unsymetrical hysteretic behavior in compression and tension. The most frequently used braced frame systems include concentrically braced frame (CBFs), eccentrically braced frames (EBFs), and the knee-brace frames. The CBFs may defined as bracing system that is the brace centerline intersects at the intersection of the column and beam centerlines. The EBFs may defined as bracing system that is at least one end of every brace is joined so as to transmit the brace force either another brace or to a column through shear and bending in a beam portion called a link. In the knee-braced frames, the main diagonal braces are joined to short knee members which span diagonally across the beam-column joints (Popov and Engelhardt, 1988). Moreover, the typical brace configurations are illustrated in Figures 2.1 and 2.2.

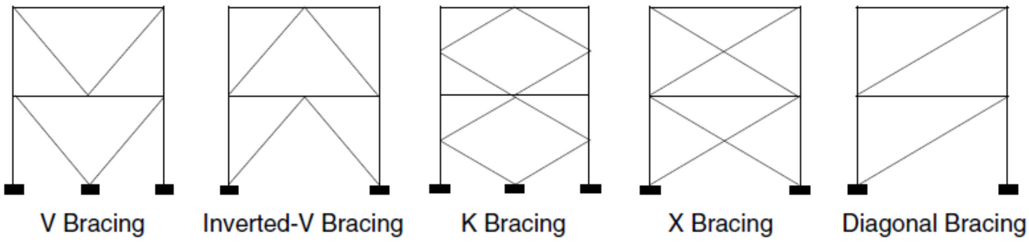


Figure 2.1 Typical bracing arrangements for CBFs (AISC, 2002)

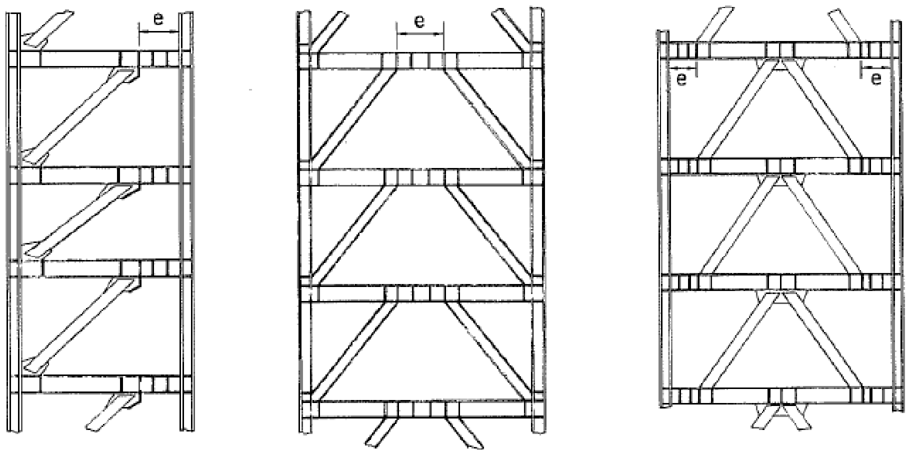


Figure 2.2 Typical bracing arrangements for EBFs (Popov and Engelhardt, 1988)

2.3 Buckling Phenomenon

The most common problem related to a compression member is flexural buckling. A failure mode in which the member deforms laterally and its stiffness and load bearing capacity decreases. As a result of this type of failure, lateral stiffness drops, frame stability decreases prominently. This causes severe damage in the structural and nonstructural elements. When a compressive loading is applied to the conventional braces, which have limited deformation ductility capacity, it exhibits unsymmetrical hysteretic cycles with a marked strength drop, as shown in Figure 2.3 (Xie, 2005).

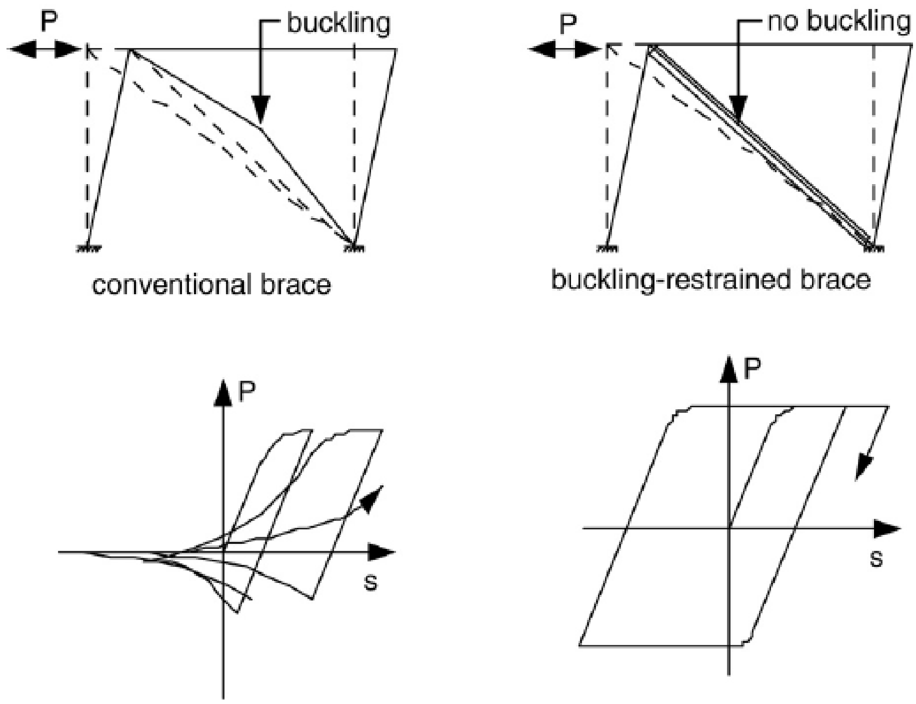


Figure 2.3 Behavior of conventional brace and BRB (Xie, 2005)

The global elastic buckling load for deformable long slender member is obtained by a classical Euler method. Moreover, the derivation of the critical buckling load (P_{cr}) is described in the following equations below. It is worthy to note that when the loading level reaches to P_{cr}, the structural member would experience the buckling. This method also assumes that (Tinker, 2011):

- The member is perfectly straight which means load eccentricity does not exist,
- Plane cross-sections remain plane after deformation,
- The material conforms to Hooke's Law, and
- The displacement of the member is small.

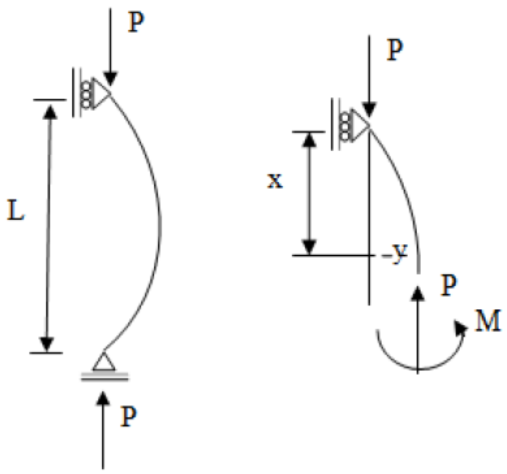


Figure 2.4 Euler's Buckling (Özkan, 2013)

i. Applying the equilibrium equation to Figure 2.4

$$M = -Py \tag{2.1}$$

ii. From the small deformations theory,

$$\left(\frac{M}{EI} = \frac{d^2y}{dx^2}\right) \tag{2.2}$$

iii. By putting Eqn. (2.2) into Eqn. (2.1), it would provide,

$$\left(\frac{d^2y}{dx^2}\right) = -\left(\frac{Py}{EI}\right)$$

$$\left(\frac{d^2y}{dx^2}\right) + \left(\frac{Py}{EI}\right) = 0 \tag{2.3}$$

iv. The general solution yields to,

$$y = A\ sin\sqrt{\frac{P}{EI}}x + B\ cos\sqrt{\frac{P}{EI}}x \tag{2.4}$$

v. From the boundary conditions $x = 0$ or $y = 0$,

$$y = A\ sin\sqrt{\frac{P}{EI}}x + B\ cos\sqrt{\frac{P}{EI}}x \tag{2.5}$$

From the boundary conditions at $x = L\ , y = 0$,

$$y = A\ sin\sqrt{\frac{P}{EI}}x + B\ cos\sqrt{\frac{P}{EI}}x \tag{2.6}$$

vi. By solving the above equations, the critical load can be obtained,

$$sin\sqrt{\frac{P}{EI}}x = 0 \rightarrow \sqrt{\frac{P}{EI}}L = \pi$$

$$P_{cr} = \frac{\pi^2 EI}{L^2} \tag{2.7}$$

2.4 Slenderness

Buckling is dominated by the slenderness ratio, λ, which is a ratio of the unsupported length of a member to the width of the face of the member under consideration. The cyclic behavior of a member subjected to compressive axial loading relies mainly on its slenderness. The slenderness ratio is a function of the brace end conditions (k), the brace length or the brace clear span (L), the second moment of area (moment of inertia) of the member about axis ii (I_{ii}), and the cross - sectional area of the member (A). The quantity $\sqrt{I_{ii}/A}$ is also defined as the radius of gyration (r_{ii}) about axis ii (Odabaşı, 1997).

$$\lambda_{ii} = kL\sqrt{\frac{A}{I_{ii}}} = \frac{kL}{r_{ii}} \tag{2.8}$$

2.5 Concept of Buckling Restrained Braces

The buckling restrained brace (BRB) is designed to overcome undesirable mode of failure by accomodating continuous lateral straint, which provides that the brace length of the core elements is effectively zero. Thus, the BRB system shows a balanced hysteresis loop as the cyclic load applied. This behavior is the trademark of the BRB system and is the particular feature when compared to other conventional braces in the sense of hysteretic response. The BRBs offer enhanced energy dissipation capacity, perfect ductility and nearly symmetrical hysteretic behavior in tension and compression. As a result of this, smaller unbalanced vertical brace forces generated, buckling restrained braced frames (BRBFs) also require smaller beam sections compared to conventional concentrically braced frames (CBFs) with chevron bracing configurations (Prasad, 2004). The BRB component can be imagined as the structural "fuse" which is part of vertical braced frame system. Simply, this "fuse" element serves itself like a stuntman during severe seismic loading. Thus, the rest of the structure is protected. In Figure 2.5, the mechanisms of the BRB are given (Sabelli and Lopez, 2004).

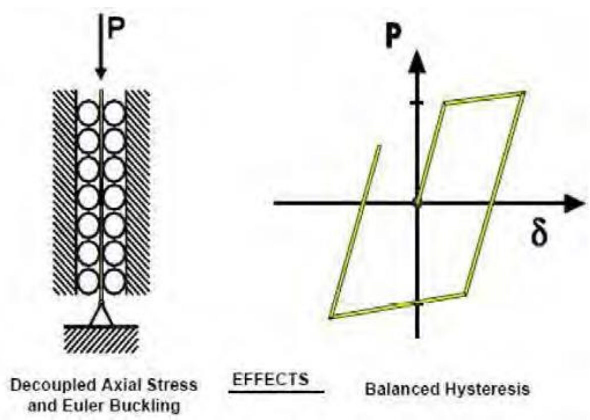


Figure 2.5 Mechanisms of the BRB (Sabelli and Lopez, 2004)

The BRB, or unbonded brace, consist of a ductile steel core that sustains the axial load of the brace and a restraining mechanism that ensures the flexural rigidity and stiffness to prevent the global buckling. The term unbonded brace derives from the need to ensure a slip surface or unbonding material layer between the steel core and surrounding concrete so that axial loads are carried only by steel core. Besides, a small gap between the steel core and the surrounding concrete has to be ensured due to Poisson effect, which causes the steel to expand under compression. The material and geometry selection have to be made carefully by considering Poisson and shearing effects in this slip layer. To permit relative movement between the steel core element and the surrounding concrete, the local buckling of the steel as it yields in compression would be taken into consideration (Sabelli et al., 2003; Kumar et al., 2007; Deulkar et al., 2010). Figure 2.6 also shows the schematic view of the typical BRB.

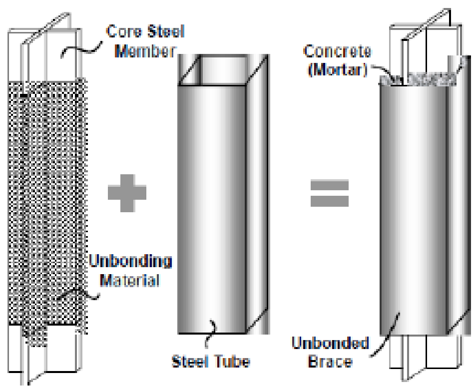


Figure 2.6 Schematic representation of BRB (Sabelli et al., 2003)

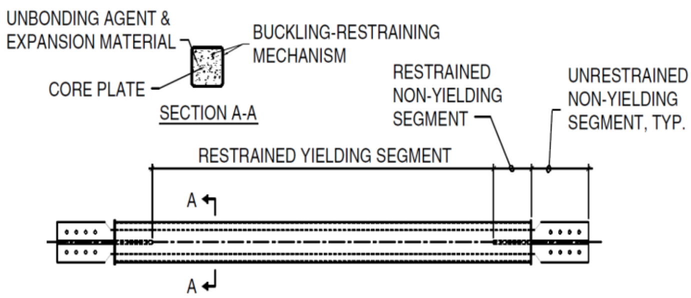


Figure 2.7 Components of BRB (Uang et al., 2004)

In the study of Uang et al. (2004), the components of BRB and their mechanism were also explained. Based on Figure 2.7, the following description are given:

- Restrained Yielding Segment: Steel grades which exhibit high ductility are preferable. Additionally, steel materials with a predictable yield strength with small variations are preferable,
- Restrained Non-Yielding Segment: This segment is generally an extension of the restrained yielding segment but with an enlarged area to provide elastic response,
- Unrestrained Non-Yielding Segment: This segment is frequently designed as a bolted connection, but design with other connection types such as a pin connection or a welded type are also possible,
- Unbonding Agent and Expansion Material: The inert material to be selected can effectively minimize or eliminate the transfer of shear force between the restrained steel segment and mortar, and
- Buckling Restrained Mechanism: This mechanism is typically insist of mortar and steel casing. When properly designed and detailed, steel casing should not resist any significant axial load.

To prevent buckling of BRBs, Watanabe et al. (1988) suggested that the steel casing would be designed for a sufficient flexural stiffness such that

$$\frac{p_e}{p_y} \geq 1.5 \tag{2.9}$$

$$P_e = \frac{\pi^2 E I_{sc}}{L_{sc}^2} \tag{2.10}$$

Where P_y is the yield strength of the restrained yielding segment, P_e is the elastic buckling strength of the steel casing, E is Young's modulus, I_{sc} is the moment of inertia of steel casing, and L_{sc} is the work point-to-work point brace length.

2.6 Previous Investigations on BRBs

The initial study was started by the researchers of Yoshino and Karino from Architectural Institute of Japan, in 1971. Yoshino and Karino introduced a sophisticated brace type in the form of "Shear Wall with Braces". This concept resulted from a yielding flat steel plate encased between the concrete wall panels, seperated by debonding materials, producing buckling restraining. It was also pointed out that a shear wall with spacing between the reinforced concrete wall and the steel plates displayed higher energy dissipation capacity compared to a shear wall with no spacing (Yoshino and Karino, 1971). The common configuration of their model and testing details are depicted in Figures 2.8 and 2.9, respectively.

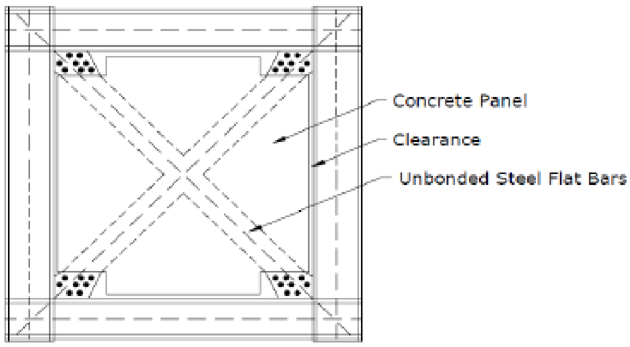


Figure 2.8 General configuration of the model (Yoshino and Karino, 1971)

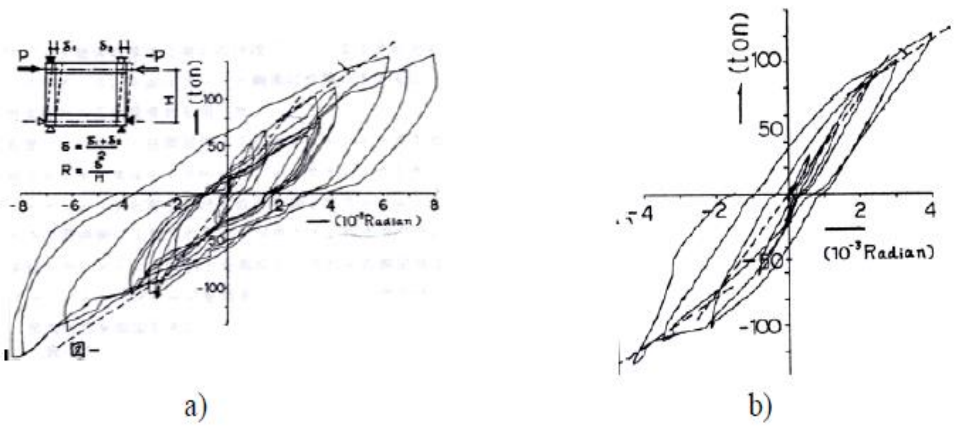


Figure 2.9 Response comparison between two specimens with internal clearance (left) and without it (right) (Yoshino and Karino, 1971)

Wakabayashi et al. (1973a; 1973b) investigated the combination of reinforced concrete panels and steel plates seperated by an unbonded layer. Their investigation included;

- For discovering the methods of debonding, they conducted the pull-out tests,
- For analysing required stiffness and strength of the precast panels in which the plates were sandwiched, they conducted the compression tests,
- For analyzing the performance of the end connection detail, they conducted the subassamblage tests, and
- For the check of the suggested frame system, they conducted the two story frame tests.

In the pull-out tests, epoxy resin, silicon resin, vinly tapes, etc. were tested for the usage purpose of debonding material. Totally eleven specimens were tested. It was inferred that the method of debonding of covering a layer of silicon resin on the top of a layer of epoxy resin proved the effectiveness with regarding constructability, debonding effect and durability. In the compression tests, twenty one specimens (details between the exposed and embedded parts (styrol foam, gaps)) were tested by adopting different reinforcing details. It was concluded that a special attention had to be paid on the reinforcement along the edges of the panels. Also it was declared to put small styrol foam in the gap and would be adequately sized to prevent restrain the stiffened ends from deformation in the precast concrete panels (Wakabayashi et al., 1973a). Figure 2.10 shows the testing details in their experimental study.

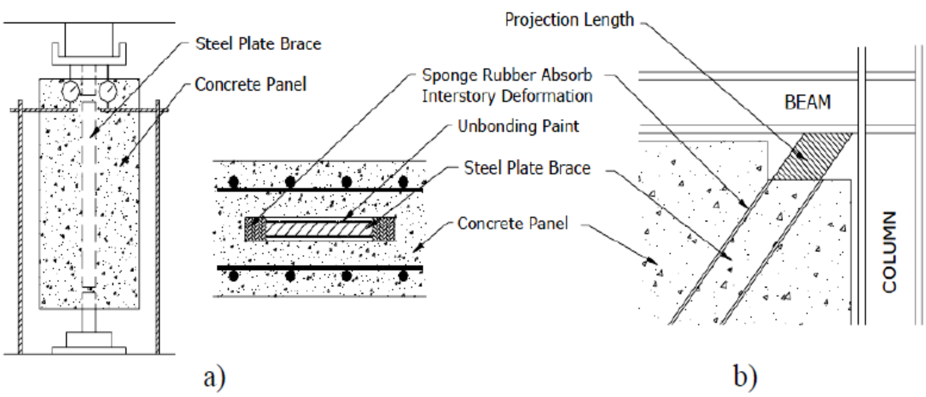


Figure 2.10 Views of a) monotonic test setup and b) gap disposition at the brace end (Wakabayashi et al., 1973a)

In the subassemblage test, fourteen x brace (in half scale) and diagonal brace frame systems with bonded and unbonded specimens were cyclically tested to evaluate the hysteretic behavior of the BRB. Figure 2.11 illustrates the test equipment and the hyteresis behavior for the diagonal pattern steel core braces encased in reinforced concrete panels considered in the study of Wakabayashi et al. (1973a; 1973b). From the test results, they concluded that unbonded braces exhibited superior performance compared to bonded braces.

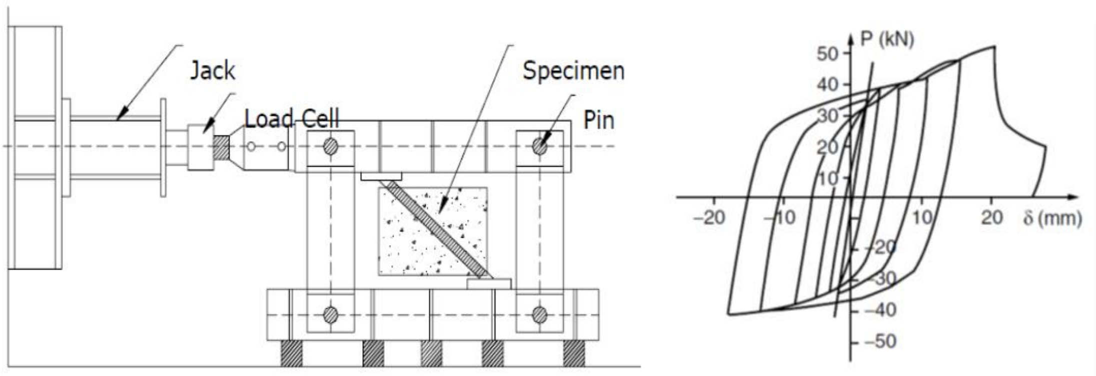


Figure 2.11 Cycling loading test setup (left) and hysteretic behavior in one of the tested specimens (right) (Wakabayashi et al., 1973a; Wakabayashi et al., 1973b)

In the system verification test conducted in the study of Wakabayashi et al. (1973a; 1973b), 2 two-story and 2 two-bay frames (in half scale) were examined. One with braces configurated diagonally and the other with braces configurated in a chevron type. They concluded that the steel braces exhibited good energy dissipation capacity. Also, the behavior of the frames and hysteretic loops were declared to be stable until local buckling phenomenon which corresponded at a lateral angle of drift of 0.025 radian. In the elastic phase of the brace, the crack occured in the concrete panel at a drift angle of 0.001 radian. The first crack along the axis of brace was observed. Moreover, it was found that at higher deformation level, as seen in Figure 2.12, the strength of the brace in compression (i.e, positive δ value) was greater than the brace in tension.

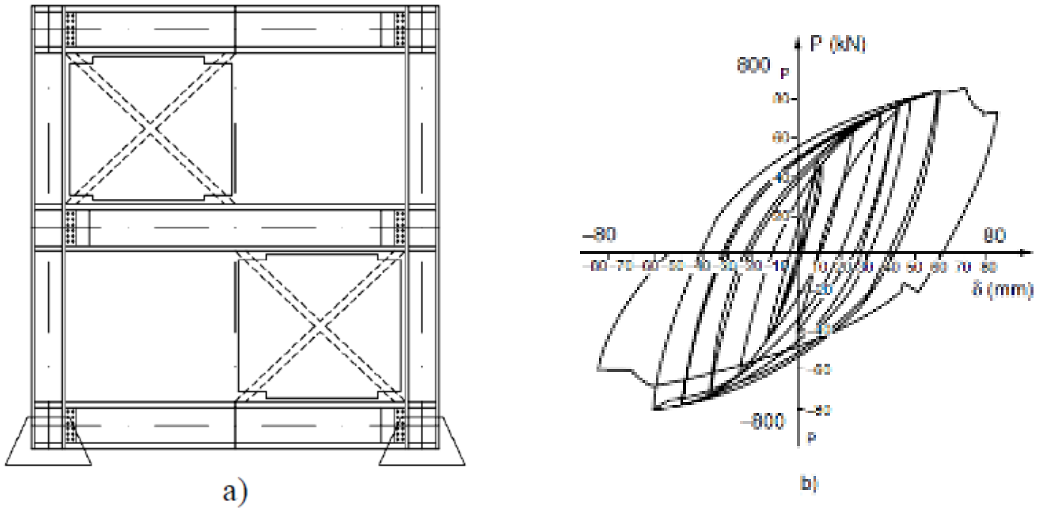


Figure 2.12 View of a) frame test using X-shaped steel brace core and b) hysteretic behavior of X-shaped BRB (Wakabayashi et al., 1973a; 1973b)

Kimura et al. (1976) developed the first working concept of conventional steel braces encased in mortar-filled steel tubes. The first experiments on this concept accomplished some stable hysteretic cycles but subsequently failed due to the transverse deformation of the mortar. This caused the local buckling of the core element outstands within the restraining tube. No unbonding material between the yielding core and the surrounding mortar was supplied so approximately 10-15% of the longitudinal strains of the core were transferred to the restraining tube. According to the test results, the core brace could exhibit greater compressive strength compared to tensile strength. In Figures 2.13 and 2.14, test specimen and its hysteretic response are demonstrated, respectively.

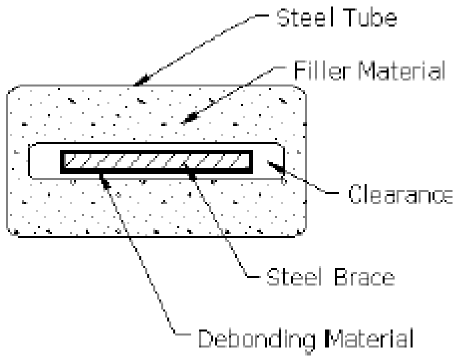


Figure 2.13 Cross section tested by Kimura et al. (1976)

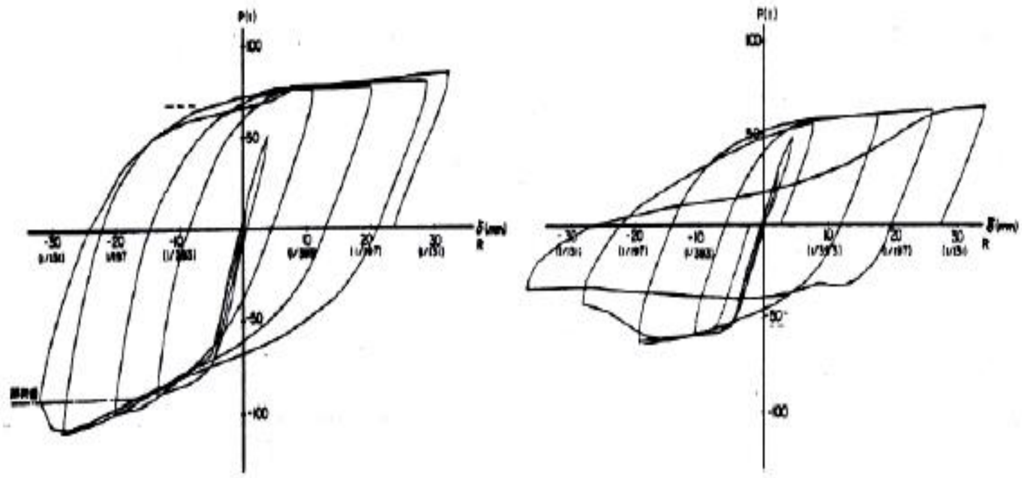
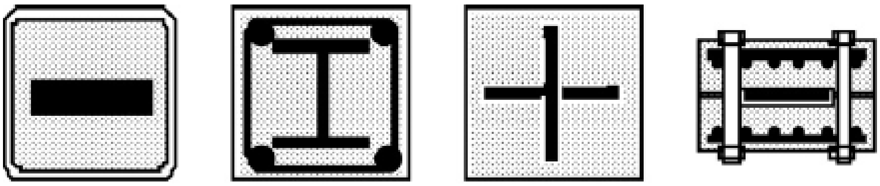


Figure 2.14 Hysteresis behavior result (Kimura et al., 1976)

Mochizuki et al. (1980) investigated the behavior of the braces encased in reinforced concrete. In the experimental study, the concrete protected from bonding to the internal core by use of a shock-absorbing material square cross-section members. The test results showed that under repeated loading concrete cracked, thus buckling restraint capacity decreased considerably.

The BRB arrangement shown in Figure 2.15a by Fujimoto et al. (1988). Such braces contained steel core, infilled mortar, and steel tube as the encasement. Fujimoto et al. expanded the study of Kimura et al. (1976) by conducting test on it with the debonding material. In this arrangement, the steel core carried the compressive forces and the outer steel tube was to ensure the sufficient stiffness to resist the buckling forces which resulted from the compressive forces (Xie, 2005).



a) b) c) d)

Figure 2.15 Cross sections of the BRB (Xie, 2005)

The cross sections of various BRBs were developed in Japan (Uang et al., 2004). The arrangement shown in Figure 2.15b by Nagao and Takahashi (1990) contained a wide flange sectioned steel core surrounded by the reinforced concrete section which had a square shape. The aim of the steel core was to carry the compresive forces and the reinforced concrete member ensured the lateral rigidity. The BRB arrangement shown in Figure 2.15c by Horie et al. (1993) composed of a steel core, which had a cruciform cross section and steel-fiber reinforced concrete section as an encasement. The BRB arrangement shown in Figure 2.15d by Inoue et al. (1992) contained a steel core, two precast concrete panels as an encasement and full-depth bolts to provide the, connection.

Watanabe et al. (1988) and Wada et al. (1989), and Watanabe et al. (1992) performed a series of test on five braces which had the same size steel core but each had a varying outer tube configuration. The purpose of the test was to investigate various outer tube configurations and flexural capacities on the overall capacity of the brace. Each brace was arranged diagoanally in the frame and an actuator was used to apply a cyclic horizontal displacement on the frame. Core material had a specified yield stress of σ_y= 2800 kg/cm^2, the outer tube material had a specified yield stress of σ_y= 3700 kg/cm^2. The inner core was coated with vinly/mastic tape to allow for Poisson's expansion to occur freely under compression. Table 2.1 summarizes the section properties, the critical loads, and the yield loads of the braces tested in their study. The various outer tube configurations given in Table 2.1 provided the levels of buckling stability varying between half yield load of the steel core to over three times the yield load ($0.55 \leq P_e/P_y \leq 3.53$), where P_y is the yield load and P_e corresponds to Euler buckling load of the outer tube. From the test results, it was obtained that the full axial load of the steel core could be occured irrespective of the outer tube configuration provided that the tube ensured sufficient buckling strength ($P_e/P_y \geq 1.0$). In the state of when the yield load of the steel core was greater than the buckling load of the outer tube, the failure of the brace was observed in a global buckling mode. The unbonded brace test, the cross-section of the brace, and the Euler buckling curves obtained from the investigation are given in Figures 2.16, 2.17, and 2.18, respectively.

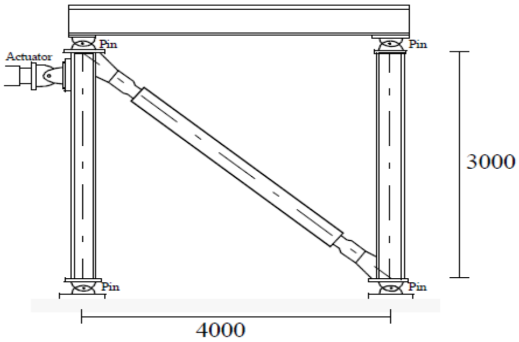


Figure 2.16 View of unbonded brace test (Watanabe et al., 1988; Wada et al., 1989; Watanabe et al., 1992)

Table 2.1 Comparison of unbonded brace test (Watanabe et al., 1988; Wada et al., 1989; Watanabe et al., 1992)

Speci-men No.	Core Member				Outer Tube		
	$b \times t$ (mm)	Cross-sectional Area, A (mm^2)	Yield Load, P_y (KN)	Local Buckling $P_m = 2\sqrt{\beta EI_o}$ ($\beta = 4500ksi$) (KN)	Tube Dimension B x D x T (mm)	Buckling Load, P_e (KN)	$\frac{P_e}{P_y}$
1	90 x 19	1684	476	1625	150 x 150 x 4.5	1677	3.53
2	90 x 19	1684	476	1625	150 x 100 x 4.5	661	1.39
3	90 x 19	1688	477	1625	150 x 100 x 3.2	492	1.03
4	90 x 19	1684	476	1625	150 x 75 x 4.5	343	0.72
5	90 x 19	1662	469	1625	150 x 75 x 3.2	257	0.55

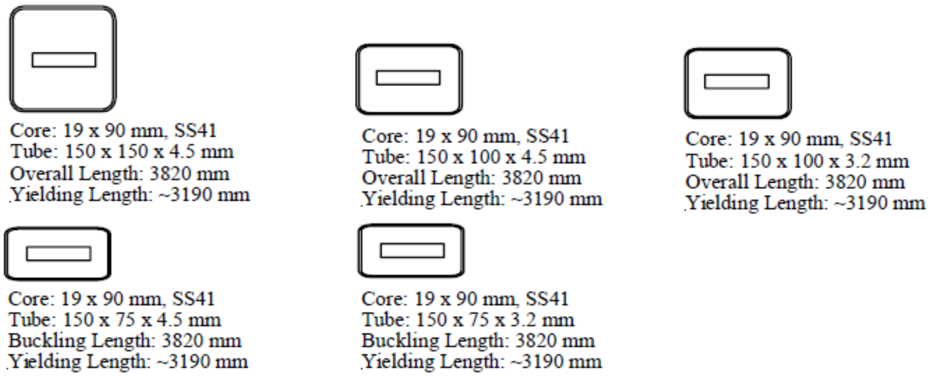


Figure 2.17 Comparison unbonded brace test in Japan (Watanabe et al., 1988; Wada et al., 1989; Watanabe et al., 1992)

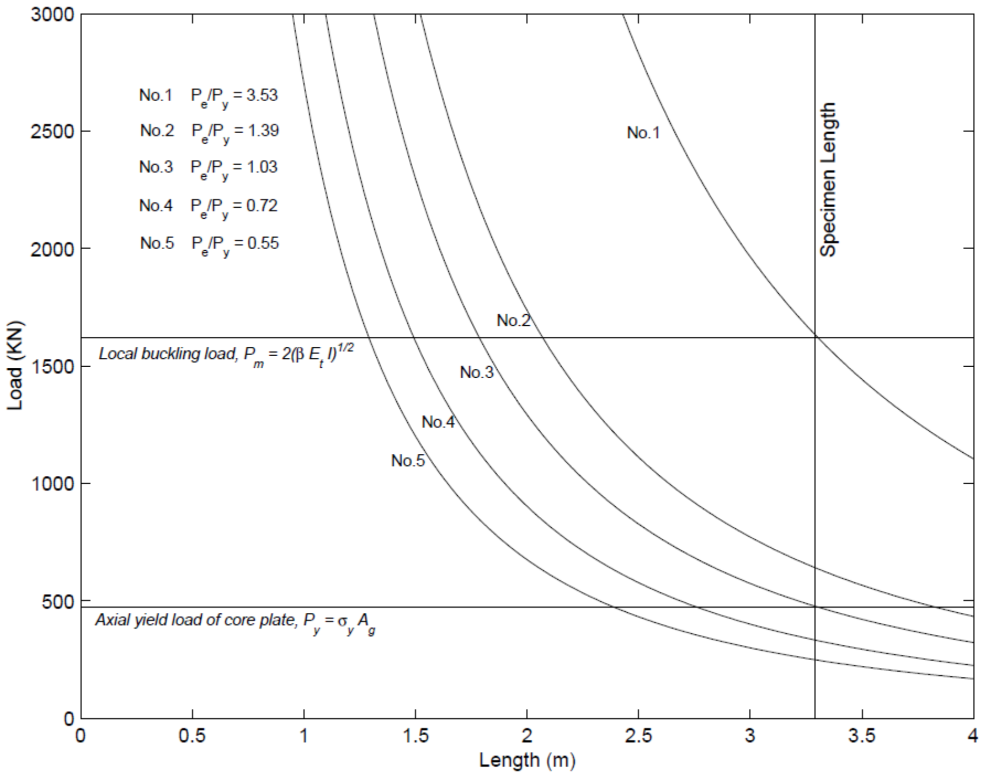
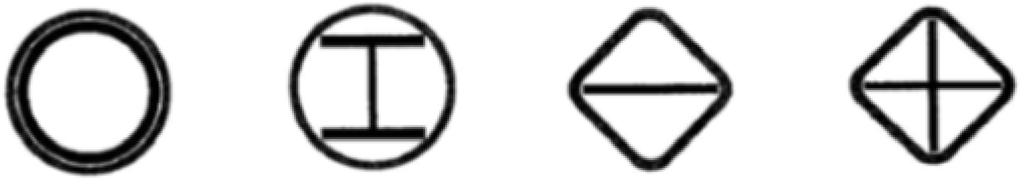


Figure 2.18 Euler buckling curves of five unbonded braces having the same inner core (19x90 mm) and different outer tube size (Watanabe et al., 1988; Wada et al., 1989; Watanabe et al., 1992)

In Figure 2.19, a few examples of the BRB specimens, which were scientifically investigated, are demonstrated. In BRBs, which took part in the figure did not contain the filler material. Thus, unbonding material was unrequired. But, for the arrangements shown in Figure 2.15, a sufficient gap size between the brace and the encasing tube was needed. So that, the necessary space for relative deformation between both members was provided and buckling effect in the inner core was prevented. Broadly, small gap sizes varying between 1 mm (Fujimoto et al., 1988; Kamiya et al., 1997; Suzuki et al., 1994) and 3 mm (Narihana et al., 2000) were used.



a) section by Tada et al., (1993) b) section by Suzuki et al., (1994) c) section by Kamiya et al., (1997) d) section by Shimizu et al., (1997)

Figure 2.19 Cross sections of various BRBs developed in Japan (Uang et al., 2004)

Hasegawa et al. (1999) conducted a shake table testing of two braces in Japan. In these tests, the brace was inclined with one end of the brace connected to the end of a vertical column, which was pinned to the shake table. A large accelerating mass laying on isolators and was connected to the vertical column by a rigid horizontal link produced dynamic force, which was applied to the test brace. The resource of the earthquake records were Kobe Marine Observatory Record from the 1995 Kobe and the 1940 El Centro. For this type of brace arrangement, the highest level corresponds to an axial strain of 7.2 percent was attained at an equivalent story deformation angle of 1/20. It was reported from this testing that the braces showed a stable hysteretic behavior throughout the testing. The unbonded brace test and the cross-section of the brace are shown in Figures 2.20 and 2.21, respectively.

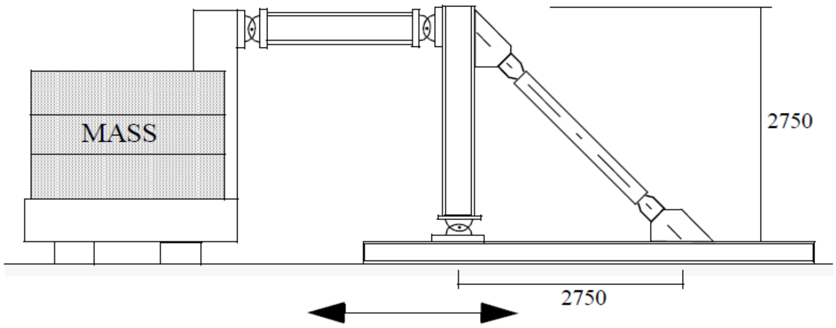


Figure 2.20 View of the unbonded brace tested Hasegawa et al. (1999)

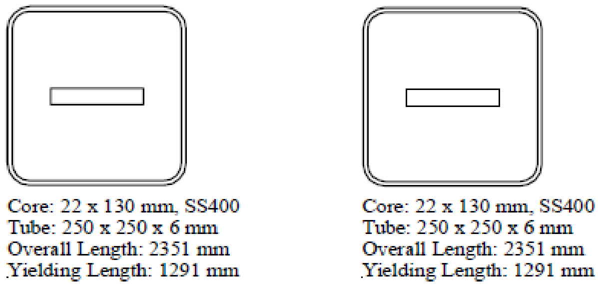


Figure 2.21 Unbonded braces tested in Japan (Hasegawa et al., 1999)

Iwata et al. (2000) examined the cyclic performance of four commercially available BRBs. The cross-section of the BRBs contained filler material. Moreover, the debonding materials were utilized to diminish the effect of friction forces in the brace core. A 1 mm opening was provided around the perimeter of all specimens so as to enable relative displacement between the members. When the compressive loading was applied to steel core, the relative displacement occured due to transverse displacement caused by the Poisson's effect. The cross-section of the unbonded brace and the test configuration of the brace are given in Figures 2.22 and 2.23, respectively.

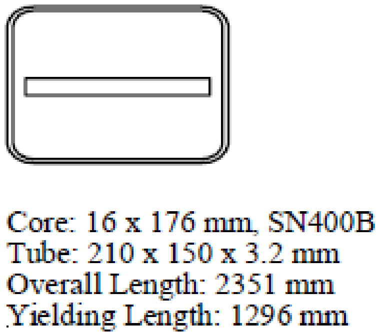


Figure 2.22 Unbonded brace test in Japan (Iwata et al., 2000)

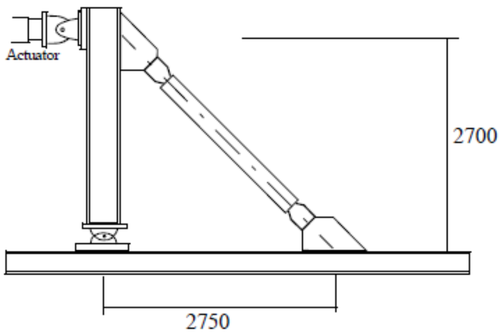
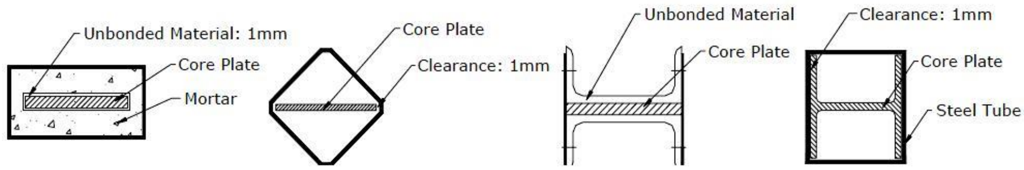


Figure 2.23 Test configuration of the model (Iwata et al., 2000)

As seen in Figure 2.24, a total of 4 cross-section was tested. Specimen 1 was designed after the investigation performed by Fujimoto et al. (1988). It contains steel core encased by mortar filled rectangular steel tube. Debonding between the steel core and surrounding mortar was provided by soft rubber material. After the fourteen cycles of loading, maximum strain level 3% was reached. Specimen 2 contained steel core surrounded by a square steel tube positioned as shown in Figure 2.24b. This arrangement was adjusted by considering the inelastic behavior model which was offered by Kamiya et al. (1997). This specimen displayed lower performance level when compared to others. During the second loading cycles, the buckling was observed in the middle of steel core at a strain level of 1%. As seen from the hysteresis cycle in Figure2.25b, specimen 2 had poor energy absorbtion capacity. Specimen 3 contained steel core restrained to prevent lateral buckling by two steel channels which were fastened with high strength bolts. This design was realized subsequently the research by Fukuda et al. (1999). Debonding was supplied with the same material as in specimen 1. Local buckling phenomenon under the first cycle of compressive loading occured at a strain level of 2.5 % in the stiffener end. Failure was observed under the first cycle of the compressive loading. One of the connecting high strength bolts reached at a strain level of 3%. A steel wide flange section was used as a core in specimen 4, surrounded by rectangular steel tube. The design was realized depending on the test results performed by Suzuki et al. (1994). This arrangement had no filler material content as in specimen 2. At a strain level of 2% during the second cycle of the compressive loading, local buckling phenomenon occured in the middle segment of brace. Again at the same loading cycle, when the strain level of 2.5% reached, some cracks was observed in the restraining tube, followed by rupture in the cracked part. Depending on the hysteresis curves in Figure 2.25, the authors ranked behavior of Specimen 1 as the best, followed by Specimens 3,4, and 2. However, all braces performed well under the 1% strain limit (Iwata et al., 2000).



a) b) c) d)

Figure 2.24 Different cross-sections of BRBs tested (Iwata et al., 2000)

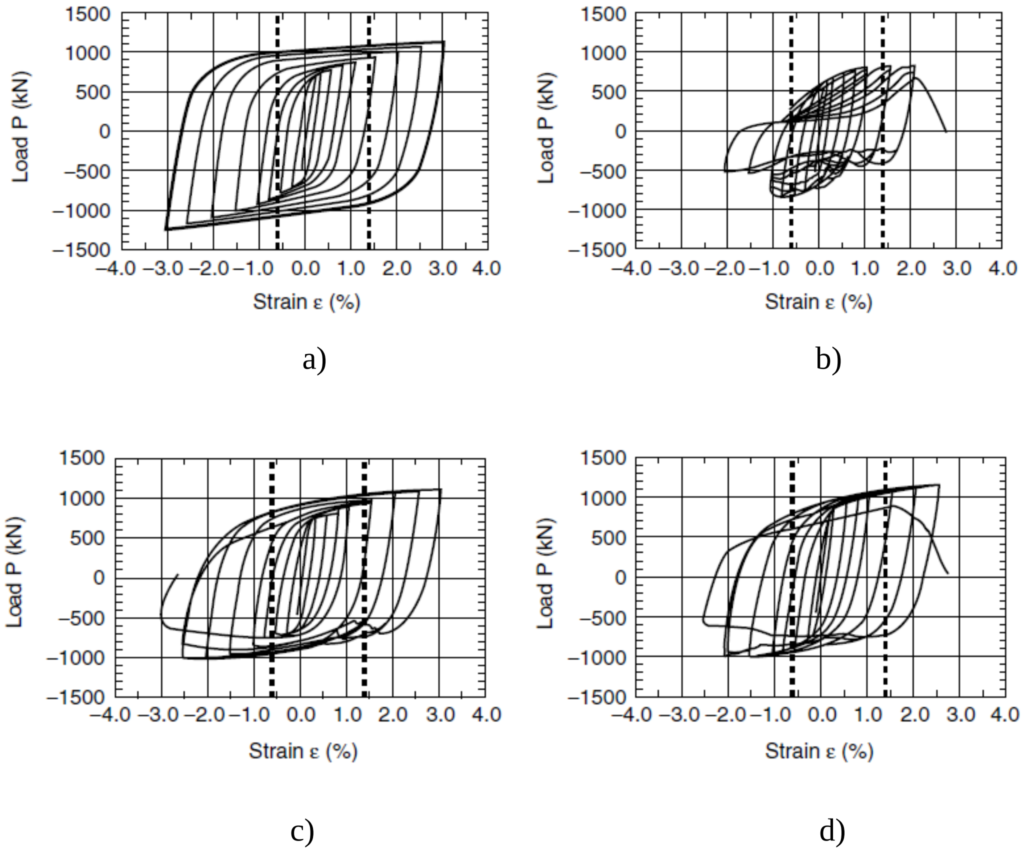


Figure 2.25 Hysteresis cycles for the four specimens (Iwata et al., 2000)

Sabelli et al. (2003) performed a comprehensive research on the earthquake response of the systems with BRBs configurated in a stacked chevron (inverted-V) pattern. A series of three story and six story buildings with BRBFs were designed by using nonlinear dynamic analysis in order to assess seismic demands on the buildings more effectively. The buildings were designed according to the 1997 NEHRP Recommended Provisions for Seismic Regulation for New Buildings and Other Structures (FEMA 302/303). Equivalent static lateral force method was applied to evaluate the shear demands on the frame. A response modification factor (R) value of either 6 or 8 was used. System overstrength factor (Ω_0) was adopted as 2. A suite of 20 horizontal ground acceleration records (Somerville et al., 1997) was utilized for the analysis. The study showed that BRBFs prevented many potential problems related to special concentric braced frames. To emphasize the weak points encountered in the system, this investigation was denoted to emphasize shortcomings with this system by maximize the predicted demands. The case study frame proposed better response compared to the conventional braced frames and moment resisting frames. The change of the R factor in the range of 6 to 8, had no effect the response of the frame. Also the response of the BRBFs could be easily affected by proportioning. Thus, it might be beneficial to estimate the dynamic response of the structures more precisely.

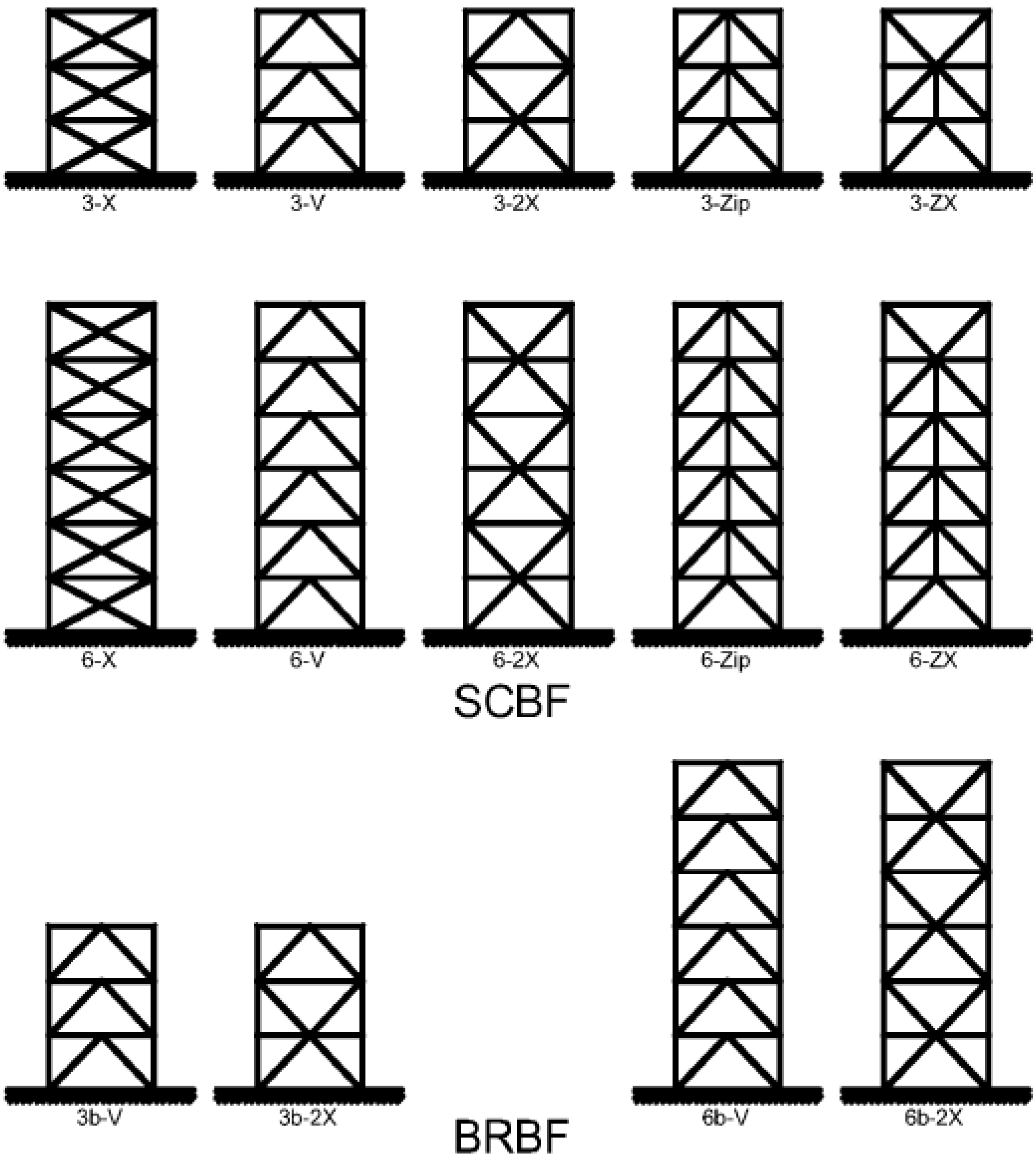


Figure 2.26 Some of the basic structural configurations of special concentric braced frame (SCBF) and buckling restrained braced frame (BRBF) (Sabelli et al., 2003)

In the study of Tremblay et al. (2006) a set of six BRB specimens were tested. In this test, two separate brace core segment lengths (two separate buckling restraining mechanisms of the concrete filled tubes and hollow steel tubes) were evaluated. The specimens were designed with reference to the requirements of the 2005 National Building Code of Canada (NBCC). Unbonding material made of polyethylene wrapping was utilized for the concrete filled BRB specimens. For the hollow steel tube BRBs, no unbonding material was utilized. The BRBs were bolted to gusset plates which were connected to the frame to create rigid brace- to-beam and brace-to-column connections. LVDTs (Linear variable differential transformers) were established to determine relative movement between the BRB core and the encasing tube. In addition, the strain gauges were fastened to the restraining tube to calculate the axial and bending strains on the tube. The applied loading histories contained a qualifying quasi-static cyclic test with stepwise incremental displacement amplitudes (H1) and a dynamically applied seismic loading (H2). The authors concluded that the BRB showed a stable, ductile inelastic response without fracture. The brace core was dog-bone shaped with a decreased cross-sectional area at its midlength within the encasing tube. The length of the decreased brace core cross-sectional area was reduced to achieve a stiffer brace. A demonstration of this concept is presented in Figure 2.27.

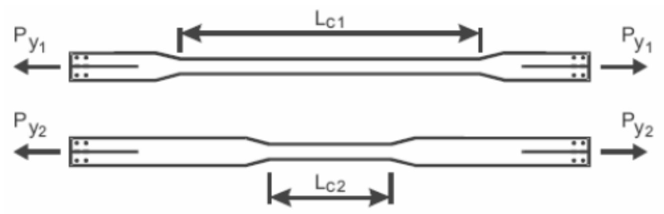


Figure 2.27 Reduced core brace length (Tremblay et al., 2006)

They also concluded that when large deformations adopted, both concrete filled specimens exhibited axial tension forces and compression resistance higher than predicted. This case caused primarily strain hardening of the material and friction developed between the brace core and buckling restraining arrangement. It was declared that a 25%-30% reduction occured in bending stiffness in the concrete filled BRBs upon axial yielding of the brace, which had no effect the axial response of the braces. The all-steel buckling restraining arrangement declared to be reasonable choice compared to the concrete BRB. But, local core buckling and strain uniformity problems occured in this type, had to be resolved. The conventional HSS (Hollow square section) brace faced with similar loading scheme (without fracture). However, it showed a poor energy dissipation capacity which was about 13% of that declared for the concrete BRB. When compared to the qualifying cyclic quasi-static loading protocol (H1), the dynamic loading protocol (H2) was declared to be less critical regarding low-cycle fatigue. The high strain rates caused by protocol H2 led to a 5% increment in brace yield resistance. They also suggested that the improvement of the loading protocol to impose the symmetrical demand was needed (Tremblay et al., 2006).

Various bracing arrangements in the frame can be utilized during the design of the BRB system. For example, Deulkar et al. (2010) employed five different configurations in their investigation as a BRB arrangement to enable the vibration control. Figure 2.28 shows such configurations for the frame system. They compared the reduction in the roof displacements for different configurations. In Figure 2.29, the roof displacement with story number for the frames studied is illustrated. As seen from the figure, the inverted V-bracing had the least roof displacement value in comparison to the others.

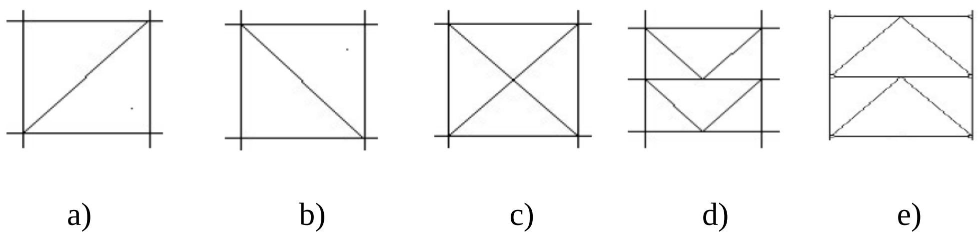


Figure 2.28 Bracing configuration of a) forward diagonal, b) backward diagonal, c) cross diagonal, d) V-bracing, and e) inverted-V bracing (Deulkar et al., 2010)

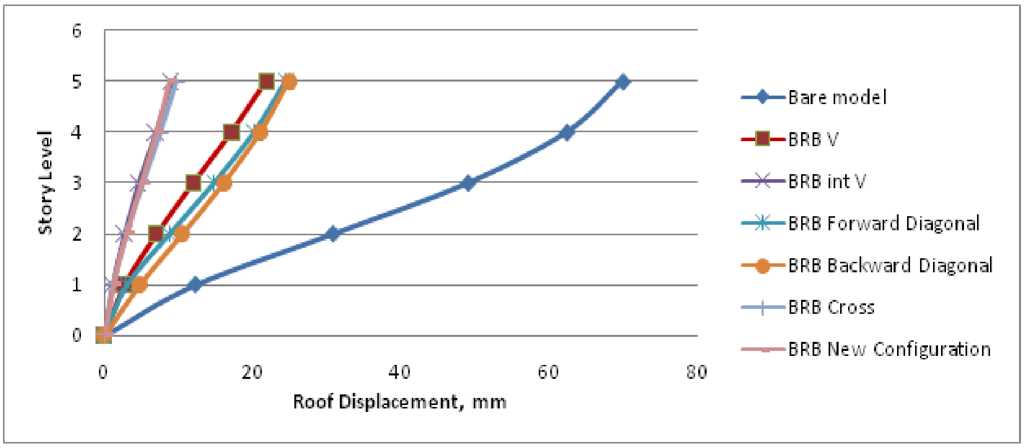
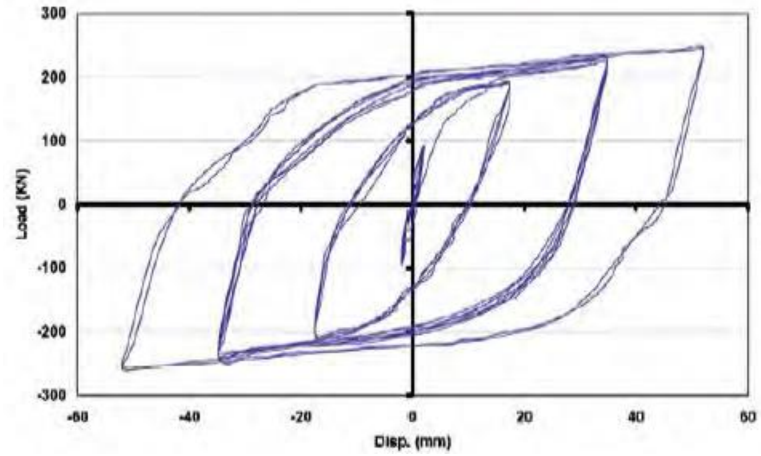


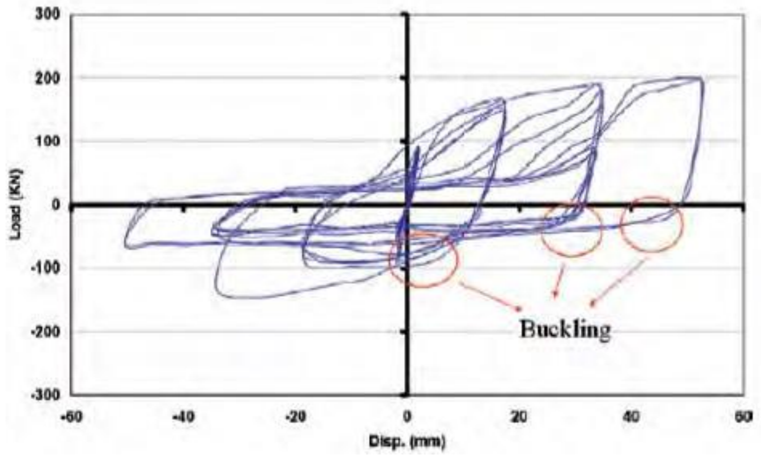
Figure 2.29 Comparison of displacement between different types of BRB bracing configurations (Deulkar et al.,2010)

Several debonding materials were utilized in the scope of BRB investigation. The first study on the BRB debonding material was conducted by Wakabayashi et al. (1973a; 1973b). They tested some materials such as epoxy resin, silicon resin, and vinyl tapes. But they decided to use a coating of silicon resin layer on the top of epoxy resin as a composite debonding material. Other debonding materials such as silicon painting, styrol foam, polyethene film sheets, silicon rubber were also utilized for this purpose (Xie, 2005).

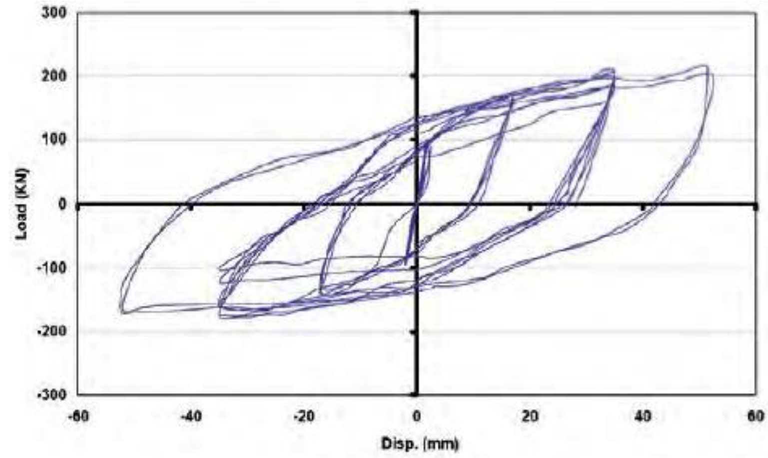
As a filler material, early studies conducted on the BRB utilized the reinforced concrete or mortar to restrain the buckling of the steel brace. A study carried out by Gheidi et al. (2009) investigated the influence of filler material. In their research, the uniaxial tests were performed on three specimens of BRB containing different filler materials. The results are illustrated in Figure 2.30, where the filler material of specimen a) is normal concrete, specimen b) is aggregate, and specimen c) is lean concrete.



a)



b)

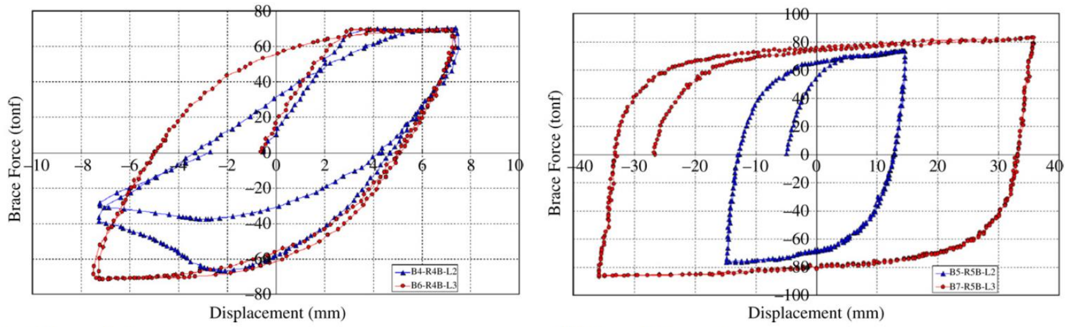


c)

Figure 2.30 Comparison of the hysteresis results between different types of filler material (Gheidi et al., 2009)

They concluded from the test results that the normal concrete was the most applicable one compared to the other filler materials. The research indicated that 25-30 MPa concrete was sufficient to prevent local and global buckling of flat plates. They also developed the specimens with no filler material content. But, regarding the risk and uncertainties in constructing a seismic resisting element and the availability of filler materials, it was safer to use filler material in the design of BRB (Gheidi et al., 2009).

The thickness of the encasing or steel tube had also an effect on the capability of the BRB elements to resist the flexural buckling. Consequently, the local buckling of the core plate and the entire strength as well as the stability of the system were affected. Takeuchi et al. (2010) investigated different arrangements of the tube restrainers which were exposed to cyclic loading tests. In the numerical analysis, the influence of the restrainer tube on the system performance was researched. It was concluded that the local buckling failure on the plate (which was related to the ratio of the width- or diameter- to- thickness of the rectangular or circular tube) might be occured. Ju et al. (2009) also concluded that the BRBs having 4 mm and 5 mm thickness of encasing tubes resulted in 34-54% higher compressive strength compared to the BRBs having a 3 mm encasing tube thickness. This case is illustrated in Figure 2.31.



(a) Thickness of tube = 4 mm. (b) Thickness of tube = 5 mm.

Figure 2.31 Comparison of hysteresis results between BRBs with different encasing tube thickness (Ju et al., 2009)

From the beginnings of 2000s, the competition grew among the BRB manufacturers. This caused development of specific connection type among the manufacturers. Nippon steel presented a typical standard bolted connection whereas Core Brace presented a modified bolted connction that used significantly fewer bolts when compared to the typical bolted connection. Another BRB manufacturer, Star Seismic developed unique true pin connection consisting of a large drift pin with retaining plates. Xie (2005) compared the advantages and disadvantages of all three types of the connection as given below. Figures 2.32, 2.33, and 2.34 show the standard bolted connection, modified bolted connection and true pin connection, respectively. For example, the standard bolted connection had the benefits of;

a) oversized holes enabled more erection tolerance,
b) multiple bolts ensured more connection redundancy, and
c) better distribution of forces to gusset plate.

Its downside might be summarized as;

a) larger gusset plates and shorter BRB yield length resulting in larger strains,
b) a large quantity of splice plates and bolts caused high installation cost, and
c) the formation of the secondary moment between the connection and brace. Even though the modified bolted connection had the same pros and cons of the standard bolted connection. It had one more advantage that significantly fewer bolts and no splice plates to install resulting in work force reduction were needed.

Finally, the true pin connection had the advantages of;

a) longer BRB yield length resulting in smaller strains for a given load,
b) a true pinned connection that eliminates in-plane secondary moments due to drift, and
c) single pin reduces installation cost. Moreover, this type of the connection had the disadvantage of very small erection tolerance.

Figure 2.32 Standard bolted connection (Xie,2005)

Figure 2.33 Modified bolted connection (Xie, 2005)

Figure 2.34 True pin connection (Xie, 2005)

CHAPTER 3
METHODOLOGY

3.1 Case study reinforced concrete structures

In this study, to evaluate the effectiveness of using buckling-restrained braces (BRBs) on the structural response of the high-rise reinforced concrete (RC) buildings, 9 and 18 story RC framed structures which were slightly modified from the models studied by El-Amoury and Ghobarah (2005) were used. The multistory frames had three 6 m long bays and the story height was 3.6 m for all stories. For the 9 and 18 story RC frames, the dimensions of the columns and beams are given in Figure 3.1. The compressive strength of the concrete and the yield strength of the steel were assumed to be 21 MPa and 300 MPa respectively. These frames were assumed to have a uniform mass distribution over the height of the structure. The weight due to the dead load on the frames was taken as the same for each floor which was 663.5 kN/floor. So as to examine the influence of the BRBs for seismic upgrading of the original structures, the BRBs were added to each story of the original frame (OF) systems in diagonal configuration. The view of the buckling restrained braced frames (BRBFs) is given in Figure 3.2.

The natural periods of vibration of the 9 and 18 story RC buildings with and without BRBs were determined through the eigen-value analysis. For the 9 story OF, the first three fundamental periods of vibration was obtained as T_1= 1.51 s, T_2= 0.55 s, and T_3= 0.31 s. On the other hand, the periods of vibration for the 9 story BRBF was determined as T_1= 0.98 s, T_2= 0.33 s, and T_3= 0.19 s. Furthermore, for the 18 story OF, the first three vibration periods were attained as T_1= 2.69 s, T_2= 0.96 s, and T_3= 0.57 s while for the BRBF they were obtained as T_1=1.94 s, T_2=0.65 s, and T_3=0.36 s. From the results of the eigen-value analysis, it was observed that with the inclusion of BRBs, the stiffness of system increased and the free vibration period of the frames had tendency to decrease. Table 3.1 summarizes the natural period of the structures.

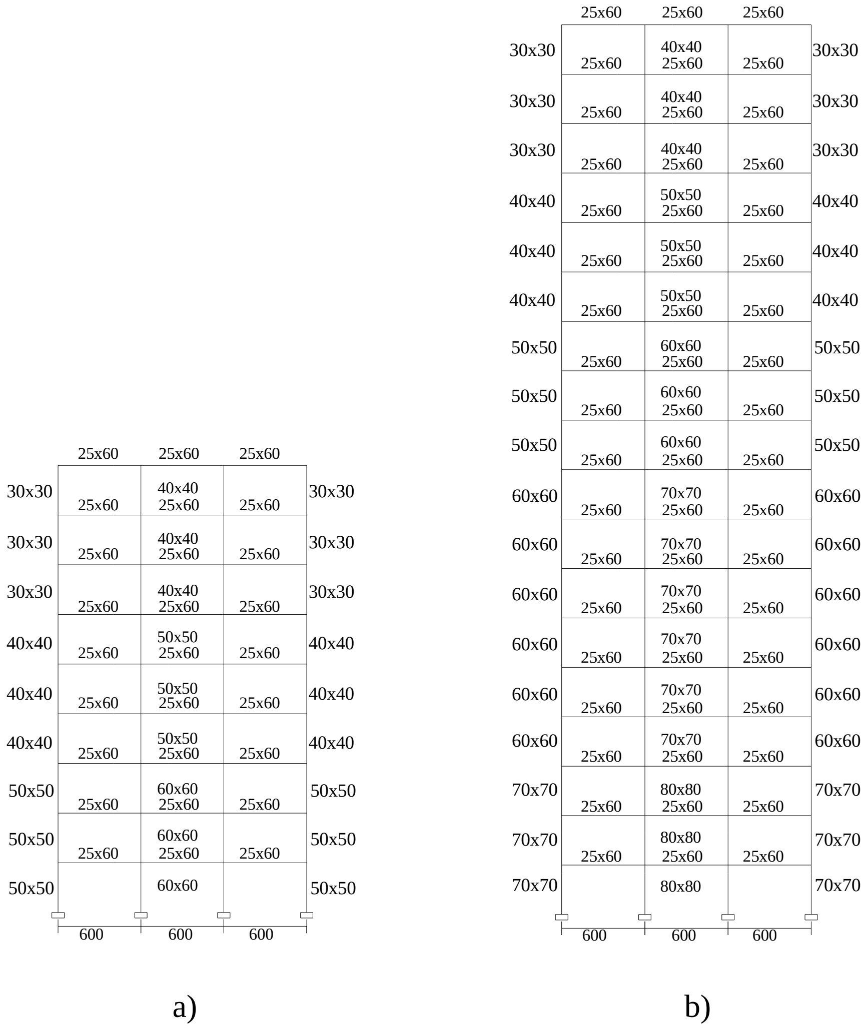


Figure 3.1 Frame elevations of a) 9 and b) 18 story original RC structures (El-Amoury and Ghobarah, 2005)

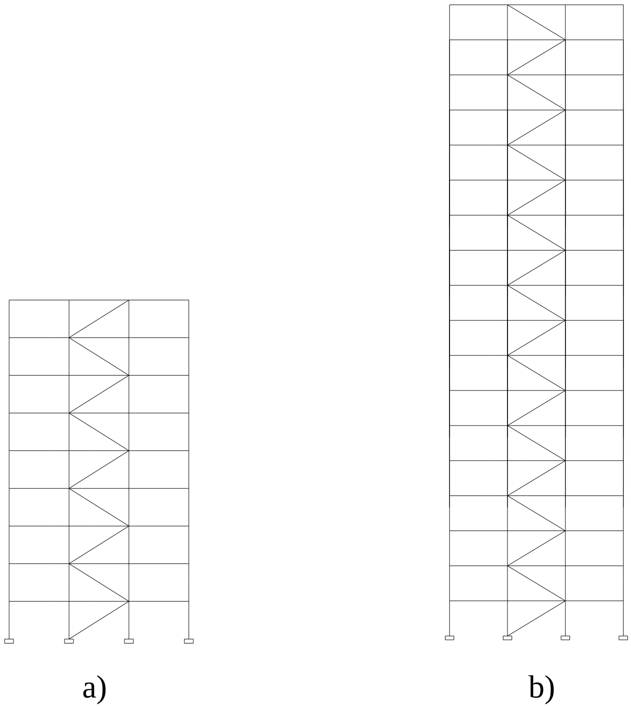


Figure 3.2 Frame elevations of a) 9 and b) 18 story BRBFs under investigation

Table 3.1 Natural periods of the RC structures

Frame Cases	Natural Periods (s)		
	T_1	T_2	T_3
9-Story OF	1.51	0.55	0.31
9-Story BRBF	0.98	0.33	0.19
18-Story OF	2.69	0.96	0.57
18-Story BRBF	1.94	0.65	0.36

3.2 Nonlinear analysis

Nonlinear dynamic time history analysis can perform so as to investigate the complete nonlinear behavior of the building structures. This method is regarded to be most accurate of the analysis methods since it approximately performs the the complete inelastic response of the structure, and has become the benchmark in which the accuracy of other methods assessed (SEAOC, 2009). Nonlinear dynamic procedure intrinsicly deals with the contribution of higher modes. In this method, the structure experiences real ground motion records different from all of the other approximate analysis procedures as the inertial forces are directly obtained from these ground motions. Then, the responses of the structure either in deformations or in forces are computed as a function of time, regarding the dynamic properties of the structure (Sedeeq, 2014).

The nonlinear time- history analysis is realized as following orders (Ameen, 2012):

- The model corresponding the building structure is formed and vertical loads, member properties, and member nonlinear behavior are defined and assigned to the model,
- Floor masses are assigned to the model,
- Hinge properties are defined as well these properties are assigned to the member ends with regarding end-offsets,
- The ground motion record is defined as in the form of the acceleration versus time function, and
- The first loading is made on the model similar to the pushover analysis to represent the initial case. This case consists of the dead loads and reduced live loads.

In this study, the analysis and time history parameters were defined so as to realize a nonlinear time history analysis by means of SAP 2000 Advanced 14.0.0 (CSI, 2011) to assess the seismic performance of 9 and 18 story RC buildings with and without BRBs. In the nonlinear time history analysis, analytical models consisting nonlinear behavior of the structural members were subjected to different ground motions. Moreover, a critical damping ratio of 5% was considered for all analysis of BRBFs and OFs. Hilber-Hughes-Taylor direct integration method was adopted and Rayleigh damping was utilized for the first and second modes.

$$\alpha = \zeta \frac{2\,\omega_i \omega_j}{\omega_i + \omega_j} \tag{3.1}$$

$$\beta = \frac{2\zeta}{\omega_i + \omega_j} \tag{3.2}$$

It was noted to emphasize that the unbraced and braced frames had the same damping ratios. In order to accomplish this, the nonlinear time history analysis was realized in two steps, in the first step; an Eigen value analysis was realized to obtain natural periods and mode shapes of the frames, in the second step; the correct α and β damping coefficients were calculated and then nonlinear dynamic time history analysis were performed.

3.3 Earthquake ground motions

The inertial forces were directly obtained under these ground accelerations and the responses of the frames were calculated as a function of time based on the properties of the structures. In the analysis of the frames, natural ground accelerations, namely, 1999 Hector Mine, 1999 Chi Chi, and 1999 Düzce earthquakes generated as spectrum compatible were utilized (PEER, 2011). Figures 3.3-3.5 demonstrate the unscaled earthquakes time histories of these records. Moreover, the design code spectrum and the average of 5% damped linear elastic response spectra of the scaled natural ground accelerations are given in Figure 3.6.

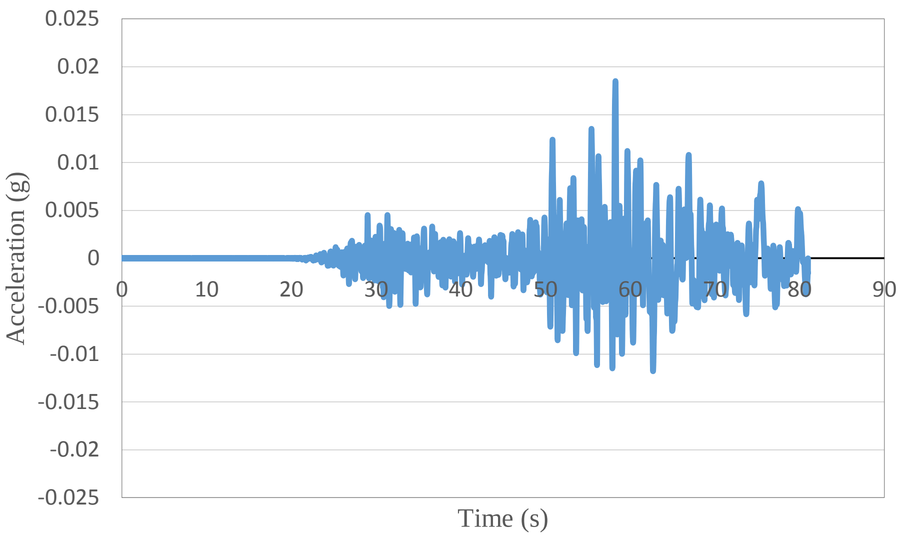


Figure 3.3 Acceleration time history of Hector Mine

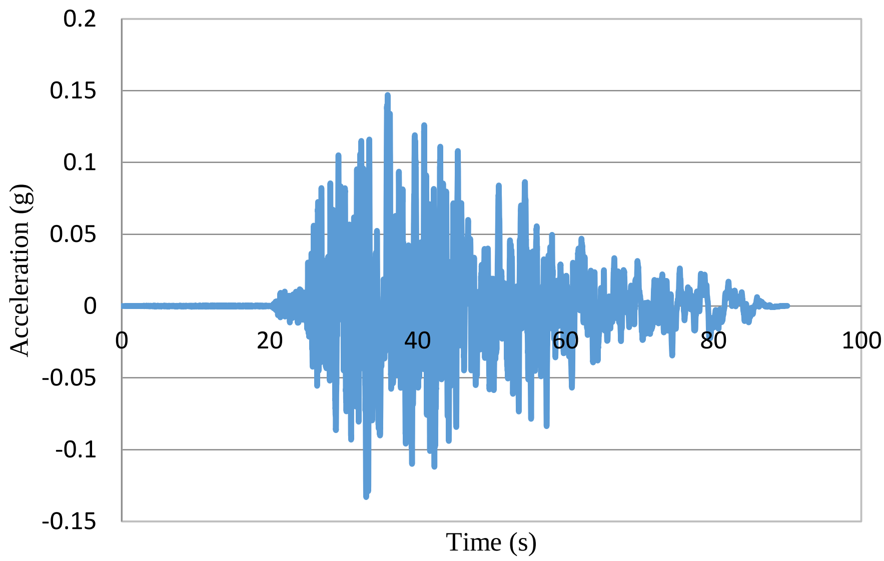


Figure 3.4 Acceleration time history of Chi Chi

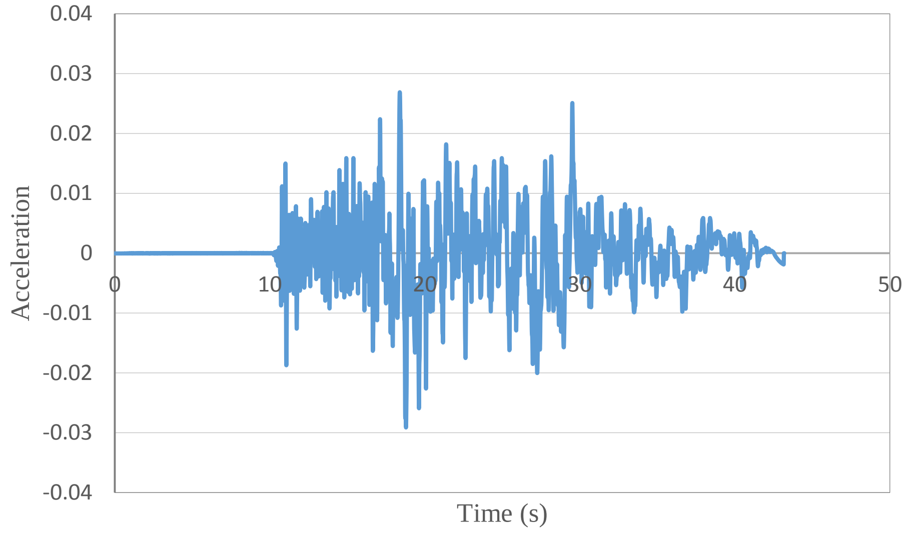


Figure 3.5 Acceleration time history of Düzce

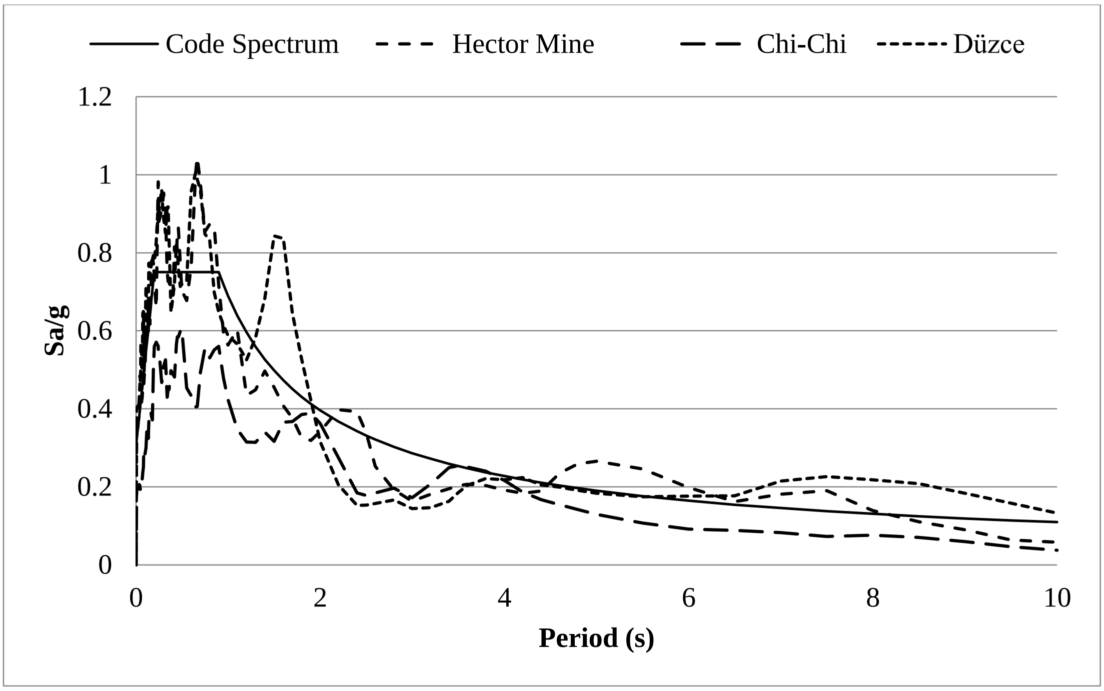


Figure 3.6 Average linear elastic response spectra of the selected ground accelerations

CHAPTER 4

RESULTS AND DISCUSSION

4.1 General

This chapter presents the results of the nonlinear time history analysis performed on original frames (OFs) and buckling restrained braced frames (BRBFs) under the spectrum compatible scaled ground accelerations. 1999 Chi Chi, 1999 Düzce, and 1999 Hector Mine earthquakes were utilized. In this regard, 4 different frame cases, namely, 9-story OFs, 9-story BRBFs, 18-story OFs, and 18-story BRBFs were considered in the analysis. Performance characteristics in terms of displacement time history and inter-story drift ratio were given below.

4.2 Displacement time history

From the nonlinear analysis, the variation in the displacement time history for the 9 and 18 story reinforced concrete (RC) structures with and without buckling-restrained braces were obtained for three different ground motions, namely, Chi Chi, Düzce, and Hector Mine. The results of the displacement vs. time for each story level are illustrated Figures 4.1 to 4.12 and 4.13 to 4.30 for the 9 and 18 story original frames (OFs) and buckling-restrained braced frames (BRBFs), respectively. As seen from the figures, under all earthquakes, the value of the displacement had a tendency to increase with increasing the story height for both the OFs and BRBFs. However, irrespective of the earthquake records used, the BRBFs had consistently lower displacement in comparison to the OFs.

Moreover, the roof displacement time history of the existing and retrofitted RC frames under Chi Chi, Düzce, and Hector Mine earthquake accelerations can be observed from Figures 4.4, 4.8, and 4.12 for the 9 story case and Figures 4.18, 4.24, and 4.30 for the 18 story case. As seen in the figures, the involvement of the BRBs considerably decreased the value of the roof displacement demand of the OFs. For instance, under

Chi-Chi earthquake, the maximum roof displacement of the 9 story OF was about 47 cm while the maximum roof displacement of the 9 story BRBF was achieved as approximately 15 cm. In addition, for the 18 story structure, under the Düzce earthquake, the maximum roof displacement of the OF was achieved as about 80 cm whereas the maximum roof displacement of the BRBF was found as about 40 cm. Therefore, it was observed that the addition of buckling restrained braces reduced remarkably the roof displacement demand of the OFs.

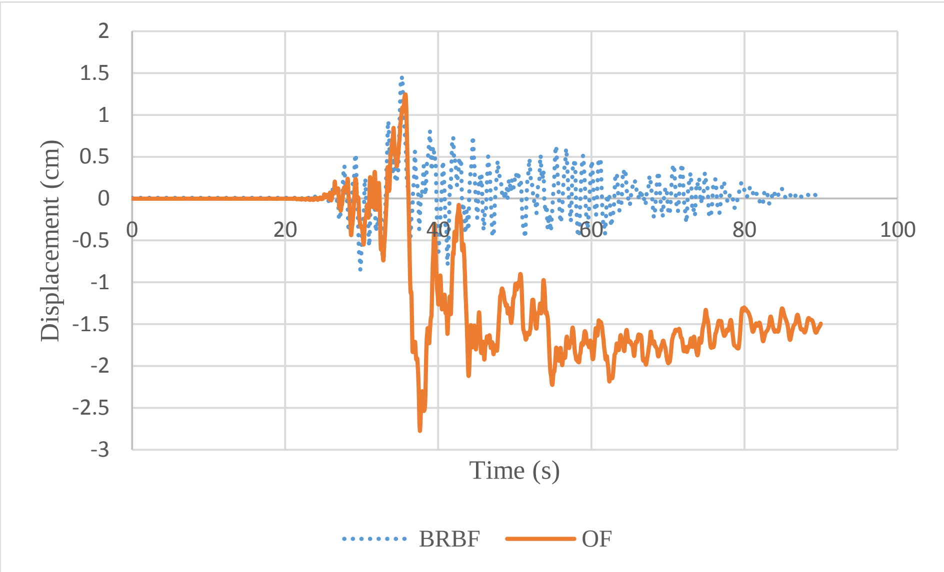


Figure 4.1 1 st story displacement time history of 9 story OF and BRBF under Chi Chi earthquake

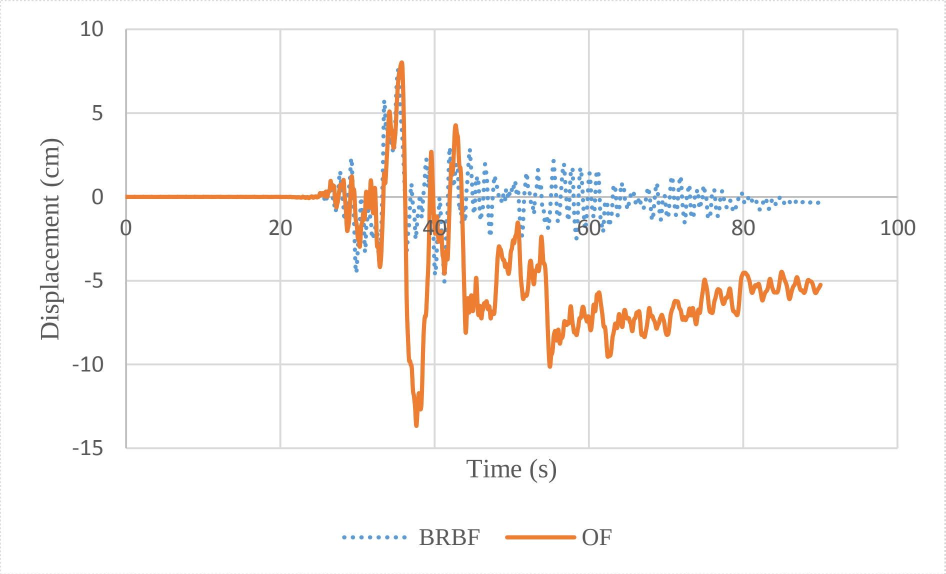


Figure 4.2 3 rd story displacement time history of 9 story OF and BRBF under Chi Chi earthquake

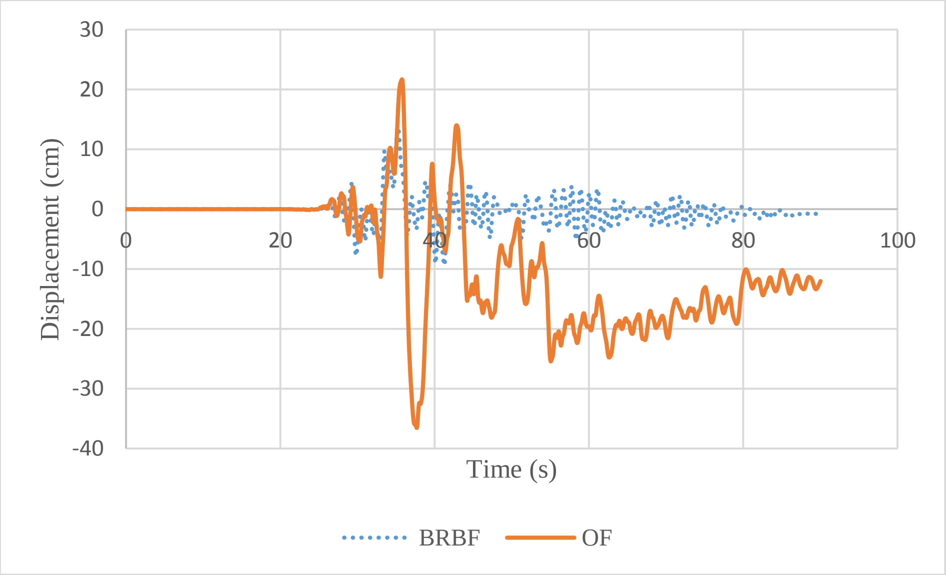


Figure 4.3 6 th story displacement time history of 9 story OF and BRBF under Chi Chi earthquake

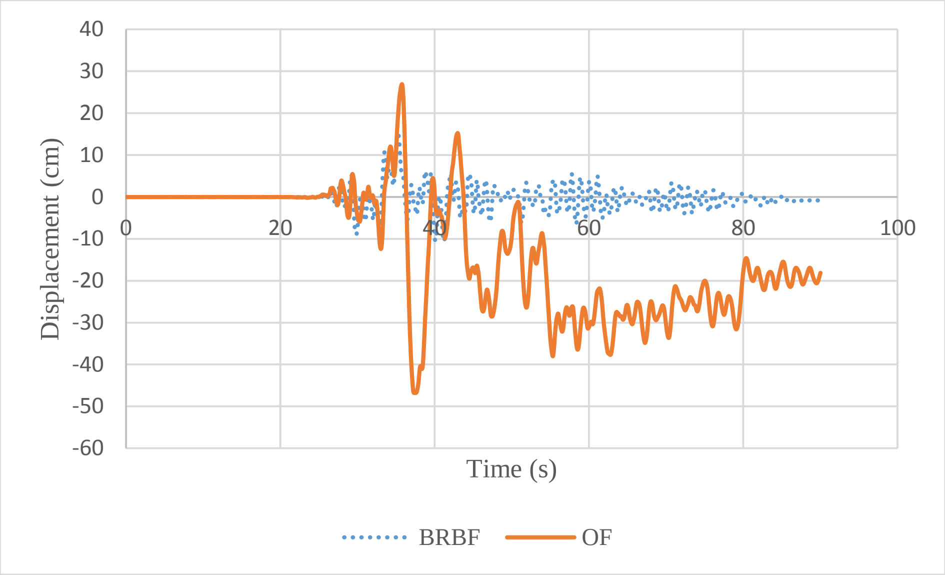


Figure 4.4 9 th story displacement time history of 9 story OF and BRBF under Chi Chi earthquake

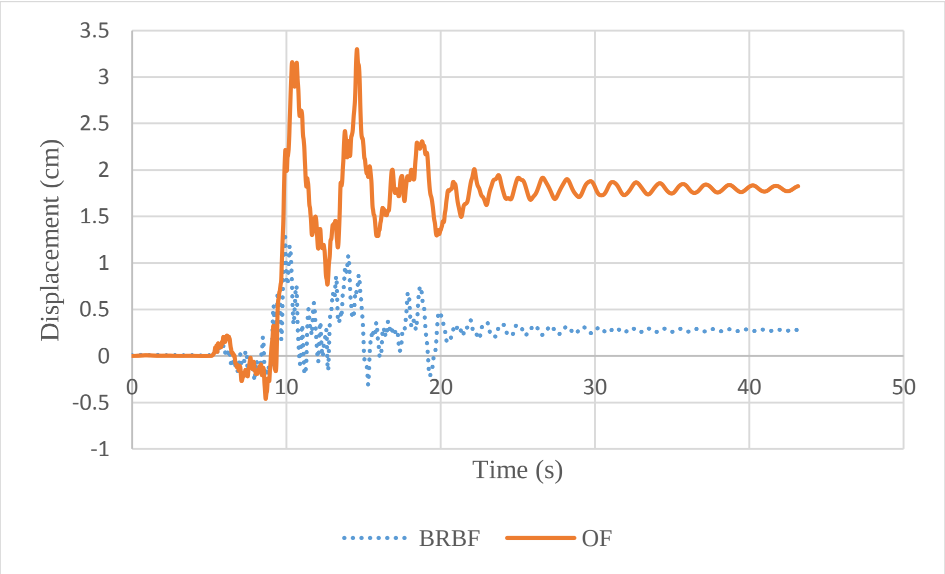


Figure 4.5 1 st story displacement time history of 9 story OF and BRBF under Düzce earthquake

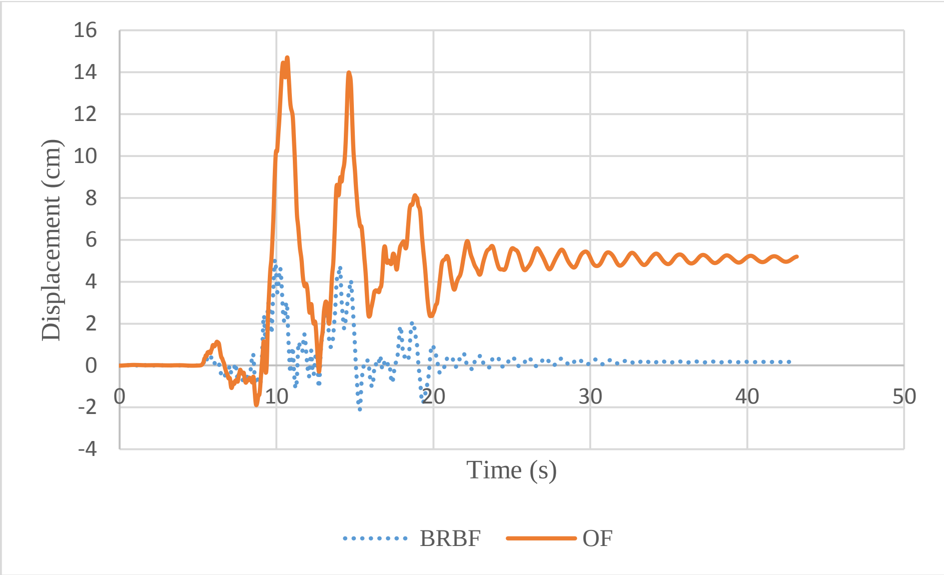


Figure 4.6 3 rd story displacement time history of 9 story OF and BRBF under Düzce earthquake

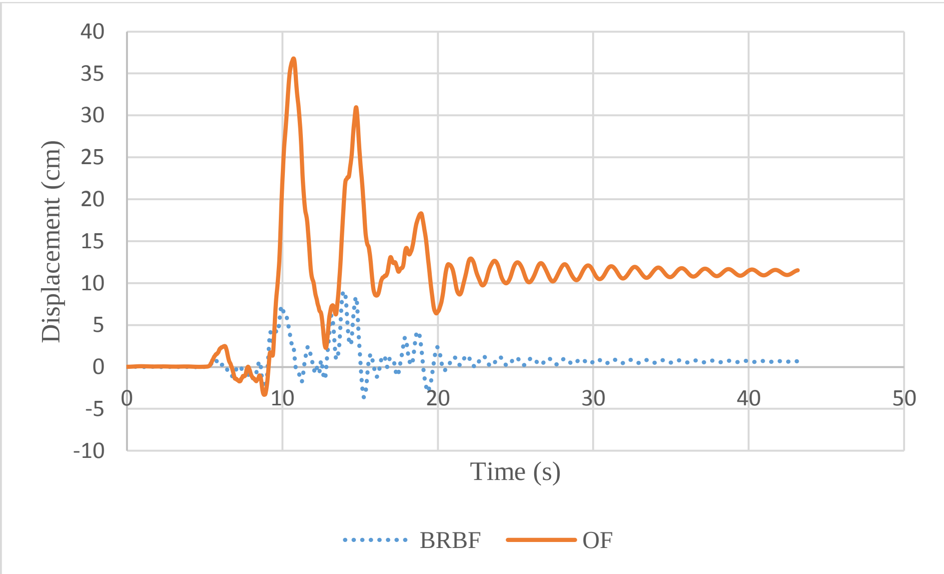


Figure 4.7 6 th story displacement time history of 9 story OF and BRBF under Düzce earthquake

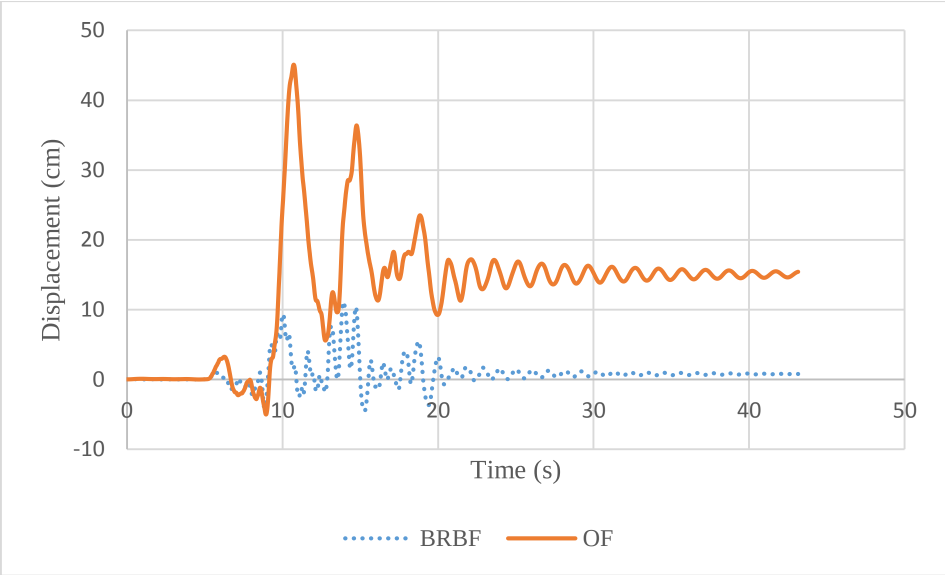


Figure 4.8 9 th story displacement time history of 9 story OF and BRBF under Düzce earthquake

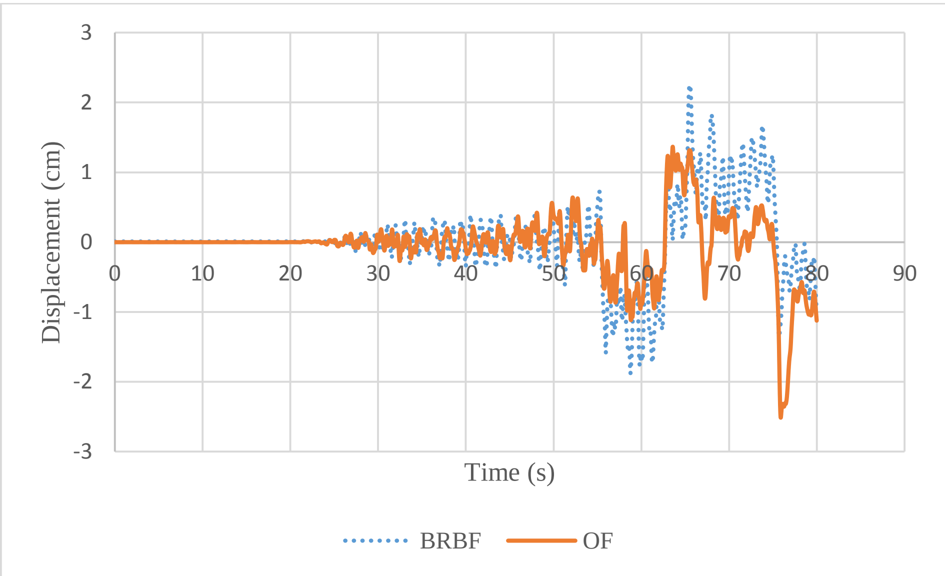


Figure 4.9 1 st story displacement time history of 9 story OF and BRBF under Hector Mine earthquake

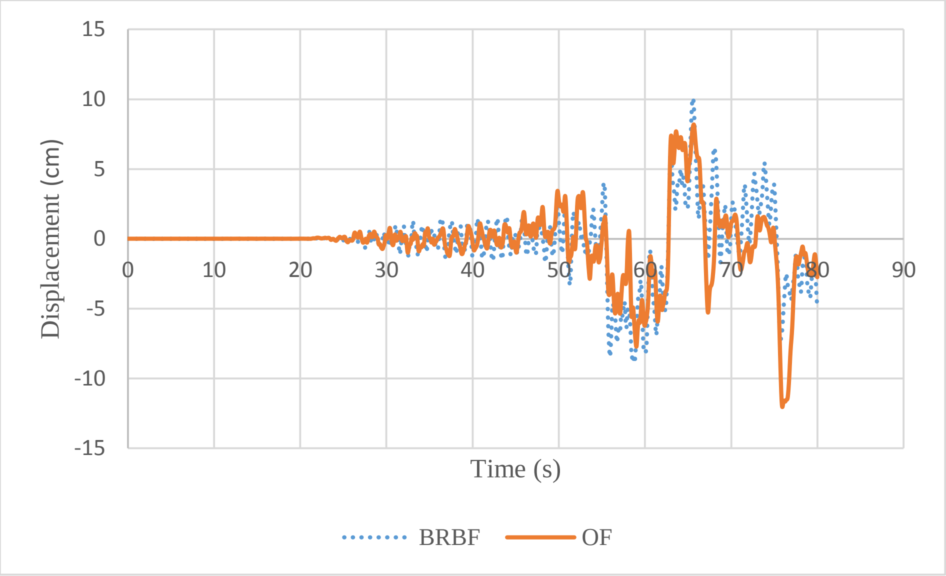


Figure 4.10 3 rd story displacement time history of 9 story OF and BRBF under Hector Mine earthquake

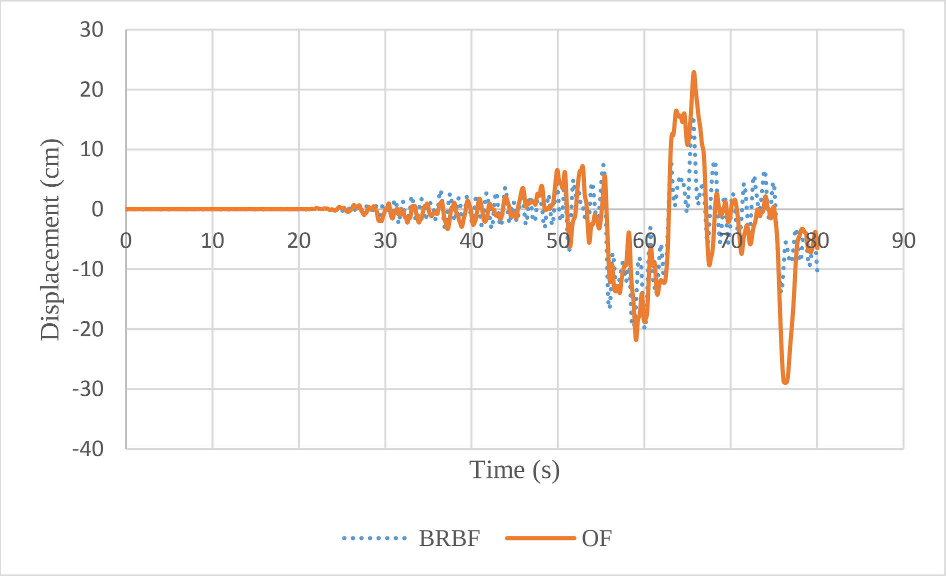


Figure 4.11 6 th story displacement time history of 9 story OF and BRBF under Hector Mine earthquake

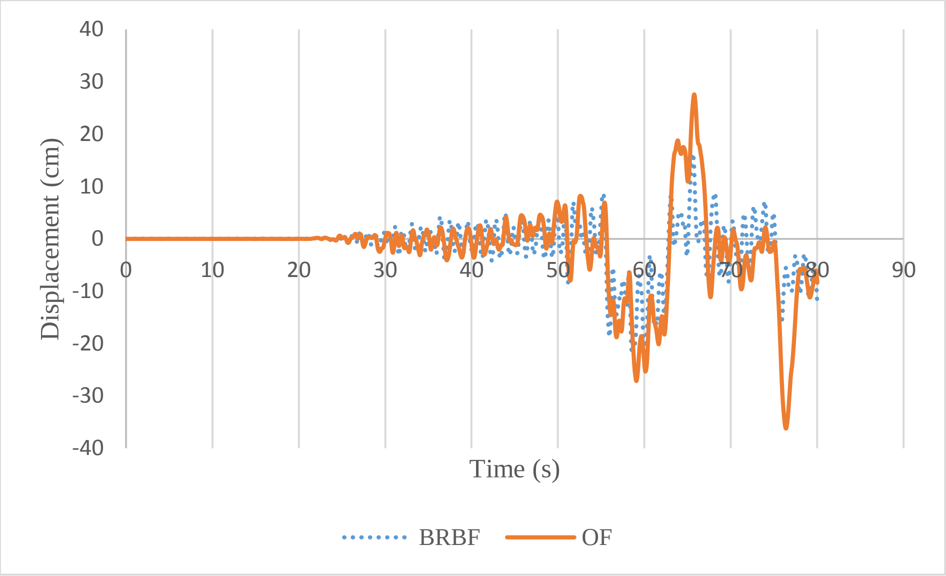


Figure 4.12 9 th story displacement time history of 9 story OF and BRBF under Hector Mine earthquake

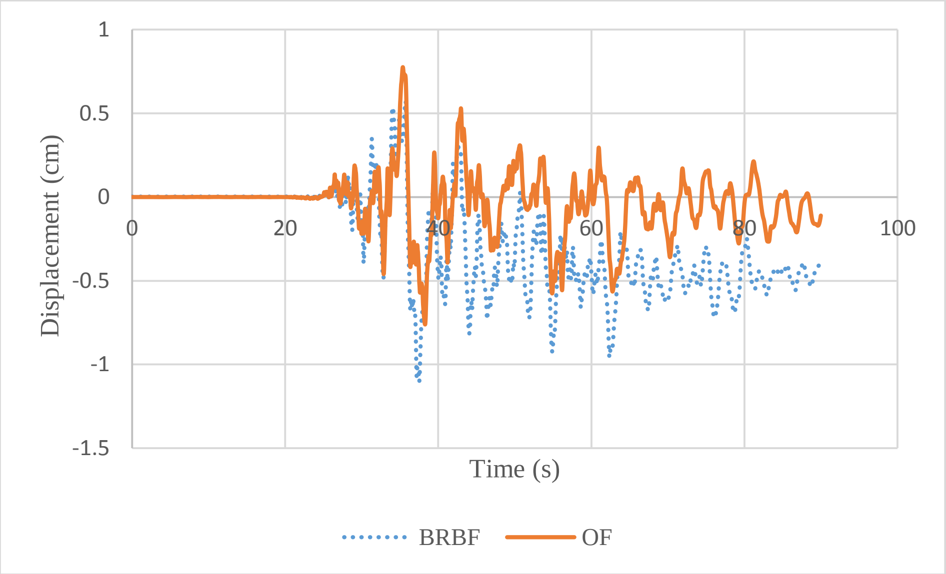


Figure 4.13 1 st story displacement time history of 18 story OF and BRBF under Chi Chi earthquake

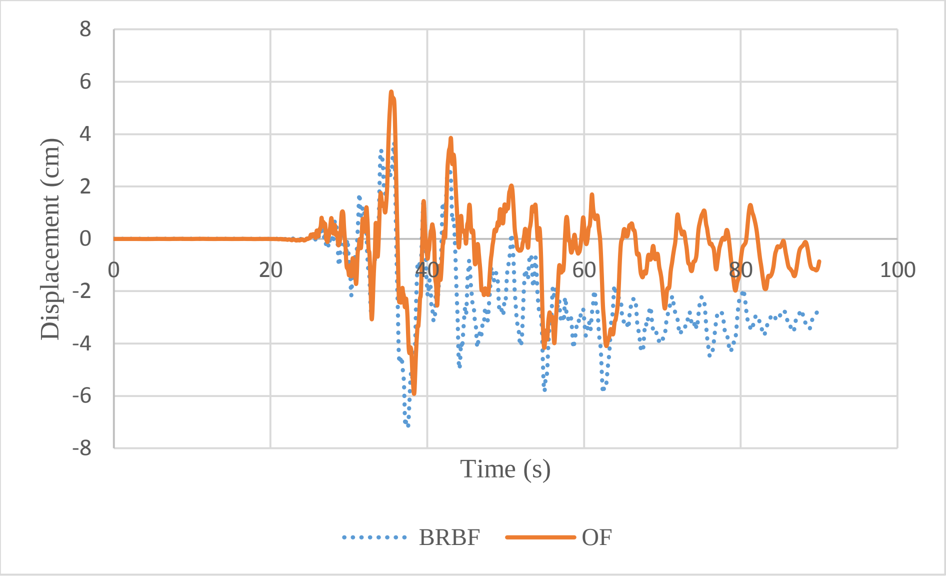


Figure 4.14 3 rd story displacement time history of 18 story OF and BRBF under Chi Chi earthquake

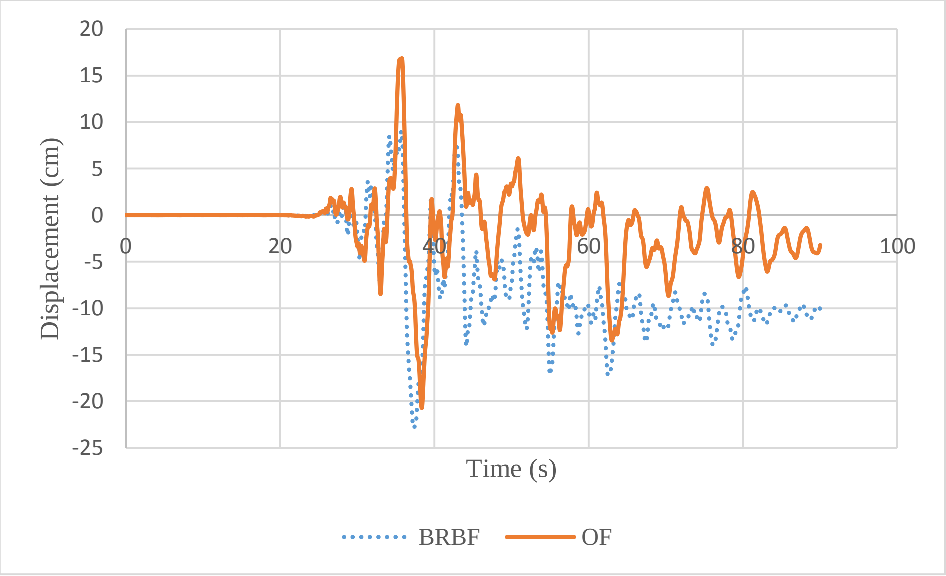


Figure 4.15 6 th story displacement time history of 18 story OF and BRBF under Chi Chi earthquake

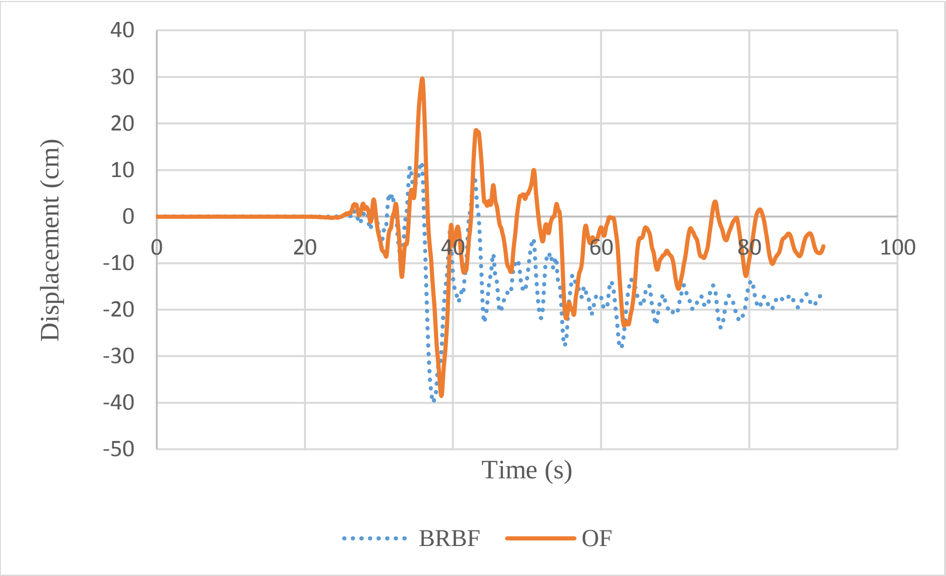


Figure 4.16 9 th story displacement time history of 18 story OF and BRBF under Chi Chi earthquake

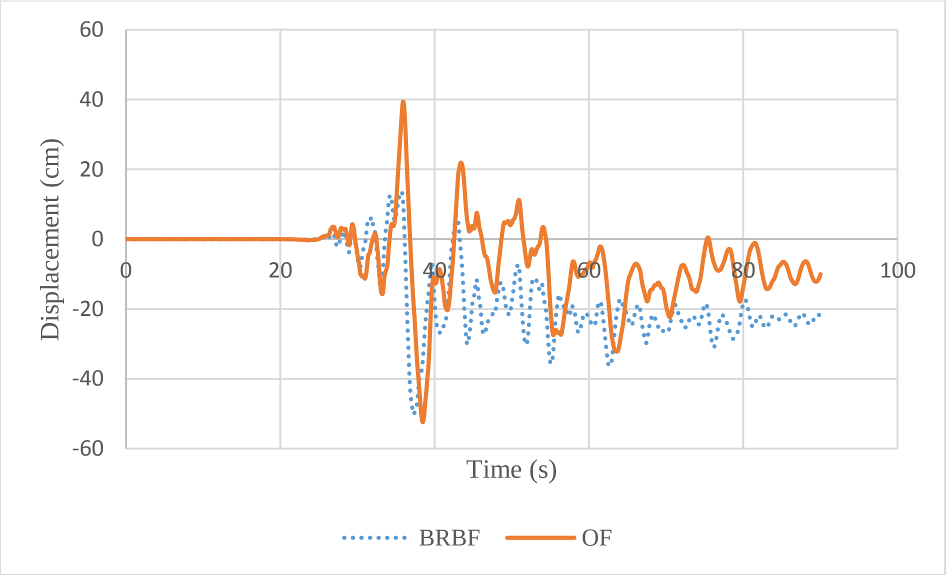


Figure 4.17 12 th story displacement time history of 18 story OF and BRBF under Chi Chi earthquake

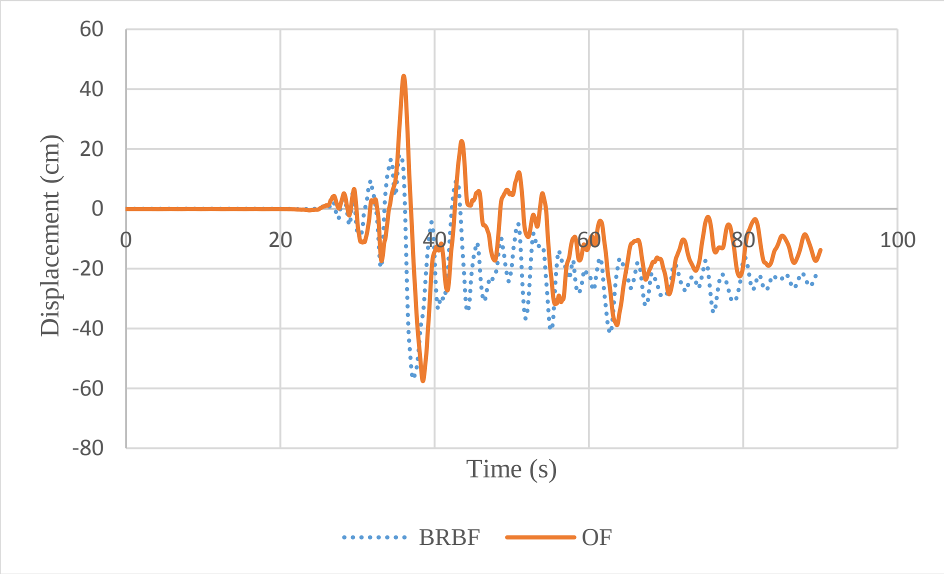


Figure 4.18 18 th story displacement time history of 18 story OF and BRBF under Chi Chi earthquake

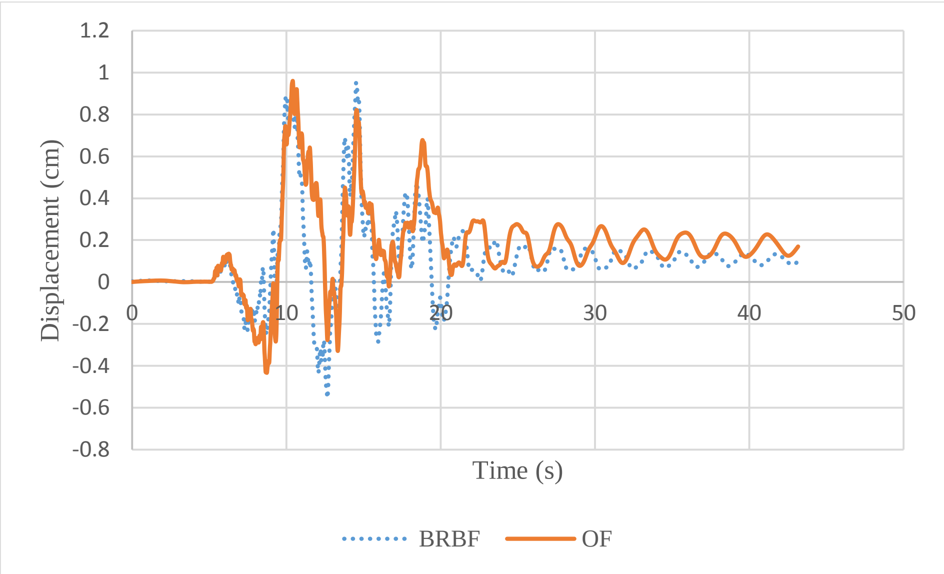


Figure 4.19 1 st story displacement time history of 18 story OF and BRBF under Düzce earthquake

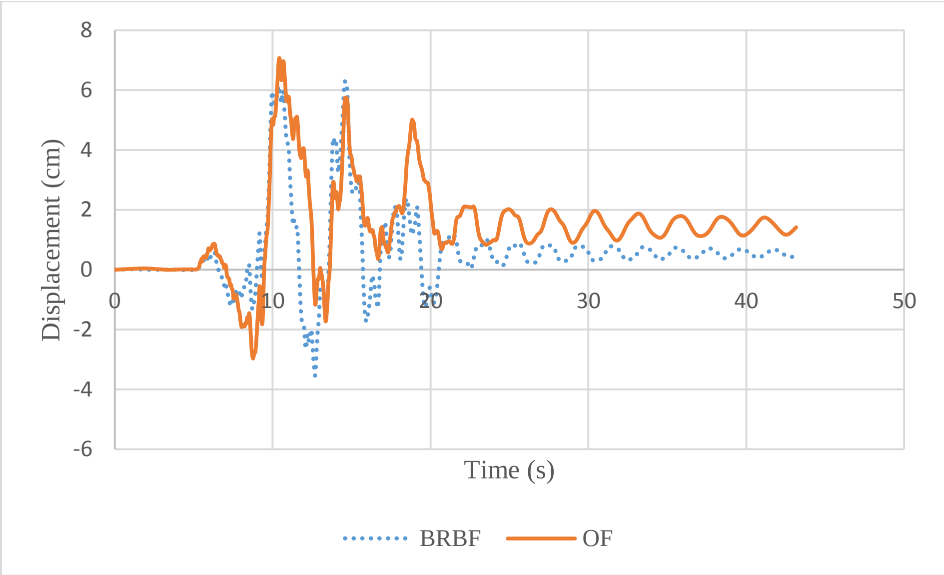


Figure 4.20 3 rd story displacement time history of 18 story OF and BRBF under Düzce earthquake

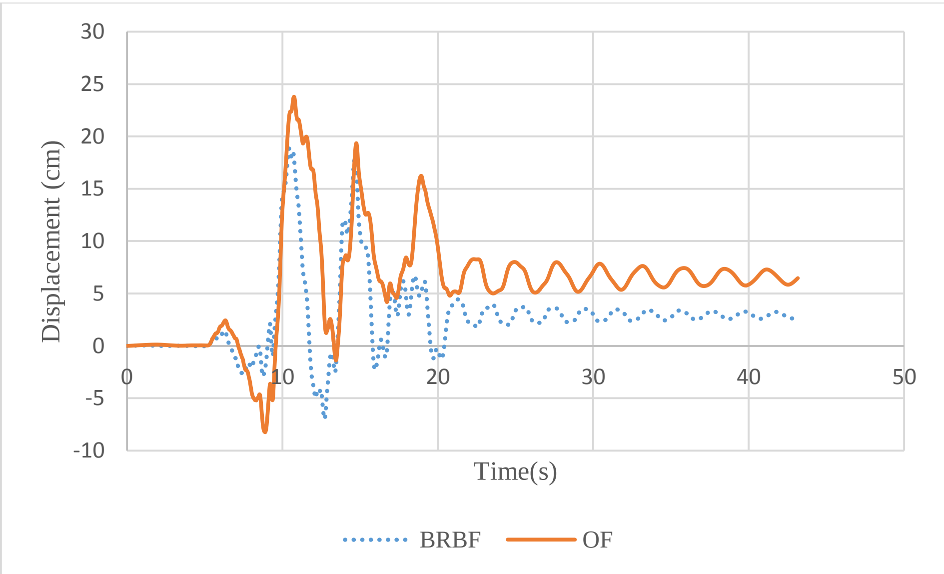


Figure 4.21 6 th story displacement time history of 18 story OF and BRBF under Düzce earthquake

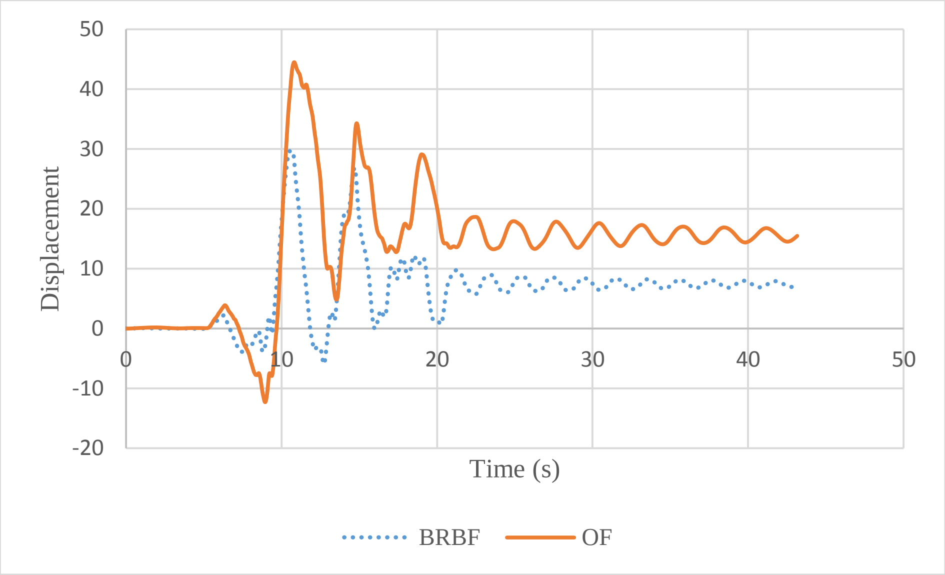


Figure 4.22 9 th story displacement time history of 18 story OF and BRBF under Düzce earthquake

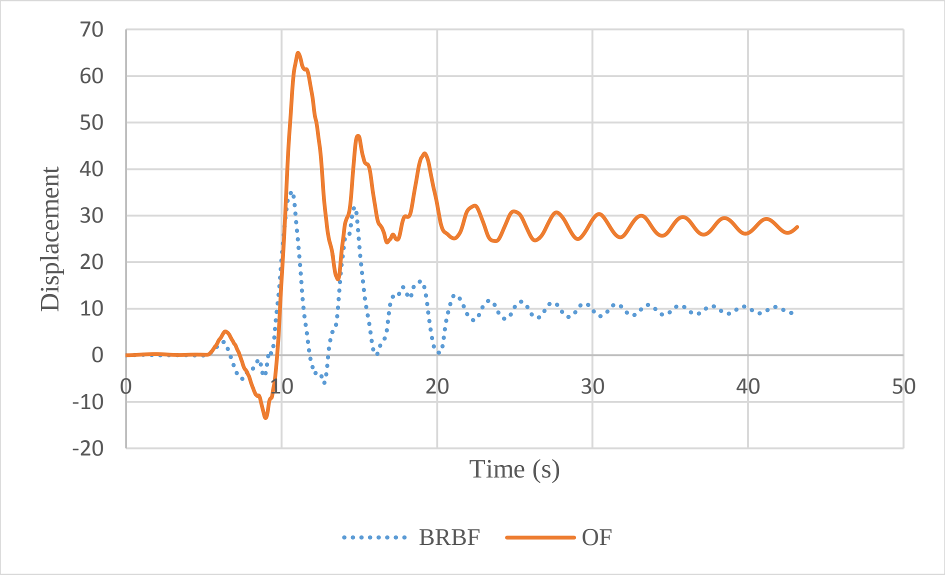


Figure 4.23 12 th story displacement time history of 18 story OF and BRBF under Düzce earthquake

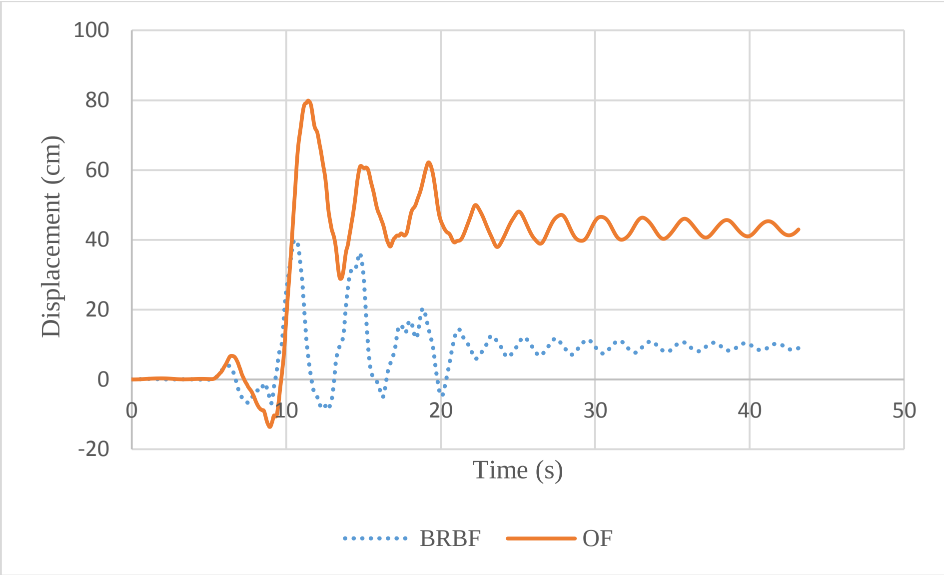


Figure 4.24 18 th story displacement time history of 18 story OF and BRBF under Düzce earthquake

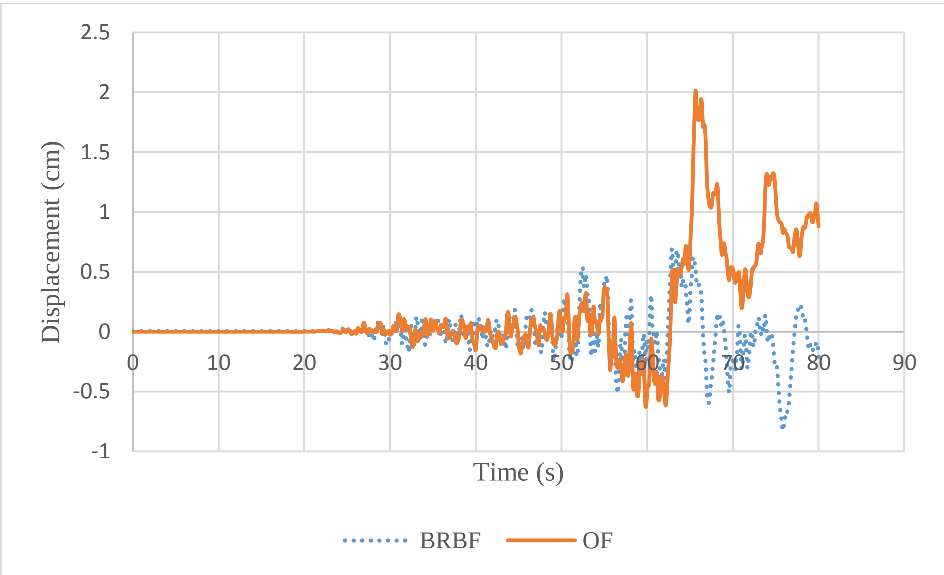


Figure 4.25 1 st story displacement time history of 18 story OF and BRBF under Hector Mine earthquake

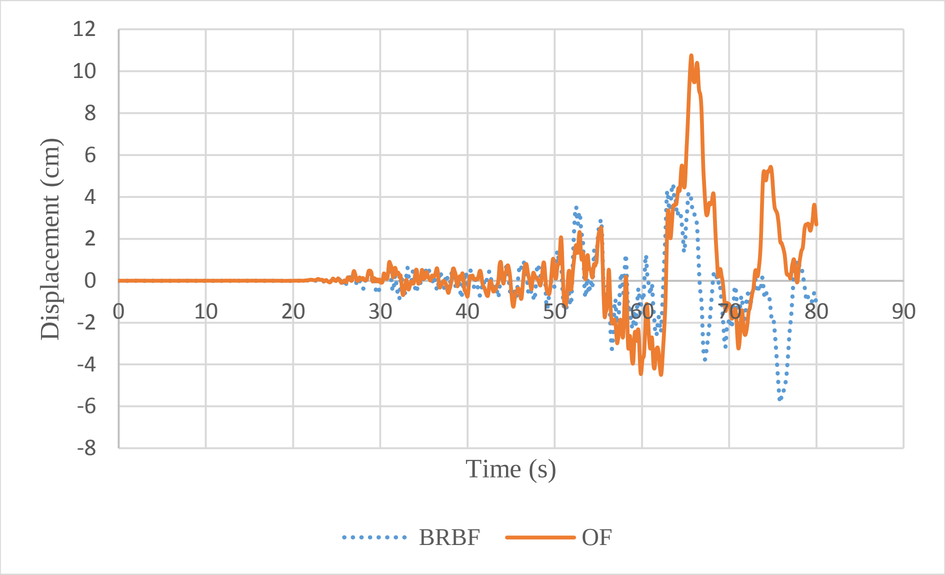


Figure 4.26 3 rd story displacement time history of 18 story OF and BRBF under Hector Mine earthquake

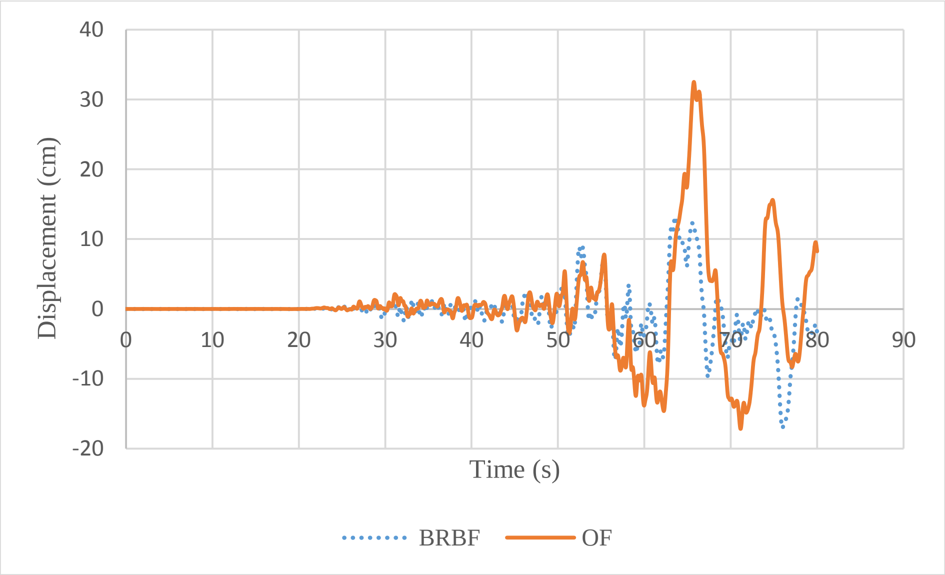


Figure 4.27 6 th story displacement time history of 18 story OF and BRBF under Hector Mine earthquake

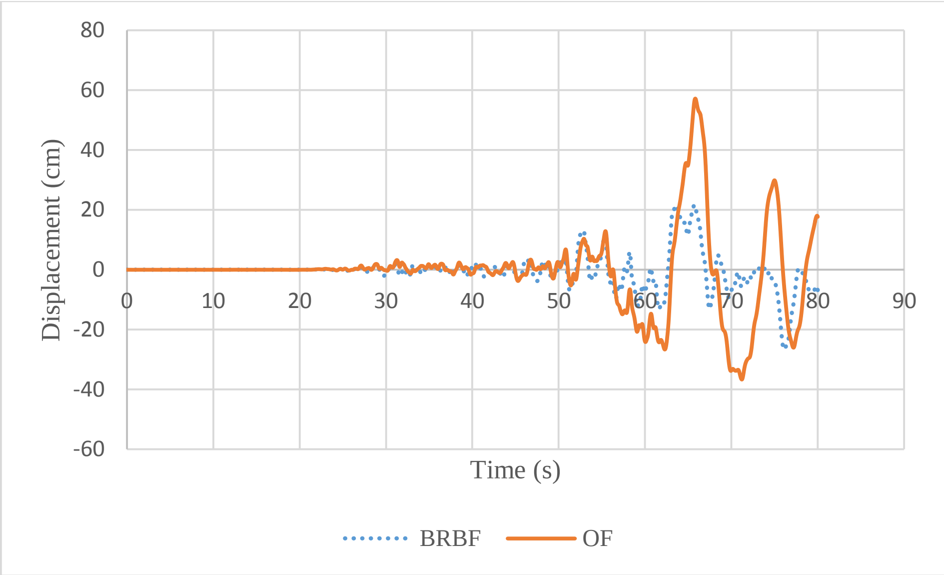


Figure 4.28 9 th story displacement time history of 18 story OF and BRBF under Hector Mine earthquake

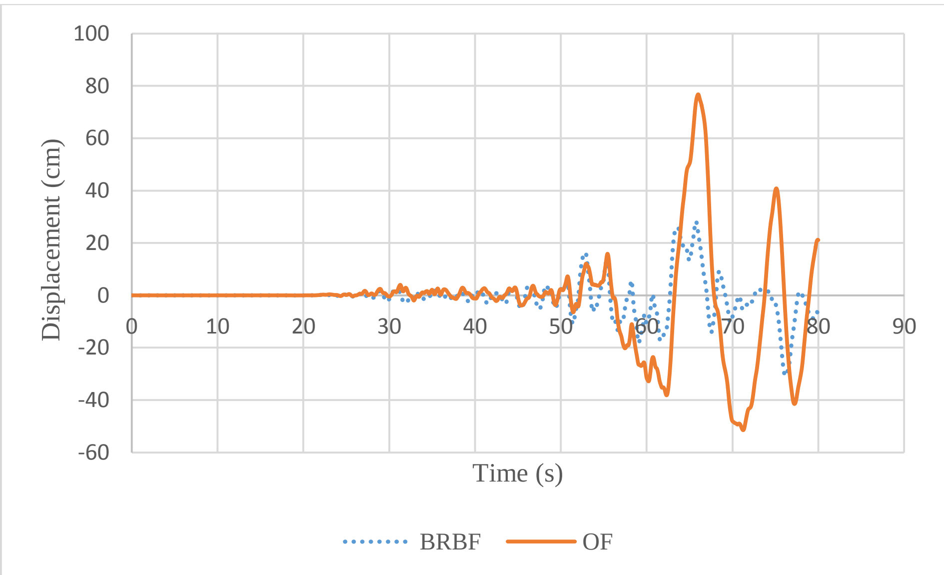


Figure 4.29 12 th story displacement time history of 18 story OF and BRBF under Hector Mine earthquake

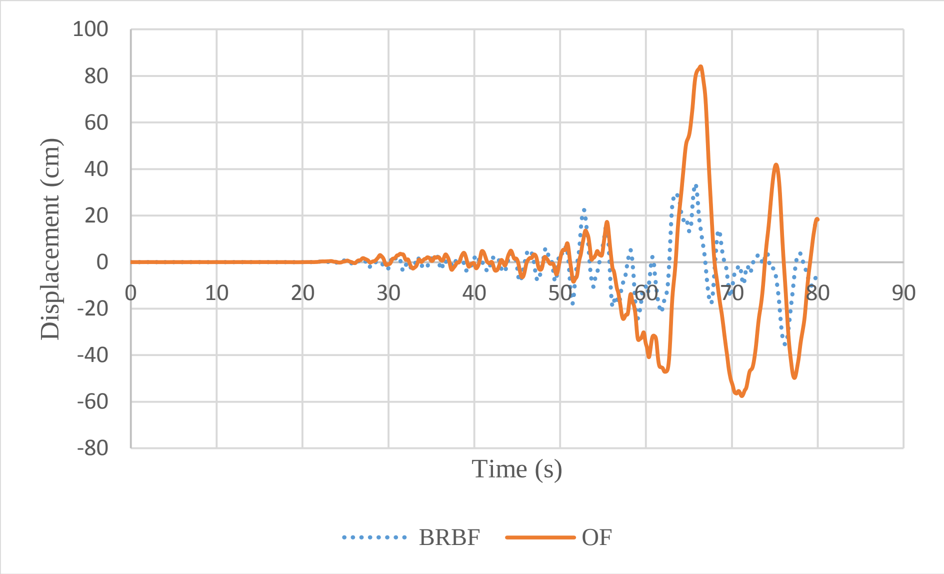


Figure 4.30 18 th story displacement time history of 18 story OF and BRBF under Hector Mine earthquake

4.3 Inter-story drift ratio

The plots for the maximum inter-story drift ratio observed at each story of the 9 and 18 story OFs and BRBFs under the earthquake accelerations are given in Figures 4.31 and 4.32, respectively. As it is seen from the plots, both 9 and 18 story BRBFs had significantly lower the maximum inter-story drift ratio than that of OFs. For example, under the Düzce earthquake, the maximum inter-story drift ratio of the 9 story OF was achieved as 2.64% while the maximum inter-story drift ratio of the BRBF was obtained as 0.68%. Similarly, the maximum inter-story drift ratio of the 18 story OF was achieved as 2.55% whereas the maximum inter-story drift ratio of the BRBF was obtained as 1.53%. Moreover, under the Hector Mine earthquake, the maximum inter-story drift ratio of the 9 story OF was achieved as 2.01% while the maximum inter-story drift ratio of the BRBF was obtained as 1.44%. In the same way, the maximum inter-story drift ratio of the 18 story OF was achieved as 2.92% while the maximum inter-story drift ratio of the BRBF was obtained as 1.34%. Furthermore, under Chi Chi earthquake, the maximum inter-story drift ratio of the 9 story OF was achieved as 2.79% while the maximum inter-story drift ratio of the BRBF was obtained as 1.11%. Likewise, the maximum inter-story drift ratio of the 18 story OF was achieved as 2.13% while the maximum inter-story drift ratio of the BRBF was obtained as 2.00%.

The comparison of the maximum inter-story drift ratio of the OFs and BRBFs also indicated that the latter had remarkably lower drift values than the former and the magnitude of the reduction in inter-story drift ratio demand varied depending on the number of stories and the earthquake acceleration.

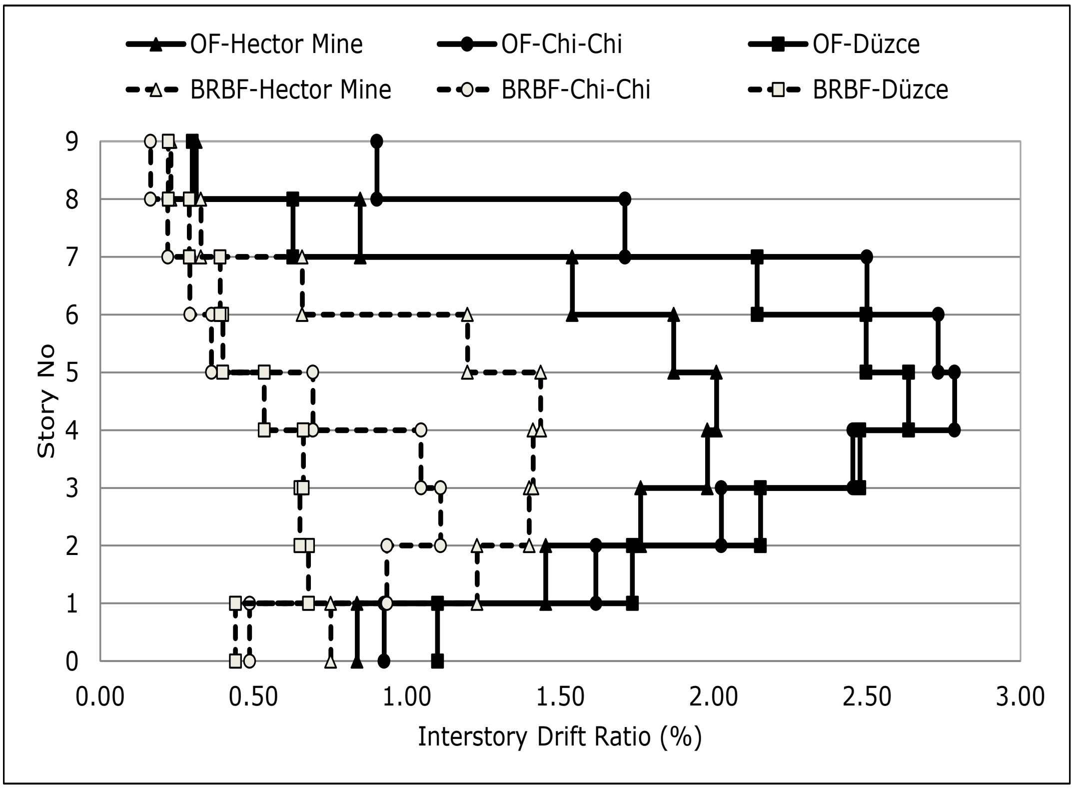


Figure 4.31 Variation of inter-story drift ratio for the 9 story RC structures having BRBs under different earthquakes

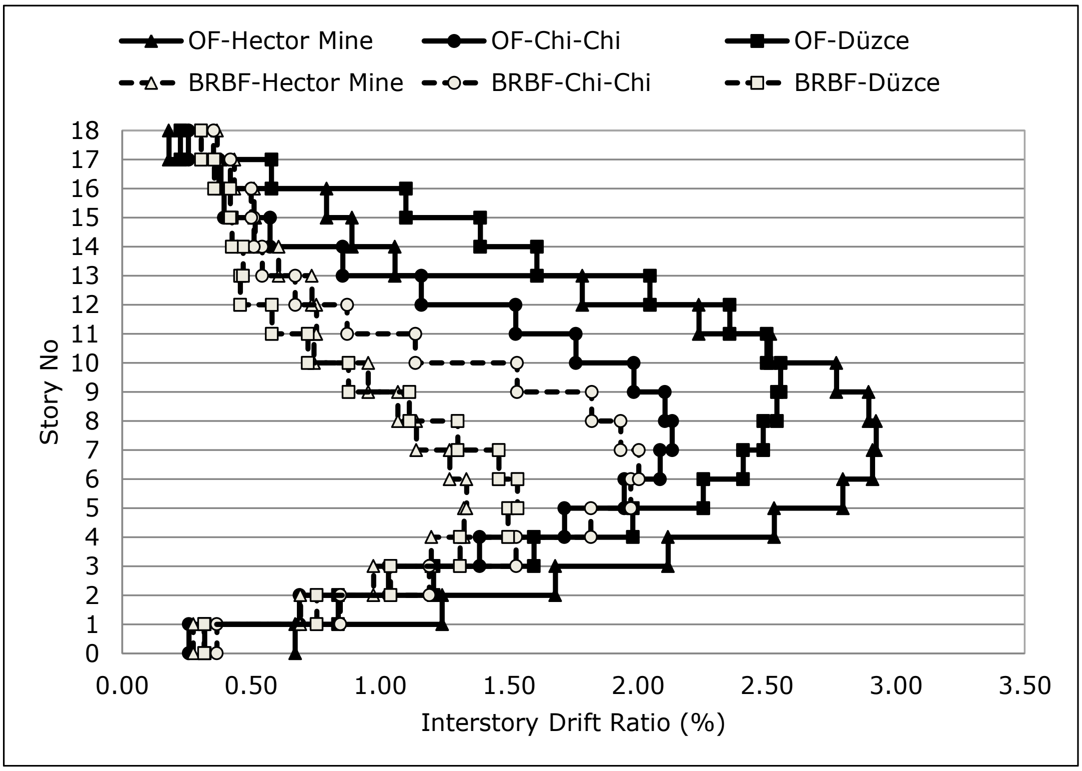


Figure 4.32 Variation of inter-story drift ratio for the 18 story RC structures having BRBs under different earthquakes

CHAPTER 5

CONCLUSIONS

In this study, an investigation was made on the seismic performance of the high-rise reinforced concrete framed structures equipped with and without diagonal buckling restrained braces. For this purpose, the nonlinear analysis was carried out on the existing and retrofitted structures having 9 and 18 stories. The three different natural ground motions generated as the spectrum compatible were employed. Based on the analysis of the results, the following conclusions were drawn:

- It was observed that the fundamental period of both 9 and 18 story structures reduced significantly with the inclusion of the buckling restrained braces. Thus, the stiffness of the system increased.
- From the analysis results, it was pointed out that the implementation of the buckling restrained braces improved the seismic performance of the seismically deficient reinforced concrete framed structures with regarding the interstory displacement and drift quantity.
- It was found that the use of buckling restrained braces for seismic upgrading brought up to 4.06 and 2.38 times reduction in the maximum roof displacement demand of 9 and 18 story structures, respectively.
- The results also showed that the buckling restrained braces were very effective in decreasing the maximum inter-story drift ratio. For example, depending upon the height of the structure and the earthquake record used, the maximum inter-story drift ratio for the original frames varied between 2.01% and 2.92% while that for the buckling restrained braced frames ranged from 0.68% to 2.00%.
- Furthermore, the use of the buckling restrained braces provided more uniform inter-story distribution in the retrofitted high-rise reinforced concrete structures. The aggregation of large drift was prevented with the application of such retrofitting solution.

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