SEISMIC BEHAVIOR OF BUCKLING RESTRAINED BRACED FRAMES

by

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ABSTRACT

SEISMIC BEHAVIOR OF BUCKLING RESTRAINED BRACED FRAMES

In this study the seismic performance of buckling restrained braced frames in steel structures in Turkey are addressed. A set of three (low rise), six (mid rise) and ten (high rise) buildings is assessed and compared. Buildings were designed and modeled in three different configurations as concentrically braced, eccentrically braced with short link and eccentrically braced with long link braced frames to investigate the response of structure under lateral loading. In order to have a good estimation about the behavior of the structures, two seismic design approaches namely “nonlinear static” and “nonlinear dynamic” according to the FEMA 356 is performed. The governing criterion for evaluation of the buildings is inter-story drift and maximum residual displacement. Comparison among three types of configurations in case of seismic behavior and their response is done to evaluate the most efficient configuration in low, mid and high rise buildings. For the case study, 3 types of steel structures in 3D modeled and designed in compliance with FEMA 356 provision implementing conventional braces to obtain the proper cross section needed for the buildings and in the second phase for simplification, one end bay of the structure as a 2D frame selected and the response of frame under nonlinear static and nonlinear dynamic procedures are examined to obtain the maximum axial force demand of the braces. In the next step, the behavior of the braces has changed according to research results and assigned for brace members and nonlinear static (pushover) and nonlinear dynamic analyses perform over the braced frames. Results compared to have a good understanding of the response of the three configurations in terms of maximum displacement and inter-story drift. The important point in response of the buckling restrained braced frame is that primary elements such as beam and columns to remain elastic during a seismic activity.

ÖZET

ULUSLARARASI MÜTEAHHİT FİRMALAR İÇİN BİR REKABETÇİLİK MODELİNİN GELİŞTİRİLMESİ

Bu araştırmada, burkulması önlenmiş olan çelik çaprazlı çerçevelerin sismik performansı Türkiye'de değerlendirilmiştir. Üç 3 (alçak), 6(orta) ve 10 (yüksek) farklı katlara sahip binalar değerlendirilip karşılaştırılmıştır. Binaların yanal yük uygulandığındaki davranışları, merkezi, kısa bağlantılı dış merkezi ve uzun bağlantılı dış merkezi çaprazlı üç farklı konfigurasyonda araştırılmıştır. Yapıların davranışını daha iyi değerlendirmek için FEMA 356'ya göre lineer olmayan istatik ve lineer olmayan dinamik gibi iki farkli dizayn yöntemi uygulanmıştır. Binaların durum değerlendirilmesinde en temel kriterler katlar arasında kayma ve kalıcı maksimum deplasmandır. Kısa, orta ve yüksek binalara uygulanan konfigurasyonlardan en verimlisini bulmak için, binaların sismik davranışları ve yanıtları karşılaştırıp değerlendirilmiştir. Örnek çalışma için, FEMA 356 yönetmeliği ve uygun kesiti elde etmek doğrultusunda 3 boyutlu 3 farklı çelik konveksiyonel çaprazli yapılar modellenip, dizayn edildi. İkinci aşamada sadeleştirmek için yapının bir uc çerçevesi 2 boyutlu olarak öngörulup çaprazlarin maksimum eksenel yüklerini bulmak amacıyla çerçevelerin davranışı lineer olmayan istatik ve lineer olmayan dinamik yükler altında elde edilmiştir. Bir sonraki adımda, çaprazların davranışı bir araştırmanın sonuçlarına göre değiştirilip çapraz elemanına tanıtılmış ve lineer olmayan istatik ve lineer olmayan dinamik prosedürler bu çaprazların üzerine uygulanmıştır. Çalışılan bu üç konfigurasyonun davranışlarını daha iyi anlayabilmek için sonuçlar karşılaştırılmıştır. Burkulmasi önlenmiş çaprazlı çerçevenin davranışındaki en önemli nokta, bir sismik aktivite esnasında binanın kolon ve kiriş gibi temel elemanlarının lineer kalmalarıdır. Bu doğrultuda döngüsel yük altında deprem enerjisinin maksimumunu akma yoluyla çaprazların çekmesi beklenmektedir.

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$A(T1)$	Spectral acceleration coefficient calculated for $T1$ period
A_0	Effective ground acceleration coefficient
A_{conn}	Area of the connection
A_{nysc}	Area of the steel core outside of the yielding region
A_{ysc}	Yielding steel core area
A_g	Effective buckling restrained brace cross section area
b	Section width
C	Base shear ratio
C_d	Seismic performance factor
C_e	Elastic base shear ratio
C_s	Design seismic load level factor
C_y	Yield shear ratio
E	Modulus of elasticity, earthquake load
F_a	Site coefficient as a function of site class
F_v	Site coefficient as a function of site class
F_y	Axial load due to the yield stresses on the brace
g	Dead load
g_i	Total dead load at i^{th} storey of building
H_i	Height of i^{th} storey of building measured from the top foundation level
I	Moment of inertia of the section, building importance factor
I_{sc}	Moment of inertia of steel casing
i	Radius of gyration
K	Lateral stiffness
K_{eff}	Effective stiffness of the buckling restrained brace
L_{conn}	Length of the connection
L_{nysc}	Length of non-yielding region of steel core
L_{ysc}	Length of the yielding region of the steel core
l	Total length of the brace

l_p	Length of plastic hinge
M	Lumped mass
M_p	Plastic moment capacity of the section
n	Live load participation factor
P	Normal force
P_e	Elastic buckling strength of the restricting encasing
P_n	Nominal compressive strength
P_y	Yield strength of the restrained yielding segment
P_{ysc}	Axial yield strength of the steel core
q	Live load
q_i	Total live load at i^{th} storey of building
R	Structural behavior factor
R_a	Seismic load reduction factor
$R_a(T_1)$	Seismic load reduction factor corresponding to $T1$ period
R_μ	Ductility reduction factor
r	Gyration radius, yielding core thickness
S_a	Spectral acceleration
S_d	Spectral displacement
S_{xs}	Response acceleration parameter
S_{x1}	Design response acceleration parameter at a one-second period
S_s	Mapped spectral acceleration at short-period
T	Structural period
T_1	First natural vibration period of building
V_e	Base shear in the elastic system
W	Seismic weight

β	Effective viscous damping
γ	Link rotation

ω	Angular natural frequency, strain hardening adjustment factor
Δ_e	Elastic displacement
Δ_{max}	Maximum displacement
Δ_y	Yielding displacement
δ	Axial deformation of the brace
μ	Ductility factor
λ	Slenderness ratio of steel columns
θ	Plastic rotation of the columns

LIST OF ACRONYMS/ABBREVIATIONS

$AISC$	American Institute for Steel Construction
$ASCE$	American Society of Civil Engineering
ASD	Allowable Stress Design
BRB	Buckling Restrained Brace
$BRBF$	Buckling Restrained Braced Frame
CBF	Concentrically Braced Frame
$FEMA$	Federal Emergency Management Agency
$LRFD$	Load Resisting Frame Design
$OCBF$	Ordinary Concentrically Braced Frame
RC	Reinforced Concrete
SAP	Structural Analysis Program
$SCBF$	Special Concentrically Braced Frame
$SDOF$	Single Degree of Freedom
$SLRS$	Seismic Load Resisting System
TEC	Turkish Earthquake Code
TDY	Turkish Earthquake Code

1. INTRODUCTION

1.1. General Considerations

Earthquake is one of the natural phenomena which affect directly the people's life in urban areas. In recent decades the effort for turning buildings to more reliable places for being safe in earthquake incidents is addressed by researchers and scientists to design buildings in such a manner which resist in ground motions with high probability of damage. Turkey is one of the countries which are prone to high seismic activities and in several years a lot of ground motions with different intensities happened. Over years, this country subjected to many life losses during strong earthquakes occurred. The inefficiency of buildings is one of the reasons for most of collapses in structures. Thus, the concern for strengthening buildings and modifying design codes rises in recent years among civil engineers and governmental institutes because of the tolls remains after each disaster.
The main reasons for exceeding of damage state and ultimately life losses are listed as below:

- Human error factor during the construction
- Structural deficiencies
- Unexpected factors related to the resisting ability of structures

In recent years, the risk of damage in urban areas is increasing in Turkey, mainly due to lack of enough infrastructures and services, the mounting rate of migrations to urban centers, and faulty land-use planning and construction, and environmental misuse. Several studies show that the vulnerability of Turkey's buildings is higher if it compared with California while both share roughly the same earthquake hazard level (Erdik 2007). Several reasons can be taken into account for this high vulnerability such as poor level of the quality in construction and development is the result of high inflation (leading to a limited mortgage and insurance market, a major impediment to large scale development and industrialization of the construction sector) uncontrolled urbanization which opens the demand for inexpensive housing, the lack of control or supervisory on the design and construction and regulations which are not applied by constructors in experience.

One of the systems used for decades as a resisting system is steel moment resisting frames. Steel moment-resisting frames are susceptible to large lateral displacements during severe earthquakes. As such, special attention is required in design to limit inter-story displacements so that potential problems due to geometric nonlinearities and brittle or ductile fracture of beam-to-column connections are mitigated and excessive damage to nonstructural elements is avoided.

Regarding many practical and economic issues related to conventional moment resisting systems, tend to use concentrically braced frames, as structural lateral load resisting system, increased by structural engineers. However, frequent damage to concentrically braced frames in past earthquakes, such as the 1985 Mexico, 1989 Loma Prieta, 1994 Northridge, and 1995 Hyogo-ken Nanbu earthquakes, has raised concerns about the ultimate deformation capacity of this class of structure (R. Sabelli *et al.*, 2003).

Several recent destructive earthquakes, particularly the 1994 Northridge earthquake in California, the 1995 Hanshin-Awaji (Kobe) earthquake in Japan and the 1999 Chi-Chi earthquake in Taiwan caused various levels of damage upon a large number of low-rise to medium-rise steel framed structures. The main reasons for occurrence of structural damages can be noted as insufficient lateral load carrying capacity, poor detailing of connections, and local buckling occurring in connections and/or brace members. These deficiencies raise the concern for serious investigations upon seismic design methods and the modifications need for optimizing the retrofit strategies to relieve existing steel buildings. Two retrofitting strategies emerged as being practical and efficient. The first one is to add new structural elements such as steel diagonal bracings providing the global stiffening and strengthening of the lateral resisting systems. The second one is to upgrade by selectively strengthening the deficient structural elements of the buildings including local modification of material properties and/or seismic details. Generally, the first method is preferred and lateral force resisting elements such as steel braces are prevalently used to increase the seismic strength of framed building structures (Guneyisi, 2011).

1.2. Literature Review

A very early study about the design and use of buckling restrained braces proposed by Watanabe *et al.*, 1988. They designed and fabricated a type of brace in which the brace can be has the required rigidity, yield strength without being subjected to buckling. Therefore, they used a buckling-resistant structural member consisting of a steel core members enclosed in a concrete-filled square steel tube. The design of the brace is in a way that the yielding load acting on the core member is smaller than the buckling load of steel tube so that the desired hysteretic behavior can be obtained.

Several investigations carried out by other researchers to testify the different aspects of the buckling restrained braces separately and the implementation of the brace mechanism into structural frames in earthquake resisting systems. Some of researchers conducted their experiments on the evaluation and verification of the brace's properties.

Mirtaheri *et al.*, (2011) put their effort on the effect of the length of the brace on the overall behavior of the BRB. The paper investigates the effectiveness of the core plate's length on the energy dissipation of the member. Furthermore, the BRB can be used as a damper if utilized as a structural fuse. Modified BRBs with shorter lengths can be considered as dampers. The test BRBs fabricated and tested in this study experimentally and analytically.

In order to satisfy the fatigue performance requirements of high performance seismic dampers to be used after level 2 earthquakes three times without replacing, Tsutomu *et al.*, (2011) carried out a low-cycle fatigue test to examine the fatigue life problems of BRBs. Constant and variable amplitude loadings tests and their results suggests that all of the specimens have good fatigue performance and the toe-finished method can effectively improve the fatigue performance of BRBs with relatively small strain amplitude.

Energy based design is one of the approaches used for validation the energy dissipation of BRBs. In a study conducted by Choi *et al.*, (2005) they applied the design procedure to three and eight story framed structure with buckling restrained braces. They utilized a series of twenty earthquake data to generate a design spectrum in order to validating the design procedure. According to analysis the mean values for the top story displacement correspond well with the given performance target displacement. They obtained a relatively uniform inter-story drift distributing over the height of the structure which is desired due to the uniform damage distribution.

A large scale study carried out by Clark *et al.*, (2004) to investigate the energy dissipation capacity of a type of BRB so called "Unbounded Brace" with different yielding forces subjected to a cyclic loading pattern widely used for testing steel beam-column connection. Additional tests performed to determine the behavior of the brace under a near-field ground motion data and a displacement history derived from a seismic analysis of an idealized 5-story building and low cycle fatigue test.

As a research study Kim *et al.*, (2004) utilized direct displacement design procedure for a frame with BRB composed of a hinge-connected main structure. In this study the braced system is designed to remain elastic during a seismic event and the BRB is the lateral load resisting element with the ability of dissipating dynamic energy through stable hysteretic response. Applicability of the design procedure is the ruling criteria of the displacement-based design procedure used in this paper. Coincidence of the maximum displacement and the target displacement is the confirming point of the time history analysis applied to a 3 and 5 story model structures.

The core plate local buckling is the key subject focused by Takeuchi *et al.*, (2010) research group. The flexural buckling is one of the concerning points to be prevented while designing procedure which is performed by the restraining tube wall thickness. In this study, cyclic loading and numerical analyses conducted on BRB members. Various configurations were considered for the tube thicknesses to figure out the influence of local buckling of the restraining mechanism on the overall behavior of the BRB in case of strength and ductility.

Another research conducted by Fahnestock *et al.*, (2007) to validate the predicted seismic response of BRBFs and their design methods with design provisions. In this framework the nonlinear dynamic analyses applied to a numerical simulation of the prototype BRBFs and the response of the structure were assessed. A good level of response is reported in terms of structural damage. The results suggest that the currently using deflection amplification factor is not reliable and underestimates the mean inelastic lateral displacement under design-level earthquakes and the system over strength factor may be unconservative. It is also shown that the currently using method for estimating the maximum ductility demand of the BRBs is unconservative.

The cyclic structural behavior of buckling restrained brace is studied by Lopez-almansa *et al.*, (2012). They presented a numerical model of the structural behavior of dissipative buckling restrained brace which is used as an alternative for conventional bracing systems. They modeled the core member and the mortar and exterior tube member in a analysis program. The cyclic behavior of the component is evaluated and is compared with the experimental results and a satisfactory agreement is obtained.

The overall behavior of all-steel buckling restrained braces is investigated in Hoveidae *et al.*, (2012) research. They performed a finite element analysis to testify the restraining mechanism of a set of all-steel buckling restrained braces with identical core member sizes. The study is conducted by a parametric study of different amount of gap between the core member and the restraining tube. The global behavior of the brace is the governing factor. The results show that the restraining member's flexural stiffness effects the global buckling of the brace member regardless of the gap amount. The Euler buckling load of the resisting member to the yielding strength of the core member controls the behavior.

The reliability of the assessment of buckling restrained braced frame examined by Blake *et al.* (2009). They used experimental limit-state model and stochastic dynamic analysis to evaluate the BRBF subjected to seismic loads. The efficiency of the risk of BRB failure due to low-cycle fatigue fracture of the BRB core member is the objective of study. The probability of brace failure assessed regarding the effect of seismic loading, characteristics of the BRB and BRBF.

Blake *et al.*, (2009) developed ductility capacity models for buckling restrained braces. The concern about the BRBs high ductility capacity and the lack of accepted method for predicting the cumulative plastic ductility of the brace led to carry out the research. The statistical framework and the past experimental results are used for this study. The research outcome is a model with sufficient accuracy for estimating the BRB failure.

A study done by Pierson *et al.*, (2010) on a high rise building to obtain the seismic response of the structure in a high earthquake risk region like Los Angeles. Nonlinear response analysis done on three various 40 story structure to compare the seismic demand of buckling restrained braced frames implemented in these structures. The design approaches used in this study are "code based design", "performance based design" and "performance based design plus". Spectrum-matched and simulated ground motions are utilized in this research. The design performed based on safe inter-story drift ratio. The obtained inter-story drift ratio is higher for simulated ground motion.

Ductility demand estimation of BRBF under earthquake loading is the study carried out by Fahnestock *et al.*, (2003) which is one of the important criteria in performance-based design method. An analytical study conducted on a one bay 4 story BRBF with material and geometrical nonlinearity. Nonlinear static pushover and time history analyses are used for purpose of this study. The resultant demands are compared with the anticipated capacities. The demand results are substantially higher than those anticipated in code provisions. These results suggest that the code provisions anticipated capacities need to be modified.

A research done by Mahmoudi *et al.*, (2013) shows an effort for determination of the response modification factors used for design of buckling restrained braced frames.

The authors try to evaluate the modification factors of BRBs used in retrofitting of steel structures. The results suggest that the BRBF's modification factor for some extent is higher than the conventional braced frames and also the number of braced bays plays a vital role in the determination of response modification factor.

A nonlinear time history analysis performed by Balling *et al.*, (2009) on BRBF to develop design curves and obtain the cross sectional areas for a multi-story building's braces and compare the results with a common equivalent lateral force design approach. Interestingly the results show that the derived cross sections variations go nearly linear from story to story which is not predicted by the common design method.

A number of researchers utilized the BRBs for retrofitting the reinforced concrete buildings. D' Aniello *et al.*, (2008) conducted the experimental tests of a RC building retrofitting with BRBs. Because of the large inelastic capacities shown in BRBs the group considered them as a good alternative to reduce the seismic vulnerability in an as-built RC building. Two type of only-steel BRBs with different cross section shapes are used in this study and their results evaluated.

Another research done by Di Sarno *et al.*, (2010) in which a non-ductile RC framed building retrofitted by the use of BRBs. In order to validate the efficiency of BRBs nonlinear static pushover and nonlinear dynamic response history analyses performed on both as-built and retrofitted structures. A set of code-reliant ground motions applied to the structures and their responses are evaluated based on several limit states insisted in code provisions. As a result the local and global displacements were reduced significantly and an elongation of the natural period of the structure occurred. There was an enhancement in the ductility and energy dissipation of the retrofitted structure.

Some of the researchers put their effort on the evaluation of different configurations of the BRBFs. Sabelli *et al.*, (2003) conducted an investigation on the seismic demands of BRBFs incased in a steel structure. The focus of the study is on the response of a three and six story concentrically braced frames with implemented BRBs.

A set of ground motions applied to examine the response of the structure through nonlinear dynamic analysis and the mechanical and statistical characteristic aspects were evaluated regarding to the benefits of using BRBFs in steel structures.

Additional to aforementioned paper another study carried out by Prinz *et al.*, (2012) focusing on the "seismic performance of buckling-restrained braced frames with eccentric configurations". Some architectural problems lead to use of braced frames in eccentric configurations. In this case there are some limitations due to the link-to-column connection requirements. In this study the performance and economy of the BRBF in eccentric shape compared with conventional eccentrically braced frames. Results show that the BRBFs have greater residual drifts but less susceptible to failure at link-to-column connection than conventional configuration.

1.3. Scope and Objective

In this study the seismic behavior of buckling restrained braced frames in three different configurations are examined according to FEMA 356 provision. The performance based design approach is performed and the response of braced frames is evaluated to compare various configurations. The focus of this study is on the performance level of different configurations of BRBs and to determine the most reliable one with ductile manner which lets the structure primary elements to remain elastic. The aim of the study is to model and analyze a structure with three different BRBF configurations and perform nonlinear static (pushover) and nonlinear dynamic (time-history) procedures to evaluate the structural performance and the BRBF's contribution to the behavior of the structure. To have a good estimation about the performance of different configurations, detailed study process is explained step by step.

1.4. Thesis Outline

In this study, the first step is to briefly discuss the investigations done earlier to introduce and verify the beneficial aspects, such as mechanical and characteristic properties, of the BRBs and framed implemented by BRBs. Later, the seismic behavior of the BRBFs is addressed by the researchers. The retrofitting of old-build steel and RC buildings by using BRBFs is the other subject focused by papers.

In the second step, the general consideration about the design of steel structures is addressed and also the performance levels in which the structural design have to be comply with are explained.

In the third step, detailed requirements for buckling restrained braced frames are discussed according to the American Institute for Steel Construction code AISC seismic provision 2005. The design methods and the limitations for design will be expressed in this section.

In the forth step, the focus is on the concentrically braced frames design and configuration considerations. The benefits and the weaknesses of this kind of configurations are taken into account.

In the fifth step as well as the previous section, eccentrically braced frames and the differences between short link and long link and the link behavior are discussed in detail.

In the sixth step the detailed analysis and the basic requirements for design and analysis are discussed based on the TEC 2007 and FEMA 356 provision codes for steel structures and seismic design procedure. The prototype structures are modeled and for simplify the design process the structures considered as symmetric one with no geometry and property nonlinearities. Applied loads and load combinations are in agreement with the AISC load requirements.

At the final step of the study nonlinear static push over analysis has been done on the models for controlling that buildings supply required performance level, life safety. The nonlinear application of the SAP 2000 V 15.0.0 structural analysis program is used. Push over analysis has been done according to FEMA 356 coefficient method and base shear vs. displacement of the structures has been selected as scale of comparing.

2. DETAILED ASSESSMENT STUDY

The Seismic Provisions for Structural Steel Buildings, hereinafter referred to as these Provisions, shall govern the design, fabrication and erection of structural steel members and connections in the seismic load resisting systems (SLRS) and splices in columns that are not part of the SLRS, in buildings and other structures, where other structures are defined as those structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral load-resisting-elements. These Provisions shall apply when the seismic response modification coefficient, R, (as specified in the applicable building code) is taken greater than 3, regardless of the seismic design category. When the seismic response modification coefficient, R, is taken as 3 or less, the structure is not required to satisfy these Provisions, unless specifically required by the applicable building code. These Provisions shall be applied in conjunction with the AISC Specification for Structural Steel Buildings, hereinafter referred to as the Specification. Members and connections of the SLRS shall satisfy the requirements of the applicable building code, the Specification, and these Provisions.

2.1. Building Target Performance Levels (FEMA 356)

Building performance is a combination of the performance of both structural and nonstructural components. Building performance in FEMA 356 is expressed in terms of target Building Performance Levels. These target Building Performance Levels are discrete damage states selected from among the infinite spectrum of possible damage states that buildings could experience during an earthquake. The particular damage states identified as target Building Performance Levels in FEMA 356 have been selected because they have readily identifiable consequences associated with the post-earthquake disposition of the building that are meaningful to the building community. These include the ability to resume normal functions within the building, the advisability of post earthquake occupancy, and the risk to life safety. Due to inherent uncertainties in prediction of ground motion and analytical prediction of building performance, some variation in actual performance should be expected.

2.1.1. Structural Performance Levels and Ranges (FEMA 356)

The Structural Performance Level of a building shall be selected from four discrete Structural Performance Levels and two intermediate Structural Performance Ranges defined in this section. The discrete Structural Performance Levels are Immediate Occupancy (S-1), Life Safety (S-3), Collapse Prevention (S-5), and Not Considered (S-6). Design procedures and acceptance criteria corresponding to these Structural Performance Levels would be as specified in Chapters 4 through 10. The intermediate Structural Performance Ranges are the Damage Control Range (S-2) and the Limited Safety Range (S-4). Acceptance criteria for performance within the Damage Control Structural Performance Range shall be obtained by interpolating the acceptance criteria provided for the Immediate Occupancy and Life Safety Structural Performance Levels. Acceptance criteria for performance within the Limited Safety Structural Performance Range shall be obtained by interpolating the acceptance criteria provided for the Life Safety and Collapse Prevention Structural Performance Levels.

2.1.2. Life Safety Structural Performance for Braced Steel Frames (FEMA 356) level (S-3)

Structural Performance Level S-3, Life Safety, shall be defined as the post-earthquake damage state that includes damage to structural components but retains a margin against onset of partial or total collapse in compliance with the acceptance criteria specified in FEMA 356 for this Structural Performance Level.

Structural Performance Level S-3, Life Safety, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

2.1.3. Seismic Hazard

Seismic hazard due to ground shaking shall be based on the location of the building with respect to causative faults, the regional and site-specific geologic characteristics, and a selected Earthquake Hazard Level. Seismic hazard due to ground shaking shall be defined as acceleration response spectra or acceleration time-histories on either a probabilistic or deterministic basis.

2.1.4. Response Acceleration Parameters

The design short-period response acceleration parameter,S_{xs} and design response acceleration parameter at a one-second period,$S_x 1$ for the BSE-2 Earthquake Hazard Level shall be determined using values of S_s and S_1 taken from approved MCE spectral response acceleration contour maps and modified for site class.

2.1.5. Probabilities of Exceedance Greater than 10% Per 50 Years

For probabilities of exceedance greater than 10% per 50 years and when the mapped short-period response acceleration parameter,S_1 is less than 1.5g, the modified mapped short-period response acceleration parameter,S_x and modified mapped response acceleration parameter at a one-second period, S_1 shall be determined.

2.1.6. Adjustment for Site Class

The design short-period spectral response acceleration parameter, S_{xs} and the design spectral response acceleration parameter at one-second, S_{x1} shall be obtained respectively as follows:

$$S_{xs} = F_a S_s \tag{2.1}$$

$$S_{X1} = F_v S_1 \tag{2.2}$$

Where, F_a and F_v are site coefficients determined respectively from Table 2.3 and Table 2.4, based on the site class and the values of the response acceleration parameters S_S and S_1 for the selected return period.

Table 2.1. Fa Values Dependent on Site Class (FEMA 356).

values of Fa as a function of site class and mapped short-period spectral response acceleration S_s					
site class	mapped spectral acceleration at short-periods				
	$S_s \leq 0.25$	S_s=0.5	S_s=0.75	S_s=1	$S_s \geq$1.25
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.2	1.2	1.1	1	1
D	1.6	1.4	1.2	1.1	1
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

Table 2.2. Fv Values for Dependent on Site Class (FEMA 356).

values of Fv as a function of site class and mapped spectral response acceleration at one-second period S_1					
site class	mapped spectral acceleration at short-periods				
	$S_1 \leq 0.1$	S_1=0.2	S_1=0.3	S_1=0.4	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.2	1.2	1.1	1	1
D	1.6	1.4	1.2	1.1	1
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

2.2. General Response Spectrum

A general response spectrum shall be developed according to specifications denoted in FEMA 356 provisions. Regarding this provisions the general horizontal response spectrum shall be developed using following equations to determine spectral response acceleration, S_a versus structural period, T in horizontal direction.

$$S_a = \begin{cases} S_{XS}\left[\left(\frac{5}{B}-2\right)\frac{T}{T_S}+0.4\right] for 0 < T < T_S \\ {S_{XS}}/{B_S} for T < T_S \\ {S_{X1}}/{(B_1 T)} for T > T_S \end{cases} \tag{2.3}$$

Where T and T_S are given by

$$T_S = {S_{X1} B_S}/{(S_{XS} B_1)} \tag{2.4}$$

$$T_0 = 0.2 T_S \tag{2.5}$$

Use of spectral acceleration obtained in Equation 2.3 in the extreme short-period range ($T < T_0$) shall only be permitted in dynamic analysis procedures and only for modes other than the fundamental mode. B_S and B_1 are taken from the following table:

Table 2.3. Damping Coefficients (FEMA 356).

damping coefficients BS and B1 as a function of effective damping B		
effective viscous damping B	BS	B1
≤ 2	0.8	0.8
5	1	1
10	1.3	1.2
20	1.8	1.5
30	2.3	1.7
40	2.7	1.9
≥ 50	3	2

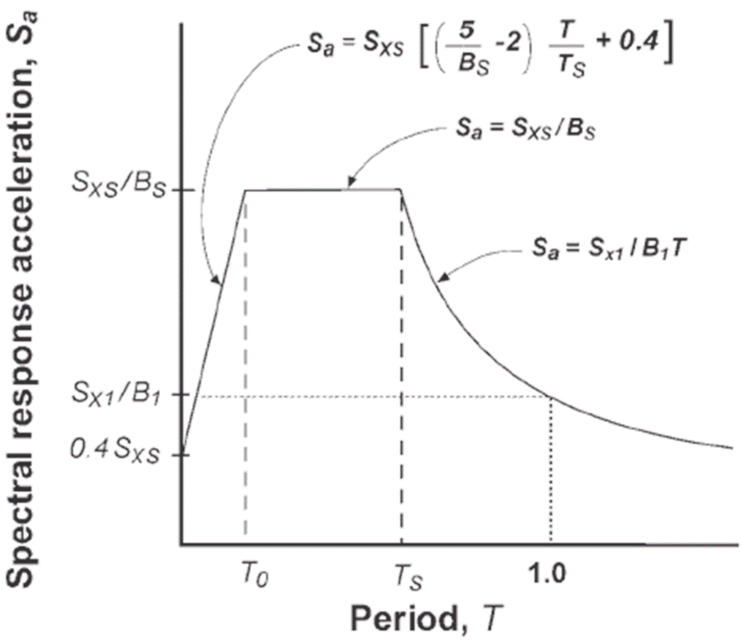

Figure 2.1. FEMA 356 Response Spectrum Curve.

2.2.1. Damping Ratios

Response spectra shall be developed for an effective viscous damping ratio of 5% of critical damping and for other damping ratios appropriate to the indicated structural behavior as mentioned in previous section.

2.2.2. Minimum Spectral Amplitude

The 5% damped site specific spectral amplitudes in the period range of greatest significance to the structural response shall not be specified less than 70 % of the spectral amplitudes of the general response spectrum.

2.2.3. Zones of Seismicity

The zone of seismicity shall be defined as High, Moderate or Low. The definition of this classification is as the follow:

- Zones of high seismicity
- Zones of moderate seismicity
- Zones of low seismicity

3. SEISMIC DESIGN PHILOSOPHY

Designers after utilizing computers for analysis procedure used elastic design method. However, seismic designers realized that allowing structures to respond in the inelastic range was beneficial and most often unavoidable. When properly designed, plastic mechanisms would form and dissipate energy imparted by the earthquake ground motion to the structure.

Modern seismic design codes are based on decades of research and field observation after earthquakes. Although such codes to date do not explicitly use the plastic design methods, a key fundamental concept of these codes is the need for ductility and ductile plastic mechanism. The seismic design community goes one step further by incorporating the capacity design concept in parallel with ductility design. Those two highlighted concepts are the basis for seismic design of structures.

3.1. Elastic Response and Response Spectrum

To study seismic effect on a structure, it is first assumed that a one story frame can idealized as a single-degree-of-freedom (SDOF) system where K is the lateral stiffness and M is the lumped mass tributary to the roof level. The angular natural frequency, ω, and the natural period, T, of the structure are:

$$\omega = \sqrt{{}^{M}\!/_{K}} \tag{3.1}$$

$$T = \frac{2\pi}{\omega} = 2\pi\sqrt{{}^{K}\!/_{M}} \tag{3.2}$$

It is also assumed that this system has a 5% equivalent various damping ratio. With applying a sample earthquake ground motion the elastic response of the frame can be obtained using structural dynamic theory (Chopra 2007). The resultant maximum displacement is defined as the spectral displacement, $S_d(T)$ at period T. To design the structure it is necessary to know the maximum force in the member. From the Hook's law, the maximum structural force or base shear,V_e in the elastic system is:

$$V_e\left(T\right) = K\Delta_{max} = KS_d\left(T\right) \tag{3.3}$$

Together with what discussed above:

$$V_e\left(T\right) = M\omega^2 S_d\left(T\right) \tag{3.4}$$

Defining the pseudo-acceleration,$S_a(T)$, as the following:

$$S_a\left(T\right) = \omega^2 S_d\left(T\right) \tag{3.5}$$

and accordingly, the displacement response becomes:

$$V_e\left(T\right) = MS_a\left(T\right) = W\left(\frac{S_a\left(T\right)}{g}\right) \tag{3.6}$$

where W is the reactive weight of the system.

3.2. Inelastic Response and Ductility Reduction

The structure in order to remain elastic a portion of the reactive weight should be considered as the base reaction:

$$V = C.W \tag{3.7}$$

In which C is the base shear ratio and then C_e represents elastic base shear ratio. This required force level corresponds to point A in Figure 3.1 where the elastic response is shown in dashed line O-A. Generally it is not economical to design a structure to remain elastic during a strong earthquake. If an effort is made to ensure that the structure possesses ductility, the required base shear force can be significantly reduced. In such a case the expected elasto-plastic response is shown as O-B-C. C_y is the yield shear ratio of the frame. The ductility factor, μ is defining as:

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{3.8}$$

where Δ_u and Δ_y are the maximum and yielding displacements, respectively. The reduced yield base shear can be expressed as:

$$C_y = \frac{C_e}{R_\mu} \tag{3.9}$$

where R_μ is defined as the ductility reduction factor.

By varying the period of the structure, the constant-ductility R_μ spectrum can be constructed. Newmark and Hall (1982) proposed a very simple ductility reduction rule for SDOF systems. In the moderate and long period range (i.e. in the velocity and displacement amplification regions of the response spectrum), it was observed that the maximum displacement of the elastic and inelastic systems are about the same. This observation leads to the following:

$$R_\mu = \mu \tag{3.10}$$

This corresponds to the so-called equal displacement rule. The above relationship can be easily derived from Figure 3.1 by assuming the inelastic displacement, Δ_u is equal to elastic displacement, Δ_e. In the short period range (i.e. in the acceleration amplification region of the response spectrum), the relationship is closer to:

$$R_\mu = \sqrt{2\mu - 1} \tag{3.11}$$

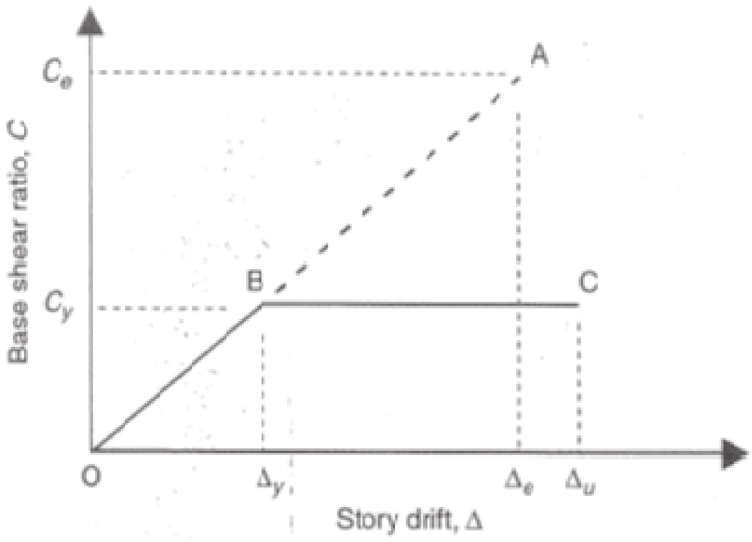

Figure 3.1. Definition of Ductility Factor and Ductility Reduction Factor for an SDOF System (Bruneau, Uang and Sabelli, 2012).

This corresponds to the so-called equal energy rule. The above equation can be derived from Figure 3.1 by equating the areas (i.e. energies) under both the O-A and O-B-C response curves. For structures with very short period, Newmark and Hall also observed that R_μ=1, that is, ductility is ineffective in reducing the elastic seismic force. However, this would only be the case for structure very short in height and/or extremely stiff; the majority of building structures do not fall in this period range.

3.3. Collapse Mechanism Versus Yield Mechanism

In plastic analysis and design, the term "collapse mechanism" is used to describe a state beyond which the structure has reached its capacity to carry monotonically increasing, static or dynamic load and becomes unstable. The term "collapse" is appropriate when the load is monotonically applied in one direction. However, this definition is not applicable for earthquake loading because seismic response is cyclic and transient in nature. This can be demonstrated for the inelastic response of the one-story frame presented earlier. Because the structure is designed and, thus, allowed to yield, a mechanism starts to form once plastic hinges from both ends of the columns. Upon load reversal, the columns will respond elastically again before plastic moment is reached in the reverse direction and plastic rotation starts to accumulate again. It is through this "on" and "off" process that the earthquake energy is dissipated by plastic deformation in the structure. One of the most major goals of seismic design is to maximize the energy dissipation while controlling the damage. Therefore the term "yield" or "plastic" mechanism, rather than "collapse" mechanism, is more appropriate to describe the seismic response of structures.

3.4. Design Earthquake

According to what discussed so far, the inelastic dynamic analysis is require for accurate seismic response prediction and design. However, it is not applicable practically for routine design methods for two main reasons:

- Due to many factors affecting the earthquake load like the characteristic and the magnitude and the rupture of the earthquake, it is not possible to determine the time history of the ground motion.
- The shape of the response spectrum corresponding to a specific ground motion it is impossible to incorporate the same time-history for analysis purpose and seismic codes are usually provide uniform hazard level to exceeding in a number of years.

The international building code (ICC 2009) references to ASCE 7 for the construction of the elastic design spectra. ASCE 7 first specifies the spectral acceleration values for a maximum considered earthquake probability of 2% in 50 years. Two third of these records are considered to use with the probability of 10% in 50 years. A brief summary of the design procedure to construct the spectrum is as the follow:

(i) Determine the site ground motion
(ii) Determine site or soil classification
(iii) Adjust mapped maximum considered earthquake spectral acceleration for the site effect.
(iv) Determine design spectral acceleration

3.5. Physical Meaning of Seismic Performance Factors (ASCE 7)

Factors considered in ASCE 7 are the response modification factor, R, and C_d and Ω_0 and are named seismic performance factors. These factors greatly simplify the design process. The physical meaning of ductility reduction factor of R_μ, for a SDOF system was discussed earlier in this chapter. Although both R and R_μ are using for reducing the elastic seismic forces, the physical meaning of them is different. The R_μ factor is defined for an SDOF system where the inelastic behavior can be considered as an elasto-perfectly plastic response. To apply for design of more redundant structures such as multistory frames, however, the redundancy of the structure may lead to yield progressively before the ultimate strength of the structure is reached. The physical meaning of the seismic performance factors in ASCE 7 is shown in the following.

Figure 3.2 shows the typical response envelope of a structure, which is used for explaining the R-factor seismic design procedure. Based on the fundamental period of the structure, the elastic design base shear C_e is calculated. According to ASCE 7 the C_e factor reduces to a design seismic load level factor C_s by a reduction factor R. point S represents the first significant yielding point, beyond which the structure will respond inelastically. In other words, under lateral loads, a structure designed based on this reduced seismic force level first responds elastically, then shows an inelastic response while the lateral force increases beyond that level. At this stage the redundancy in the structure causes to form a series of plastic hinges, resulting in the yielding mechanism at the strength level C_y. If the actual structural response curve can be idealized by an elasto-perfectly plastic curve, the system ductility factor μ_s can be defined as:

$$\mu_s = \frac{\Delta_u}{\Delta_y} \tag{3.12}$$

Then the system ductility reduction factor R_μ can be defined as:

$$R_\mu = \frac{C_e}{C_y} \tag{3.13}$$

The strength between the yielding point C_y and the significant yield level C_s is so-called system over strength factor Ω_0:

$$\Omega_0 = \frac{C_y}{C_s} \tag{3.14}$$

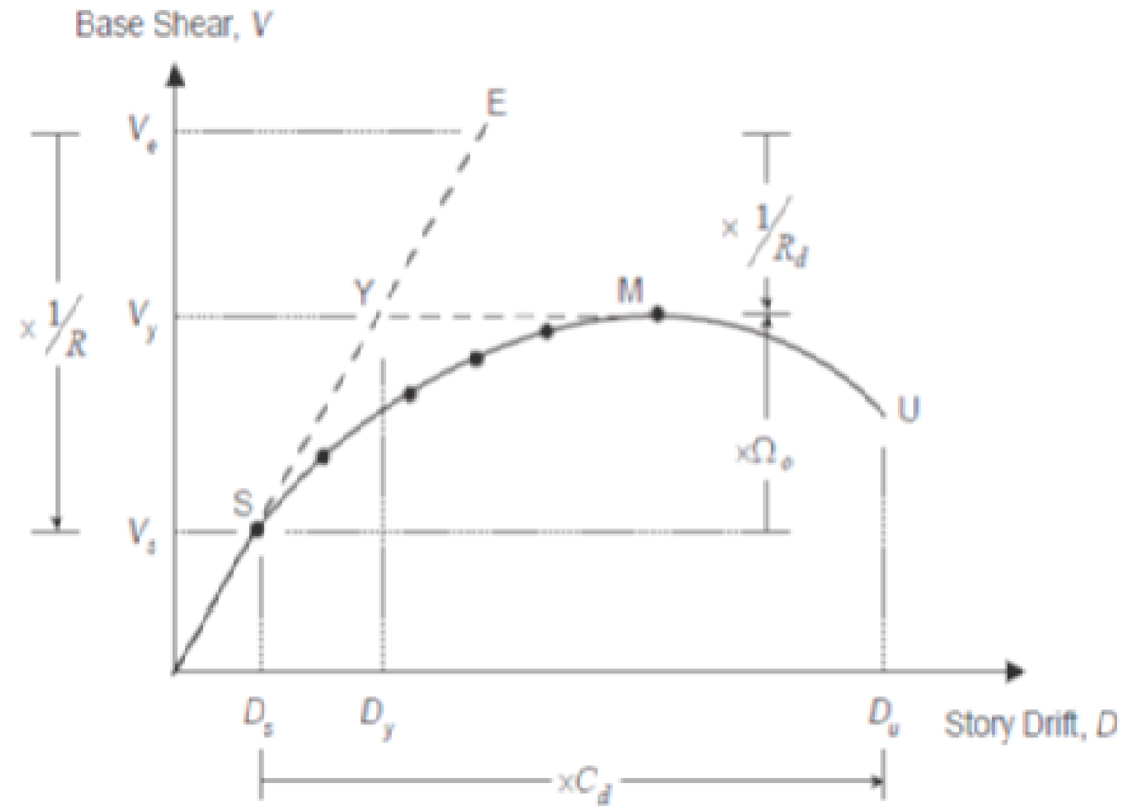

Figure 3.2. Seismic Performance Factors (ASCE 7).

4. CONCENTRICALLY BRACED FRAMES

4.1. General Behavior and Plastic Mechanism

As described in previous chapters, what is needed for seismic design is to take forces considerably smaller than those that would have to be considered to achieve full elastic response during the seismic activity. In order to have such behavior the structure elements should consider such that some elements yield in forces less than elastic response to allow other elements to remain elastic and show ductile response.

During earthquake, CBFs are expected to yield and dissipate energy through post buckling hysteretic behavior of their bracing members. The one direction drift is obtained when the brace members buckle in compression and subsequently yield in tension. The same action is possible when the forces act in reverse direction and the yielded member will buckle in compression. Figure 4.1 shows a schematic of CBF inelastic behavior

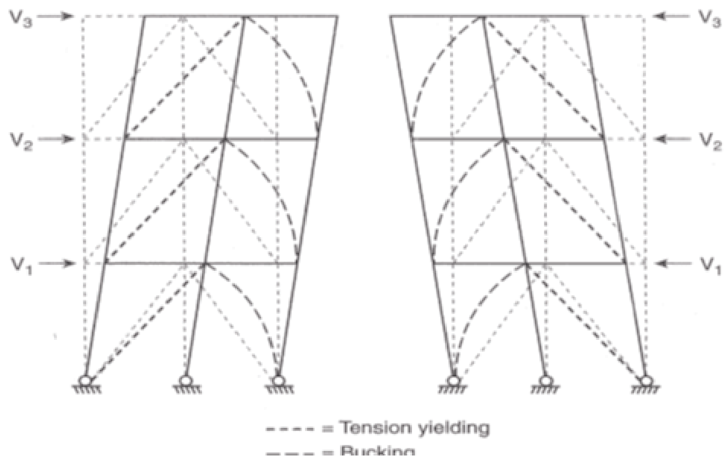

Figure 4.1. Schematic Inelastic Behavior of CBF (Bruneau, Uang and Sabelli, 2012).

For having this type of behavior, special ductile detailing is needed. During past earthquakes substantial damages observed due to insufficient detailing in design procedures. A good understanding of characteristics of the cyclic behavior of CBFs is the governing factor of capacity design principles in the design of CBFs to perform desirable ductile behavior.

4.1.1. Design Considerations

To have adequate earthquake resistance, CBFs must be designed in a way that is capable to demonstrate sufficient strength and ductile response.

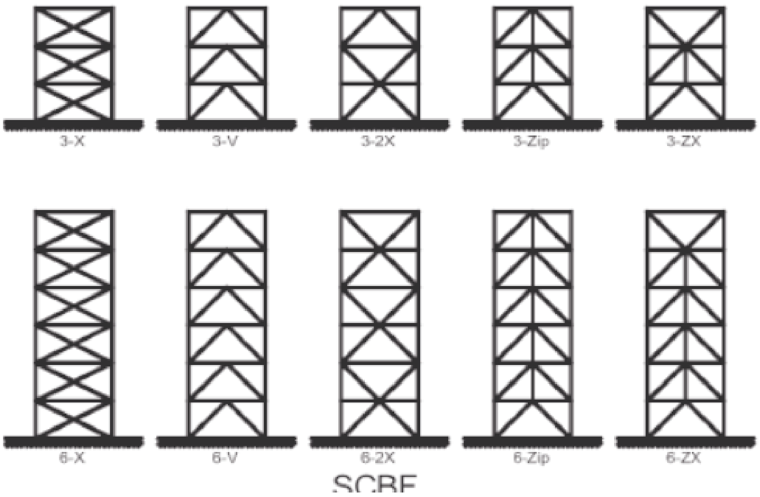

Figure 4.2. Typical Specially Concentric Brace Frame Configurations (Sabelli *et al.*, 2003).

The more recent thinking on these matters recognizes the importance of delaying low-cycle fatigue at the plastic hinge location, and allows using more slender brace members that correspondingly have lower ductility demands in compression, relying proportionally more on tension yielding of the braces to dissipate seismic energy. Although advantages this approach has, the most ductile CBFs have consistently been assigned structural response modification factor R on the order of 75% of the maximum value assigned to special moment resisting frames.

This penalty is attributed mainly as a consequence of the less ideal energy dissipation provided by compression brace, the observed pinching of the hysteretic curves of the braced frame due to the strength degradation of the compression brace, and the absence of effective strength hardening as typically occurs in moment frames.

Two types of CBF systems are permitted by design code namely, Special Concentric Braced Frames (SCBFs) and the Ordinary Concentrically Braced Frames (OCBFs). The focus of this study is on SCBFs which are designed for stable inelastic performance and energy dissipation capability, and correspondingly for the largest force reduction factor, R. Figure 4.1 shows some typical SCBFs which are investigated by researchers.

4.2. Hysteretic Behavior of Single Braces

Hysteretic behavior of single brace by defining brace's physical inelastic cyclic behavior can be expressed truly.

4.2.1. Brace Physical Inelastic Cyclic Behavior

An understanding of the physical inelastic behavior of an individual brace member subjected to reversed cycles of axial loading is necessary to design ductile braced frames using design concepts provided in provisions.

The behavior of axially loaded members is commonly expressed in terms of the axial load P, axial deformation δ, and transverse displacement at midlength Δ. According to convention, tension forces and deformations are taken as positive and compression forces and deformations as negative. Figure 4.3 shows a sample hysteresis loop curve of a brace under cyclic loading.

Regarding to the curve, the behavior of the single brace while the acting load increases, is to deflect until the formation of the plastic hinge. This behavior goes on until the failure of the plastic hinge.

4.3. Hysteretic Behavior and Design of Concentrically Braced Frames

Capacity design is needed for designing concentrically braced frames for the purpose of this study in order to have a desirable response in cyclic loadings.

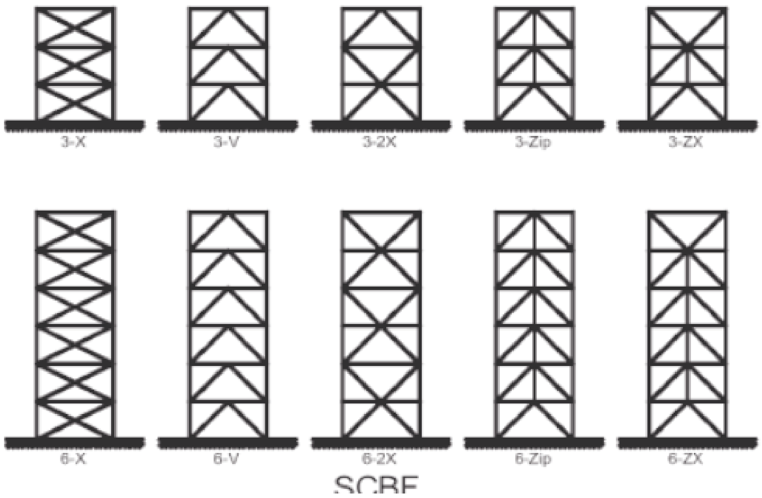

Figure 4.3. Sample Hysteresis of A Brace Under Cyclic Axial Loading (Bruneau, Uang and Sabelli, 2012).

4.3.1. Capacity Design and General Issues

According to most of the design approaches, a successful design regardless to the configuration, must recognize and account for load redistribution after buckling of the brace in compression and yielding in tension and losing compression strength upon larger drifts and during cyclic loadings. In the absence of such a resisting systems and capacity designs, the structure will witness an undesirable behavior and eventually buckling of structural elements such as beams and columns and buckling of these primary elements or a failure in connections. Although most of the code requirements are meeting by any frame configurations, many codes and standards still treat V and inverted V-braced frames with separate requirements in recognition of their unique characteristics as a result of the unbalanced forces applied to their beams during brace yielding and buckling.

Braces are the first elements designed in concentrically braced frames. The forces considered for their design are typically calculated through an elastic analysis, but their behavior is completely nonlinear due to their yielding and buckling during analysis process. Braces are the members designed as energy dissipating mechanisms for this type of structural systems. Yielding and buckling of braces typically occurs at drifts of

0.3 to 0.5% and post buckling axial deformation of braces can reach 20 times their yield deformation. During design procedure the brace strength is the dominant factor in the impact of forces on other structural components. These components are expected and designed to remain elastic during a seismic activity.

Note that the nonlinear analysis urges to predict the concentration of damage in one level of braced frames. However, in real-life experiments it is not applicable because of the effect of beam-to-column connections on the overall behavior of the structure and spreading of yielding and buckling along building height. As a result, minimum in design of this type of structural frames is to provide the continuity of columns along the height of the building.

4.4. Code Design Requirements for SCBFs (AISC, Seismic Provisions)

First thing to be considered is the member's properties which should be checked according to specified code.

4.4.1. Members

One of the criteria for checking the properties of the member in steel structures is the member's slenderness especially for the elements under the axial loading.

4.4.1.1. Slenderness. Bracing members shall have $Kl/r \leq 4\sqrt{E/F_y}$
Exception: Braces with $4\sqrt{E/F_y} < Kl/r \leq 200$ are permitted in frames in which the available strength of the column is at least equal to the maximum load transferred to the column considering R_y (LRFD) or (1/1.5) R_y (ASD), as appropriate, times the nominal strengths of the connecting brace elements of the building. Column forces need not exceed those determined by inelastic analysis, nor the maximum load effects that can be developed by the system.

4.4.1.2. Required Strength. Where the effective net area of bracing members is less than the gross area, the required tensile strength of the brace based upon the limit state of fracture in the net section shall be greater than the lesser of the following:

(i) The expected yield strength, in tension, of the bracing member, determined as $R_y F_y A_g$(LRFD) or $R_y F_y A_g/1.5$ (ASD), as appropriate.
(ii) The maximum load effect, indicated by analysis that can be transferred to the brace by the system.

4.5. Lateral force Distribution

Along any line of bracing, braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30 % but no more than 70 % of the total horizontal force along that line is resisted by braces in tension, unless the available strength of each brace in compression is larger than the required strength resulting from the application of the appropriate load combinations stipulated by the applicable building code including the amplified seismic load. For the purposes of this provision, a line of bracing is defined as a single line or parallel lines with a plan offset of 10 % or less of the building dimension perpendicular to the line of bracing.

4.6. Required Strength of Bracing Members

4.6.1. Required Tensile Strength

The required tensile strength of bracing connections (including beam- to-column connections if part of the bracing system) shall be the lesser of the following:

(i) The expected yield strength, in tension, of the bracing member, determined as $R_y F_y A_g$(LRFD) or $R_y F_y A_g/1.5$ (ASD), as appropriate.
(ii) The maximum load effect, indicated by analysis that can be transferred to the brace by the system.

4.6.2. Required Flexural Strength

The required flexural strength of bracing connections shall be equal to 1.1 $R_y M_p$ (LRFD) or (1.1/1.5) $R_y M_p$ (ASD), as appropriate, of the brace about the critical buckling axis.

4.6.3. Required Compressive Strength

Bracing connections shall be designed for a required compressive strength based on buckling limit states that is at least equal to 1.1 $R_y P_n$ (LRFD) or (1.1/1.5) $R_y P_n$ (ASD), as appropriate, where P_n is the nominal compressive strength of the brace.

4.7. Special Bracing Configuration Requirements

As one of the best configurations, V-type and inverted V-type braces are used in this study.

4.7.1. V-Type and Inverted V-Type Bracing

V-type and inverted V-type SCBF shall meet the following requirements:

(i) The required strength of beams intersected by braces, their connections, and supporting members shall be determined based on the load combinations of the applicable building code assuming that the braces provide no support for dead and live loads. For load combinations that includes earthquake effects, the earthquake effect, E, on the beam shall be determined as follows:
   (i) The forces in all braces in tension shall be assumed to be equal to $R_y F_y A_g g$
   (ii) The forces in all adjoining braces in compression shall be assumed to be equal to 0.3 P_n.

(ii) Beams shall be continuous between columns. Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = L_{pd}$. Lateral braces shall meet the requirements of $M_r = M_u = R_y Z F_y$(LRFD) or $M_r = M_a = R_y Z F_y/1.5$

(ASD), as appropriate, of the beam and C_d=1.0.

As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) bracing, unless the beam has sufficient out-of plane strength and stiffness to ensure stability between adjacent brace points

5. ECCENTRICALLY BRACED FRAMES

5.1. Design Consideration

For eccentrically braced frames the behavior of the system and plastic mechanism is crucial.

5.1.1. General Behavior and Plastic Mechanism

In eccentrically braced frames the axial forces incorporated in braces transfer to either columns or beams through small section of beams as shear and bending forces. Typical EBF geometries are shown in Figure 5.1. These types of bracing systems are suitable for architectural considerations. The critical beam section is called “link” and is designated by a length, *e*, in the figure. In EBF systems links act as structural fuses to dissipate earthquake energy in a building in a stable manner.

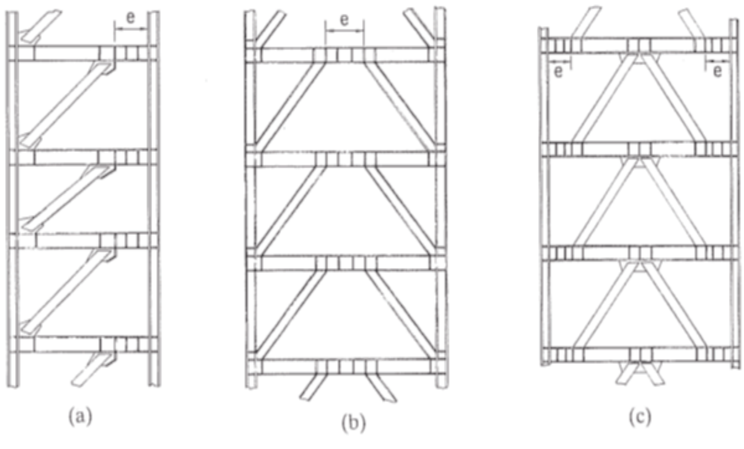

Figure 5.1. Typical EBF Geometries (Popov *et al.*, 1988).

Figure 5.2. Example of EBF Application (SEAOC 2000).

As shown in Figure 5.2 the beam-to-column moment connections are vulnerable to brittle fracture. Experiments show that the link-to-column moment connections are subjected to both high moment and high shear that makes them more vulnerable to brittle fracture. For this reason, it is important to avoid configurations with link-to-column connections.

5.1.2. Design Philosophy

The desirable behavior of the EBFs is to occurrence of plastic mechanism and yield in beam's link segments under cyclic loading. Thus, the remaining parts of the structure are designed to remain elastic during the event. In the case of comparison between SCBF and EBF systems plastic mechanisms, in former, braces are designed and detailed as structural fuses. In the latter, however, links need to be properly designed and detailed to have adequate strength and ductility. Other structural components in EBFs should be designed to remain elastic during design earthquake.
EBF systems are more efficient in seismic design because of their outstanding features in:

- High stiffness
- Excellent ductility

- More energy dissipation capacity

The governing point in EBFs design is the beam link segment behavior. In order to have proper design the key point is to restrict the inelastic behavior of the link. The most important part of the design is to achieve large demands of plastic deformation in beam link members.

5.2. Code Design Considerations for EBFs (AISC, Seismic Provisions)

Eccentrically braced frames are considered as good resisting systems in earthquake prone areas.

5.2.1. Scope

Eccentrically braced frames (EBFs) are expected to withstand significant inelastic deformations in the links when subjected to the forces resulting from the motions of the design earthquake. The diagonal braces, columns, and beam segments outside of the links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain hardened links, except where permitted in this Section. In buildings exceeding five stories in height, the upper story of an EBF system is permitted to be designed as an OCBF or a SCBF and still be considered to be part of an EBF system for the purposes of determining system factors in the applicable building code. EBF shall meet the requirements in this Section.

5.2.2. Links

Links are the most important parts of the eccentrically braced frames.

5.2.2.1. Limitations. According to AISC 7 design code links shall be seismically compact. The web of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted.

<u>5.2.2.2. Shear Strength.</u> Except as limited below, the link design shear strength, $\O_\nu v_N$, and the allowable shear strength, v_N/Ω_ν, according to the limit state of shear yielding shall be determined as follows:
V_n nominal shear strength of the link, equal to the lesser of

$$V_p or \frac{2M_p}{e}, kips\,(N) \tag{5.1}$$

$$\phi_v = 0.90\,(LRFD) \tag{5.2}$$

$$\Omega_v = 1.67\,(ASD) \tag{5.3}$$

where is the

$$M_p = F_y Z, kip - in\,(N - mm) \tag{5.4}$$

is the

$$V_p = 0.6 F_y A_w, kips\,(N) \tag{5.5}$$

where is the e link length, in (mm),

$$A_w = (d - 2t_f)\,t_w \tag{5.6}$$

The effect of axial force on the link available shear strength need not be considered if:

$$P_u \leq 0.15 P_y\,(LRFD) \tag{5.7}$$

$$P_a \leq (0.15/1.5)\,P_y\,(ASD) \tag{5.8}$$

where is the P_u required axial strength using LRFD load combinations,kips (N), is the P_a required axial strength using ASD load combinations,kips (N), is the P_y nominal axial yield strength= $f_y A_g$, kips (N)

$$P_u > 0.15P_y\,(LRFD) \tag{5.9}$$

$$P_a > (0.15/1.5)\,P_y\,(ASD) \tag{5.10}$$

as appropriate, the following additional requirements shall be met:

(i) The available shear strength of the link shall be the lesser of:

$$\emptyset_v V_{pa} and 2\phi_v M_{pa}/e(LRFD) \tag{5.11}$$

or

$$V_{pa}/\Omega_{\mathrm{v}} and 2(M_{pa}/e)/\Omega_v(ASD) \tag{5.12}$$

as appropriate, where

$$\phi_v = 0.90\,(LRFD)\,\Omega_v = 1.67\,(ASD) \tag{5.13}$$

$$V_{pa} = V_p\sqrt{1-(P_r/P_c)^2} \tag{5.14}$$

$$M_{pa} = 1.18M_p\left[1-(P_r/P_c)\right] \tag{5.15}$$

$$P_r = P_u \, (LRFD) \, or P_a \, (ASD) \tag{5.16}$$

(ii) The length of the link shall not exceed:

$$[1.15 - 0.5\rho'(A_w/(A_g))]1.6M_p/V_p \tag{5.17}$$

when nor

$$\rho'(A_w/A_g) \geq 0.3 \tag{5.18}$$

$$1.6M_p/V_p when \rho'(A_w/A_g) < 0.3 \tag{5.19}$$

where is the $A_w = (d - 2t_f) \, t_w$, is the $\rho' = P_r/V_r$ and where is the $V_r = V_u \, (LRFD)$ or $V_a(ASD)$, as appropriate, is the v_u Required shear strength based on LRFD load combinations, kips, is the V_a Required shear strength based on ASD load combinations, kips.

5.2.3. Diagonal Brace

The required combined axial and flexural strength of the diagonal brace shall be determined based on load combinations stipulated by the applicable building code. For load combinations including seismic effects, a load $Q1$ shall be substituted for the term E, where $Q1$ is defined as the axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link R_yV_n, where V_n is as defined in previous section. The available strength of the diagonal brace shall comply as designated in Specifications.

6. BUCKLING RESTRAINED BRACED FRAMES (BRBFs)

Compared with moment resisting frames as well as concentrically or eccentrically braced frames, buckling restrained braced frames is a relatively new system for seismic applications. First developed in Japan in the 1970s, through further research this system gained rapid acceptance after the 1994 Northridge Earthquake and became codified in the United States in a relatively short time.

Watanabe *et al.*, (1988) insisted that in a frame fabricated by incorporating in rigid frame structure, slender braces have a good behavior under tensile forces unless the joint break. However, the same property is not applied for compressive forces and the lateral deformations occur easily and they cannot bear compressive forces. For a frame with slenderness between high and small, it is difficult to design in which the energy absorption by plastic deformation is expected, because buckling is of an embrittling nature especially under compressive forces. Against this deficiency the researchers under Watanabe's supervision designed a brace that has a stable forth-deflection characteristic and enables compressive yield strength to be considered equal to tensile yield strength in order to further materialize the desired characteristics needed to mitigate the problem. In this brace, a core steel member with a rectangular section is restrained at its ends by the concrete encased in a steel tube insulate from frame to restrain buckling and coating materials are used between the concrete and the core member to prevent the transmission of axial forces to the concrete by friction.

They carried out a research on various specimens to testify their behavior under cyclic loads laterally acting on the member and the results were relatively desired such that the buckling did not occur and much energy absorbed. The other outstanding result was the stable hysteresis curve corresponding to the core member and the core member was yielded when the yielding load was reached.

Specially designed and detailed concentrically braced frames can provide high elastic stiffness to limit story drift, relying on diagonal braces to buckle and yield to dissipate energy. Although design specifications, such as AISC 341 (AISC 2010) and the CSA S16 (CSA 2009) provide requirements to limit the slenderness ratio and width-thickness ratio of the brace to ensure sufficient ductility, brace buckling is inevitable. Therefore, brace buckling leads to both strength and stiffness degradation, which can in some instances cause concentration of damage to a limited number of stories and, hence, increases the potential for building collapse.

The disadvantages of the CBF system can be overcome if the brace can yield in both tension and compression without buckling. Figure 6.1 shows the difference between the conventional and BRB type brace systems' behavior under cyclic loading. A braced frame that incorporates this type of brace, (Buckling Restrained Brace) is called buckling-restrained braced frame (BRBF).

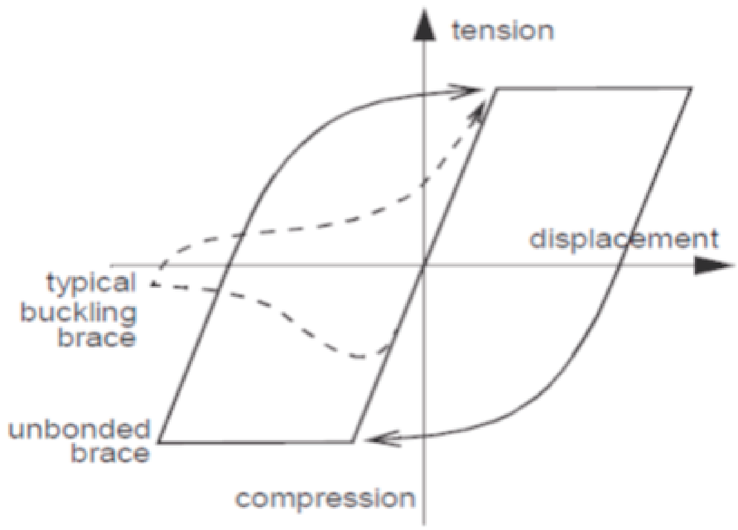

Figure 6.1. Behavior of Conventional Brace Versus BRB (Clark *et al.*, 1999).

Some of outstanding properties of BRBFs which gives priority over moment resisting frames are cited as follow (Shuhaibar *et al.*, 2002):

(i) Compared with moment frames, BRBFs exhibit high elastic lateral stiffness at low-level seismic input motions, making it easy to satisfy code drift requirements.

(ii) BRBFs eliminate the undesirable buckling of conventional CBFs by yielding in both tension and compression, thereby providing larger and stable energy dissipation at high level seismic input motions.

(iii) BRBFs provide economical installation through a bolted or pinned connection to gusset plates, which eliminates costly field welding and inspection.

(iv) Braces act as replaceable structural fuse, which minimizes damage to other elements and it is possible to replace damaged braces after major seismic events.

(v) BRBFs offer flexibility because both the strength and stiffness of the braces can be easily tuned. Furthermore, it is easy to model the cyclic behavior of BRBs for inelastic analysis.

(vi) For seismic rehabilitation, BRBFs can be more advantageous than the conventional bracing system because capacity design provisions for the latter system may require expensive foundation and floor diaphragm strengthening.

BRBFs may have some disadvantages:

(i) Most BRBs are proprietary.

(ii) If not properly controlled, steels commonly used to fabricate restrained yielding segment may have a wide range of yield strength.

(iii) Field erection tolerances are generally lower than those of conventional braced frames.

(iv) Large permanent deformation may occur under high levels of seismic input because this kind of system, like many others, does not have a recentering mechanism.

(v) Criteria for detecting and replacing damaged braces need to be established.

6.1. Components of Buckling Restrained Brace

Since many of the potential performance difficulties associated with conventional braced frames rise from the difference between tensile and compressive capacity of the brace and the degradation of brace capacity under compressive and cyclic loading, considerable research has been devoted to development of braces which exhibit more ideal elasto-plastic behavior. One means of achieving this ideal behavior through metallic yielding, where buckling in compression is restrained by an external mechanism. A number of approaches to accomplish this have been suggested including enclosing a ductile metal (usually steel) core (rectangular or cruciform plates, circular rods, etc.) in a continuous concrete filled steel tube, a steel tube with intermittent stiffening fins, and so on. Figure 6.2 shows a typical buckling restrained brace component and different shapes for steel core members.

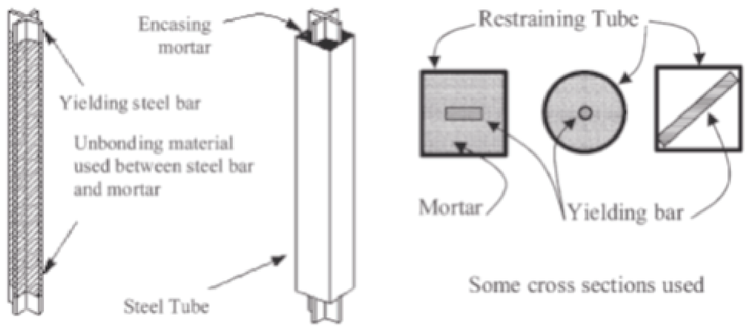

Figure 6.2. Some Typical Details for Buckling Restrained Braces (Sabelli *et al.*, 2003).

The assembly is detailed so that the central core member can deform longitudinally independent from the restraining mechanism in the case of lateral and local buckling. Through appropriate selection of the strength of the strength of the material, and the areas and lengths of the portions of the core that are expected to remain elastic and to yield a wide range of brace stiffness and strength can be attained. Since lateral and local buckling behavior modes are restrained, large inelastic capacities are attainable.

The inelastic cyclic behavior of several types of buckling restrained braces reported. These tests typically result in hysteretic loops having nearly ideal bilinear hysteretic shapes, with moderate kinematic and isotropic hardening evident. Figure 6.3 shows an example of the hysteresis loop corresponding to one of the frame works done.

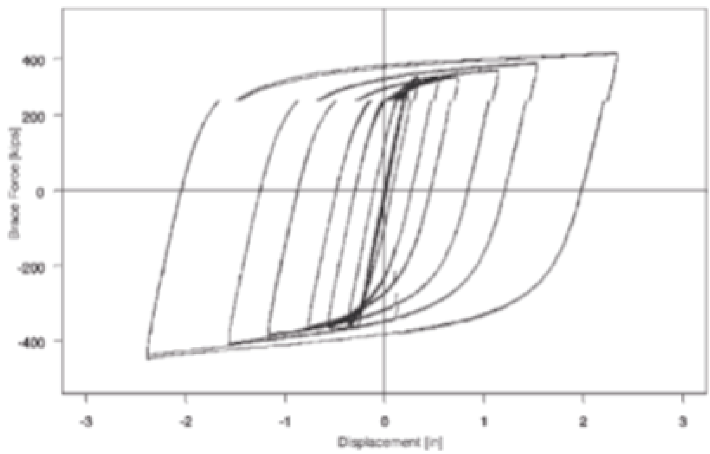

Figure 6.3. Axial Force- Displacement Response for Buckling Restrained Brace With.

Steel Core Unbounded From Mortar Filled Steel Tube (Clark *et al.*, 2000) Interestingly the difference between the tensile and compressive strength of steel results in somewhat greater strength of the buckling retrained braces in compression than in tension. Usually the hysteresis response is very stable with a large energy dissipation capacity. One of the issues with the hysteresis response of a BRB is the slight inequality between the compressive and tensile strength in which the compressive strength is a bit higher than tensile strength. This small unbalance behavior is due to the compression over strength occurs in restraining mechanism which might be taken into consideration in design procedure.

6.2. Comparison with Other Configurations

As previously described, specially designed and detailed concentrically braced frames can show high elastic stiffness to restrict story drift through yielding and buckling and dissipating energy by diagonal bracing members' performance. The buckling of the brace member, although design specifications like AISC 2010 and CSA 2009 for limiting the slenderness ratio, the width-thickness ratio of the brace member to ensure the sufficient ductility, is inevitable. Therefore, brace buckling leads to both strength and stiffness degradation, which can in some instances cause concentration of damage to a limited number of stories and, hence, increases the potential for building collapse. According to capacity design, the inequality between tension and compression strengths of the brace member, results in large force demand.

The behavior and characteristics of BRBFs discussed briefly in the second chapter of this study and here the design approaches and requirements details will explained.

6.3. Non-Ductile Failure Modes

To make sure that the desired yielding mechanism of the core member will occur in a BRBF, the non-ductile failure modes should be studied. To overcome this type of failure, capacity design procedure applied.

6.3.1. Steel Casing

In order to have a proper and detailed design, the steel casing should to have no contribution in axial force action in the brace member. To avoid global buckling the casing mechanism should have sufficient flexural stiffness. Watanabe *et al.*, 1988 suggested that the contribution of the infill mortar can be neglected and the flexural stiffness of the casing can be defined as:

$$\frac{P_e}{P_y} \geq 1 \tag{6.1}$$

where, P_y is the yield strength of the restrained yielding segment, and P_e is the elastic buckling strength of the restricting casing:

$$P_e = \frac{\pi^2 E I_{sc}}{L_{sc}^2} \tag{6.2}$$

In which, E Young's modulus, $I_s c$ moment of inertia of steel casing, and $L_s c$= work point-to-work point brace length. If it is assumed that the cyclic strain hardening would increase compressive strength of the brace by 30%, and a resistance factor, Ø, of 0.85 is included in the numerator:

$$\frac{\emptyset P_e}{1.3P_y} \geq 1.0 \tag{6.3}$$

or

$$\frac{P_e}{P_y} \geq 1.5 \tag{6.4}$$

Other requirements should be taken into consideration such as the stiffness of the mortar or concrete and the thickness of the casing wall. The mortar should be stiff enough to resist the higher-mode buckle of the core plate. Additionally, the casing wall thickness should be large enough to restrict the buckling of the core plate about its strong axis.

6.3.2. Brace Connection

For the purpose of this study, the bolted connection is taken into consideration.

6.4. BRB Configuration

Several configurations are applicable for BRBFs with minor difference in behavior. V-braced and inverted V-braced configurations as well as other configurations are suggested for BRBFs. The angle of brace inclination has effect on the resisting action of the structure for axial loading. In addition to that, this angle of inclination has degree of influence on the elastic stiffness of the structure. Nevertheless, none of the configurations are subjected to a large redistribution of forces as braces yield and the structure goes beyond the elastic range. The type of the configuration has an indirect effect on the behavior of the structure because of the difference of the yielding segment of the brace core member. Then the V-type configuration has the higher yielding strength because of the longest yielding segment among all other configurations.

BRBFs better to not considered as frames with buckling restrained braces, rather considered as frames in which high inelastic drift capacity is the product of the brace members and can be designed in a way that significant performance and ductility can be achieved.

6.5. Design of Buckling Restrained Braced Frames

Comparing with the special concentric braced frames (SCBFs) and other frame types, buckling restrained braces need less design requirements and design restrictions and the design procedure is much simpler.

6.5.1. Brace Design

In order to have sufficient axial strength, proper sizing of the core plate of brace members is needed. This is a straight forward procedure which is related to the strength of the material. The design strength can be obtained as:

$$\phi P_{ysc} = \phi F_{ysc} A_{sc} \tag{6.5}$$

where $F_y sc$ specified minimum yield stress of the steel core, is the $A_s c$ cross sectional area of the yielding segment of steel core, is the $\phi 0.9$ for the limit state of yielding. The obtained strength is valid for tension and compression as the buckling is completely restrained by the casing.

6.6. Elastic Modeling

For determination of the brace strength and the elastic dynamic characteristic of the structure elastic model is used. Some adjustments are needed for modeling of the structure to capture the proper elastic stiffness of the brace element. After proper assumptions and estimations made to the end zone and yielding segment of the brace such as the length and cross sectional areas, the effective axial stiffness of the brace can be defined as:

$$K_{eff} = \frac{E}{\left(\frac{L_{ysc}}{A_{ysc}} + \frac{L_{nysc}}{A_{nysc}} + \frac{L_{conn}}{A_{conn}}\right)} \tag{6.6}$$

Where E is Young's modulus, A_ysc is the yielding steel core area, A_nysc is the area of the steel core outside of the yielding region, A_conn is the area of the connection (typically approximate), L_ysc is the length of the yielding region of steel core, L_nysc is the length of non-yielding region of steel core, L_conn is the length of the connection. Shorter braces tend to have a greater portion of their length given over to non-yielding and connection regions, the same is true of braces with high axial forces.

6.7. Capacity Design of BRBF

The design of the other components of BRBF including restraining mechanism, connections, beams and columns is performed by capacity design approach. The maximum force in the brace can be obtained regarding the core steel strength, the expected strain hardening under cyclic loading, the compression over-strength, and safety factor based on judgment.

Different configurations of BRBs must be considered and act as a proper mechanism and to ensure the reliable performance of every mechanism testing is needed. In order to decide on the testing program and to evaluating a manufacturer's testing program for a specific project several considerations must be done. All the considerations and required specifications are presented in AISC 341 design code.

For capacity design AISC 341 requires the designer to consider the following adjusted brace tensile strength:

$$T_{max} = \omega R_y P_{ysc} \tag{6.7}$$

where P_ysc $(= F_yscA_sc)$ is the axial yield strength of the steel core calculated based on the minimum specified yield stress. The ω factor is the strain hardening adjustment factor.

The above tensile strength is further increased by a compression strength adjustment factor, β, to compute the adjusted brace compressive strength:

$$C_{max} = \beta T_{max} = \beta \omega R_y P_{ysc} \tag{6.8}$$

In contrast with SCBFs, the compressive strength is substantially higher than tensile strength in BRBFs. Figure 6.4 demonstrates the physical definition of the above mentioned factors.

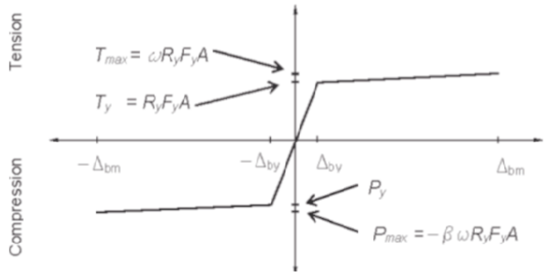

Figure 6.4. The Definition of ω and β Factors of a BRB (AISC 2005).

6.7.1. Brace Casing

According to the expected behavior of the casing mechanism of a BRB, it should have low axial strength and rather the Euler buckling load, P_e should be taken into consideration. To control the undesirable behavior of the casing safety factors are imposed.

6.7.2. Beams and Columns

The maximum values obtained for braces can be used in design of beams and columns as well. The AISC 341 requires designing beams and columns to remain elastic when braces reach their yielding and strain-hardening point. Typically the plastic mechanism analysis performs assuming a first mode deformation. In such a mechanism, the braces are assumed to resist no gravity forces. In this capacity design the aim of design procedure is to determine the strength of the beams and columns to prevent the buckling.

6.8. Nonlinear Modeling

The use of capacity design procedure can result in very large columns for taller buildings. For such structures the assumption of having full yielding in first mode of deformation is unrealistic. Nonlinear models would show significantly lower column axial forces for taller buildings. Such nonlinear models, therefore, offer significant advantages in design, provided that modeling is done correctly, the ground motions considered are appropriate and sufficient, and the results are properly interpreted to provide sufficient reliability, particularly when column buckling is the limit state under consideration.

Nonlinear brace element models should capture the elastic brace stiffness and the expected strain hardening. Bilinear models are typically used, although trilinear models create somewhat closer match to tested brace behavior. Cyclic models can explicitly include both kinematic and isotropic strain hardening.

Typically braces are kept within their ductile deformation range for design. Braces are not designed to allow their deformation capacity to be exceeded (which would allow significant compression to develop in the casing and, at sufficiently large deformations, overall buckling of the brace). There is very little data on brace behavior beyond this range and thus capturing such behavior in a nonlinear model is problematic.

6.9. Code Design Considerations for BRBFs (AISC, Seiscim Provisions)

Buckling-restrained braced frames (BRBF) are a special class of concentrically braced frames. Just as in special concentrically braced frames (SCBF), the center-lines of BRBF members that meet at a joint intersect at a point to form a complete vertical truss system that resists lateral forces. BRBF have more ductility and energy absorption than SCBF because overall brace buckling, and its associated strength degradation, is precluded at forces and deformations corresponding to the design story drift. Note that neither X-bracing nor K-bracing is an option for BRBF.

BRBF are characterized by the ability of bracing elements to yield inelastically in compression as well as in tension. In BRBF the bracing elements dissipate energy through stable tension-compression yield cycles (Clark, Aiken, Kasai, Ko and Kimura, 1999).

Buckling-restrained braced frames are composed of columns, beams, and bracing elements, all of which are subjected primarily to axial forces. Braces of BRBF are composed of a steel core and a buckling-restraining system encasing the steel core. The steel core within the bracing element is intended to be the primary source of energy dissipation. During a moderate to severe earthquake the steel core is expected to undergo significant inelastic deformations. BRBF can provide elastic stiffness that is comparable to that of EBF. Full-scale laboratory tests indicate that properly designed and detailed bracing elements of BRBF exhibit symmetrical and stable hysteretic behavior under tensile and compressive forces through significant inelastic deformations.

The ductility and energy dissipation capability of BRBF is expected to be comparable to that of a special moment frame (SMF) and greater than that of a SCBF. This high ductility is attained by limiting buckling of the steel core. The Provisions are based on the use of brace designs qualified by testing. They are intended to ensure that braces are used only within their proven range of deformation capacity, and that yield and failure modes other than stable brace yielding are precluded at the maximum inelastic drifts corresponding to the design earthquake. For analyses performed using linear methods, the maximum inelastic drifts for this system are defined as those corresponding to 200 % of the design story drift. For nonlinear time-history analyses, the maximum inelastic drifts can be taken directly from the analyses results. A minimum of 2 % story drift is required for determining expected brace deformations for testing and is recommended for detailing. This approach is consistent with the linear analysis equations for design story drift in SEI/ASCE 7 (ASCE, 2002) and the 2003 *NEHRP Recommended Provisions* (FEMA, 2003). It is also noted that the consequences of loss of connection stability due to the actual seismic displacements exceeding the calculated values may be severe; braces are therefore required to have a larger deformation capacity than directly indicated by linear static analysis.

The value of 200 % of the design story drift for expected brace deformations represents the mean of the maximum story response for ground motions having a 10 % chance of exceedance in 50 years (Fahnestock, Sause and Ricles, 2003; Sabelli and others, 2003). Near-fault ground motions, as well as stronger ground motions, can impose deformation demands on braces larger than those required by these provisions. Detailing and testing braces for larger deformations will provide higher reliability and better performance.

The design engineer utilizing these provisions is strongly encouraged to consider the effects of configuration and proportioning of braces on the potential formation of building yield mechanisms. The axial yield strength of the core, P_ysc, can be set precisely with final core cross-sectional area determined by dividing the specified brace capacity by actual material yield strength established by coupon testing, multiplied by the resistance factor. In some cases, cross-sectional area will be governed by brace stiffness requirements to limit drift. In either case, careful proportioning of braces can make yielding distributed over the building height much more likely than in conventional braced frames.

6.10. Bracing Members

The steel core is composed of a yielding segment and steel core projections; it may also contain transition segments between the projections and yielding segment. The cross-sectional area of the yielding segment of the steel core is expected to be sized so that its yield strength is fairly close to the demand calculated from the applicable building code. Designing braces close to the required strengths will help ensure distribution of yielding over multiple stories in the building. Conversely, overdesigning some braces more than others (for example, by using the same size brace on all floors) may result in an undesirable concentration of inelastic deformations in only a few stories. The length and area of the yielding segment, in conjunction with the lengths and areas of the non-yielding segments, determine the stiffness of the brace. The yielding segment length and brace inclination also determines the strain demand corresponding to the design story drift.

In typical brace designs, a projection of the steel core beyond its casing is necessary in order to accomplish a connection to the frame. Buckling of this unrestrained zone is an undesirable failure mode and must therefore be precluded.

In typical practice, the designer specifies the core plate dimensions as well as the steel material and grade. The steel stress-strain characteristics may vary significantly within the range permitted by the steel specification, potentially resulting in significant brace over-strength. This over-strength must be addressed in the design of connections as well as of frame beams and columns. The designer may specify a limited range of acceptable yield stress in order to more strictly define the permissible range of brace capacity. Alternatively, the designer may specify a limited range of acceptable yield stress if this approach is followed in order to more strictly define the permissible range of core plate area (and the resulting brace stiffness). The brace supplier may then select the final core plate dimensions to meet the capacity requirement using the results of a coupon test. The designer should be aware that this approach may result in a deviation from the calculated brace axial stiffness. The maximum magnitude of the deviation is dependent on the range of acceptable material yield stress. Designers following this approach should consider the possible range of stiffness in the building analysis in order to adequately address both the building period and expected drift.

7. CASE STUDY

The next step for this study is the design considerations and the requirements of the specified structures.

7.1. Performance Based Approach

Performance based design appears to be the future direction of seismic design codes. In the newly developed performance based seismic design approach, nonlinear analysis procedures become important in identifying the patterns and levels of damage for assessing a structure's inelastic behavior and for understanding the failure modes of the structure during severe seismic events. Pushover analysis is a simplified, static, nonlinear procedure in which a predefined pattern of earthquake loads is applied incrementally to framework structures until a plastic collapse mechanism is reached. This analysis method generally adopts a lumped plasticity approach that tracks the spread of inelasticity through the formation of nonlinear plastic hinges.

As known in newly design codes, the nonlinear static analysis procedure requires determination of three primary elements: capacity, demand and performance. The capacity spectrum can be obtained through the pushover analysis, which is generally produced based on the first mode response of the structure assuming that the fundamental mode of vibration is the predominant response of the structure. This pushover capacity curve approximates how a structure behaves beyond the elastic limit under seismic loadings. The demand spectrum curve is normally estimated by reducing the standard elastic 5% damped design spectrum by the spectral reduction method.

In this phase three 3, 6 and10 storey buildings were modeled and being analyzed. In order to having an idea about the performance of the several configurations applied to the structures, three different configurations were taken into account. The configurations used for modeling are one concentric invert chevron braced frame and two invert chevron type eccentric type bracing systems with short and long links.

The aim of this study is to investigate the performance of the concentrically, eccentrically braced frames utilizing buckling restrained braces subjected to seismic loads with the intention to understanding of the behavior of the braced frames under seismic loads and come up with the comparison of the different configurations applied to braced systems. The governing criteria for understanding the reliability of the diverse configurations are the displacement control design process.

7.2. General Assumptions

The design approach of the US Federal Emergency Management Agency (FEMA) is used for assessment of the braced frames. The procedure to follow is first to obtain the required properties of the members of the frame. The goal of this stage is to find the seismic demand of the members especially brace members.

7.3. Loading Considerations

The dedicated loading system for this study is completely in compliance of the ASCE 7-10 (Minimum Design Loads for Buildings and Other Structures) standard for Allowable Stress Design. In this method the structural members are expected to resist for the nominal loads applying to the structure and maintain their elasticity and do not exceed the requirements of the provisions. Thus, the load combinations shall be set to obtain this goal. The load combinations used in this study are defined as:

(i) DL
(ii) DL + LL
(iii) DL + 0.75 LL
(iv) DL + 0.125 EQX (0.166 EQX)
(v) DL + 0.125 EQY (0.166 EQY)
(vi) DL + LL + 0.125 EQX (0.166 EQX)
(vii) DL + LL + 0.125 EQY (0.166 EQY)

In which the DL represents the Dead Load and LL is Live Load and EQX and EQY are for code reliant design spectrums in X and Y directions respectively. For earthquake loads the assumption is to apply in both directions. To be more realistic for each load pattern the assumption is to add the current load with a percentage of the perpendicular load acting in other direction.

$$EQX = UX + 0.3UY \tag{7.1}$$

where is the UX corresponding design spectrum in X direction, is the UY corresponding design spectrum in Y direction.

The wind and snow and flood loads are neglected due to the low effect on the structural design because of the Istanbul's geographical zone.

For earthquake load patterns, the response reduction factors of 8 and 6 were applied to the loads according to Turkish Earthquake Code (TDY) for concentric and eccentric configurations.

7.4. General Information About the Structures

7.4.1. Concentrically Braced Frame

For the aim of this study 3, 6 and 10 storey buildings are chosen and the storey level starts from the ground and the plan dimensions are 24 meters in each direction. In order to comply with the symmetry rule and simplify the future consideration there is no irregularity in the plan of the structure. The length of each bay is 6 meters and accordingly there are 4 bays in each direction. The height of each floor is 3 meters which includes finishing and slabs. The braced bays are located in both ends of each X and Y directions. The bracing system is invert chevron or invert V type system. Figure 7.1 shows the plan view of the structure with the location of braced frames.

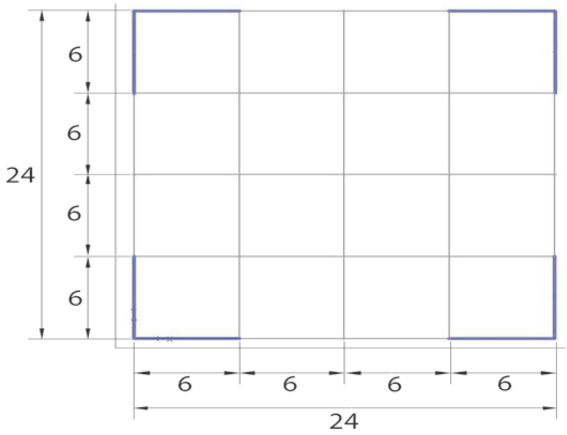

Figure 7.1. Plan View of the Structure with the Location of Bbracing Frames (SAP 2000).

In the design of steel structures the objective of design is to resist lateral loads in seismic activities. One of the key requirements for this purpose is to restrict the building from large displacements during the ground motion. For this reason the tent to use concentrically braced frames growth increasingly. However after frequent damages to concentrically braced frames, attentions turn to use of special concentric braced frames (Sabelli *et al.*, 2003). Buckling restrained frames are one of the solutions for preventing the structure from damage state and showing high elastic stiffness.

Several configurations were studied over past years and concentrically braced frames are one of the first frameworks taken into account to resist lateral loads and intimately earthquake load. As a flow of this study and to have a good understanding of the behavior of buckling restrained braces under seismic loads, first step is to recognize the characteristic and mechanical properties of the braced frame before replacing the conventional braces with the buckling restrained braces in frames.

The modeling is performed using SAP 2000 V. 15.0.1, the structural design program, and structural members were modeled using EURO CODE specified members for beams and columns. The same profiles are used to model brace elements in order to obtain the maximum demand which is measured at the end joints of the braces and calculate the DCR ratio (capacity/demand ratio). The slabs are two-way slabs and vertical loads were distributed accordingly. All floors defined as rigid diaphragms. Figure 7.2 is a schematic 2D frame with assigned frame members.

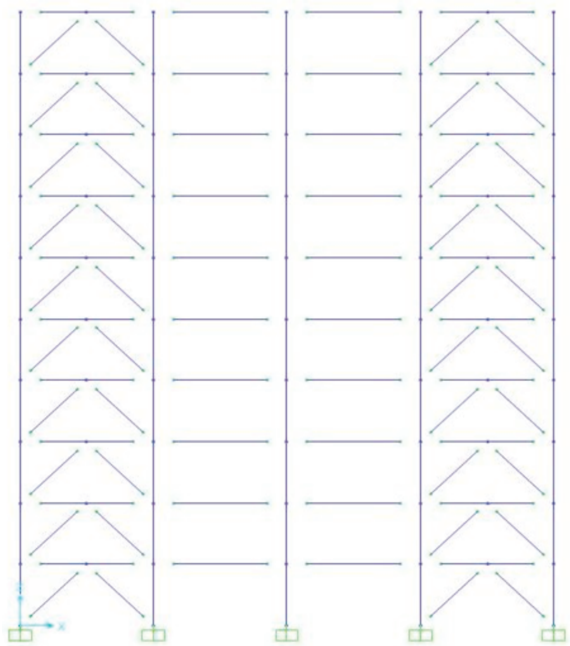

Figure 7.2. Schematic View of Concentrically Braced Frame with Assigned Frame Members.

7.4.2. Eccentrically Braced Frame

This type of bracing system is one of the well known systems in recent design strategies because of its performance in seismic events and the act of resisting lateral loads. Some of the key characteristics which differs this configuration are:

- High elastic stiffness
- High ductility and energy dissipation
- Stability in inelastic response under cyclic loading

The key distinguishing feature of an EBF is that at least one end of each brace is connected so as to isolate a segment of beam called link. In this study split K braced configuration is used as the objective of assessment (Ductile Design of Steel Structures- Bruneau, Uang, Whittaker).

The governing factor in this kind of configuration is the deformation capacity of the link. Simple relationship between the frame shear force and the link shear force can be developed for common EBF configurations for the purpose of preliminary design. For this reason the link length and behavior is of great importance.

7.4.2.1. Link Behavior and Length. In the case of link behavior, the relationship between story plastic drift (Δ_p) and link plastic rotation (γ_p) can be simply derived:

$$\Delta_p = \frac{\gamma_p e h}{L} \tag{7.2}$$

In which e is the length of the link and h is the storey height and L is the column to column distance.

The link rotation demand has the direct relationship with the link length as illustrated in Figure 7.3. Link rotation demand grows quickly as the link length decreases with respect to the frame dimension L.

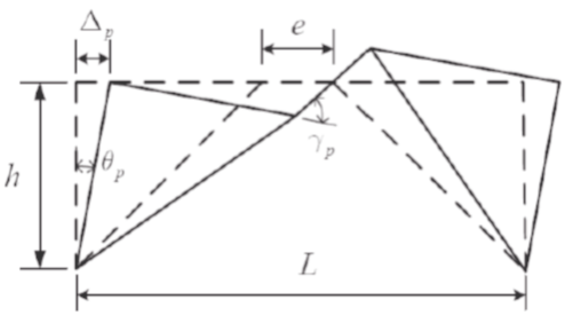

Figure 7.3. The Rotation of Link Under Lateral Force.

Figure 7.4 shows the link rotation demand versus e/L for the split K braced EBF. The limiting case of e/L equal to 1.0 corresponds to a moment resisting frame. That is γ/θ approaches 1.0 as e/L approaches 1.0. Although large link rotation demands can be realized by short (shear) links, Figure 9 demonstrates that link should not be too short or else the link rotation demands will become excessive even for a shear-yielding link.

$$\gamma = \frac{L}{e}\theta \tag{7.3}$$

The length and geometry of the links in an EBF will dictate the behavior of the frame. Short links are shear-critical and long links are flexure-critical. The critical factor affecting the inelastic behavior of a link is its length. Link length controls the yielding mechanism and the ultimate failure mode. For short links, shear dominates the link response; for longer links, flexure controls link response.

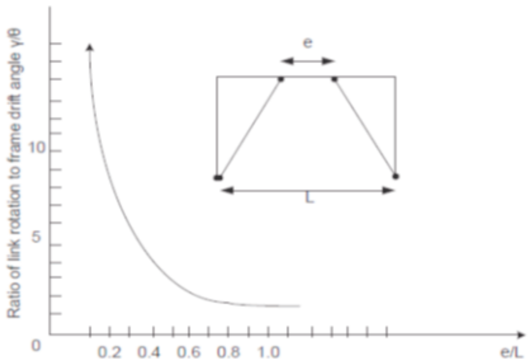

Figure 7.4. Link Rotation Demand.

Regarding to explanations above the following equations for the link length (e) can be used to classify links:
Shear (short) links:

$$e \leq \frac{1.6M_p}{V_p} \tag{7.4}$$

Intermediate links:

$$\frac{1.6M_p}{V_p} < e < \frac{2.5M_p}{V_p} \tag{7.5}$$

Moment (long) links:

$$e > \frac{2.5M_p}{V_p} \tag{7.6}$$

In which $M_(p)$ (=$0.55dt_wF_y$) is the plastic moment capacity and V_p (=ZF_y) is the plastic shear force of the link. For the purpose of this study one configuration with a shear (short) link and one another with a moment (long) link is modeled and analyzed. Figure 7.5 shows the schematic view of the mathematical model for both aforementioned configurations.

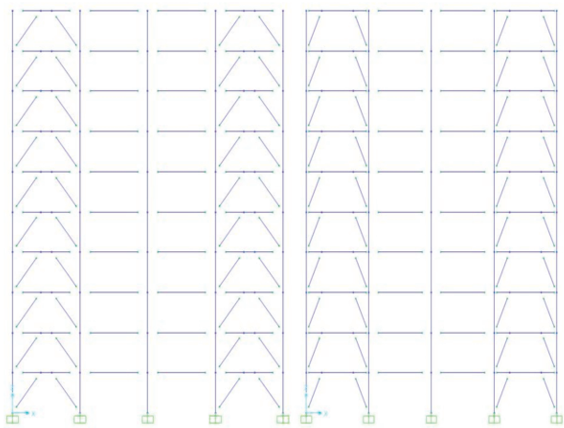

Figure 7.5. Eccentric Braced Frame in the Long Link (Left) and Short Link (Right) Shape.

7.5. Nonlinear Static Analysis (Incremental Push Over Analysis)

As this study is in accordance with the performance based design, the evaluation of the framework structure should perform based on this analysis method to get to the desirable performance level. In order to follow this procedure the first step is to run a nonlinear static analysis (pushover) to evaluate the mathematically modeled structure's response and if needed redesign the structural elements to comply with the code requirements.

In performance based design the principle of design is the inter-story drift of a multi storey structure. This criterion is the base for structural damage measurement.

The pushover analysis is a nonlinear static procedure in which a predefined earthquake load is applied incrementally to structure until the formation of the plastic collapse mechanism. This analysis method generally adopts a lumped plasticity approach that tracks the spread of the inelasticity through the formation of nonlinear plastic hinges. In this approach two items are expected to obtain:

- The pushover curve
- The performance point

The pushover curve is the representative of the structural behavior under the design spectrum which is applied according to the Turkish Earthquake Code and with the viscous damping of 5%. This curve also shows the inelastic behavior of the frame after formation of the plastic hinges. The intersection of pushover curve and the design spectrum gives a point which is called "Performance Point" and indicates the level of the structure's performance under the specified spectrum.

Three different methods are specified in Turkish Earthquake Code (TDY) for non linear procedures which are:

- Incremental Equivalent Seismic Load Method (Pushover Analysis Method)
- Incremental Mode Superposition Method
- Analysis Method in Time Domain

7.5.1. Incremental Equivalent Seismic Load Method

In this method, nonlinear pushover analysis is performed under monotonically increasing earthquake load until performance point is reached. Performance point is also named as "Target Modal Displacement Demand". Displacement, plastic deformation, increase in internal forces and relative cumulative values are determined at each pushover step. Once the system reaches its performance point, total base reactions and roof displacement values are determined.

In incremental equivalent seismic method, performance point of building is represented with base shear- roof displacement curve and modal capacity diagram. Roof displacement is the displacement calculated in each pushover step in X earthquake direction considered at center of mass of the top storey. Base shear force is the sum of equivalent earthquake loads in each step of X earthquake direction. Structural system is calculated under vertical loads and proportionally increasing earthquake loads to obtain pushover curve until the performance point is reached. For the aim of this study this method is used to evaluate the structural performance under nonlinear static loading.

7.6. General Considerations

Assigned properties are as following lines:

7.6.1. General Information

The first step is to specify the material properties of the cross sections used.

<u>7.6.1.1. Material.</u> For mathematical modeling the frame members assigned according to European Code specified profiles for steel structures. All the beams and columns and brace members defined before performing the analysis and design section and the proper cross sections redefined by iteration. The iteration is done by considering the DCR ratio and with the compliance with the AISC provision. The nonlinear analysis program SAP2000 applied for estimating the desirable design. Table 7.1, Table 7.2 and Table 7.3 illustrates the final code-reliant cross sections assigned to the structure frame. In the following the 3D views of concentrically braced frames as an instance are coming.

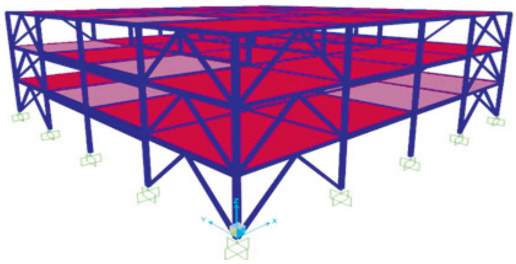

Figure 7.6. The 3D View of the 3 Story (Low Rise) Structure with CBF (SAP 2000).

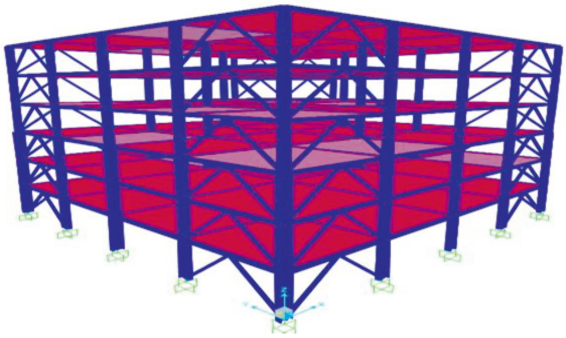

Figure 7.7. The 3D View of the 6 Story (Mid Rise) Structure with CBF (SAP 2000).

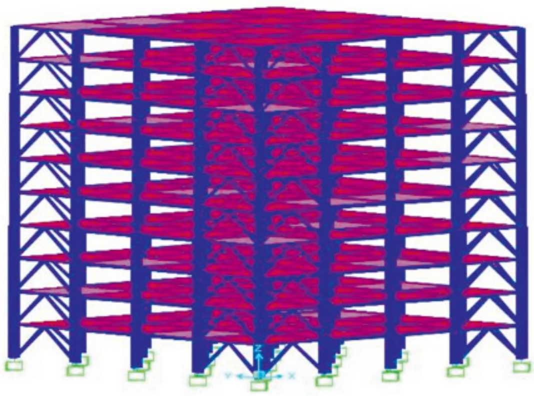

Figure 7.8. The 3D View of the 10 Story (High Rise) Building with CBF (SAP 2000).

Table 7.1. FEMA 356 Code Based Designed Cross Sections for 3 Story Building.

	column			beam			brace		
storey	CBF	EBF (SHORT)	EBF (LONG)	CBF	EBF (SHORT)	EBF (LONG)	CBF	EBF (SHORT)	EBF (LONG)
1	HE300B	HE300B	HE300B	HE220A	HE220A	HE220A	HE100A	HE100A	HE100A
2	HE300B	HE300B	HE300B	HE220A	HE220A	HE220A	HE100A	HE100A	HE100A
3	HE300B	HE300B	HE300B	HE220A	HE220A	HE220A	HE100A	HE100A	HE100A

Table 7.2. FEMA 356 Code Based Designed Cross Sections for 6 Story Building.

storey	column			beam			brace		
	CBF	EBF (SHORT)	EBF (LONG)	CBF	EBF (SHORT)	EBF (LONG)	CBF	EBF (SHORT)	EBF (LONG)
1	HE400B	HE400B	HE400B	HE220A	HE220A	HE220A	HE160A	HE160A	HE160A
2	HE400B	HE400B	HE400B	HE220A	HE220A	HE220A	HE160A	HE160A	HE160A
3	HE400B	HE400B	HE400B	HE220A	HE220A	HE220A	HE140A	HE140A	HE140A
4	HE300B	HE300B	HE300B	HE220A	HE220A	HE220A	HE140A	HE140A	HE140A
5	HE300B	HE300B	HE300B	HE220A	HE220A	HE220A	HE120A	HE120A	HE120A
6	HE300B	HE300B	HE300B	HE220A	HE220A	HE220A	HE120A	HE120A	HE120A

Table 7.3. FEMA 356 Code Based Designed Cross Sections for 10 Story Building.

storey	column			beam			brace		
	CBF	EBF (SHORT)	EBF (LONG)	CBF	EBF (SHORT)	EBF (LONG)	CBF	EBF (SHORT)	EBF (LONG)
1	HE600B	HE600B	HE600B	HE220A	HE220A	HE220A	HE160A	HE160A	HE160A
2	HE600B	HE600B	HE600B	HE220A	HE220A	HE220A	HE160A	HE160A	HE160A
3	HE600B	HE600B	HE600B	HE220A	HE220A	HE220A	HE160A	HE160A	HE160A
4	HE600B	HE600B	HE600B	HE220A	HE220A	HE220A	HE160A	HE160A	HE160A
5	HE400B	HE400B	HE400B	HE220A	HE220A	HE220A	HE140A	HE140A	HE140A
6	HE400B	HE400B	HE400B	HE220A	HE220A	HE220A	HE140A	HE140A	HE140A
7	HE400B	HE400B	HE400B	HE220A	HE220A	HE220A	HE140A	HE140A	HE140A
8	HE400B	HE400B	HE400B	HE220A	HE220A	HE220A	HE140A	HE140A	HE140A
9	HE200B	HE200B	HE200B	HE220A	HE220A	HE220A	HE120A	HE120A	HE120A
10	HE200B	HE200B	HE200B	HE220A	HE220A	HE220A	HE120A	HE120A	HE120A

7.6.2. Vertical Loads

Vertical loads defined based on TS 498 (Turkish Standard for Design Loads) for obtaining the elastic seismic loads acting on the structural system throughout the design procedure. The design loads are as shown below:

$g = 3.75 {}^{KN}/_{m^2}$ (Including slabs)

$q = 5 {}^{KN}/_{m^2}$ (Including infill walls)

Above mentioned loads applied for all of the floors. For structural elements their self weights calculated and defined separately and added to the acting dead load of the structure.

7.6.2.1. Determination of the Equivalent Seismic Load. The total equivalent seismic load (base shear), V_t, acting on the structural frame in the earthquake direction, shall be obtain by:

$$V_t = C.W = \frac{W.A\,(T_1)}{R_a\,(T_1)} \tag{7.7}$$

In which W is the seismic weight of the structure and $A(T_1)$ is the spectral acceleration coefficient and $R_a\ (T_1)$ is the seismic load reduction factor. The determination of the terms mentioned above, are as following lines. The seismic weight is the summation of floor based combination of dead and a portion of relative live load as:

$$W = \sum_{i=1}^{N} w_i = g_i + nq_i \tag{7.8}$$

In this equation N indicates the number of floors starts from ground level and n is for the live load participation factor in accordance with the code reliance table below:

Table 7.4. The Live Load Participation Multiplier.

Purpose of Occupancy of Building	n
Depot, warehouse, etc.	0.8
School, dormitory, sport facility, cinema, theatre, concert hall,car park, restaurant, shop, etc.	0.6
Residence, office, hotel, hospital, etc.	0.3

For the purpose of this study 0.3 is used as live load participation factor. As spectral acceleration factor the equation is:

$$A\,(T) = A_0.I.S(T) \tag{7.9}$$

The effective ground acceleration coefficient, A_0, is taken as 0.4 which is chosen according to the seismic zone that the structure is located in and can be seen in the following table:

Table 7.5. The Effective Ground Acceleration Coefficient (TEC).

Seismic Zone	A_0
1	0.4
2	0.3
3	0.2
4	0.1

Another parameter in the above equation is the building importance factor, I, which can be obtain from the table provided also in TEC and is 1 according to the type and the importance of the building after the seismic event. The Table 7.5 shows the importance factor presented in TEC. In order to obtain the spectrum coefficient, $S(T)$, should use one of the following equations respected to the soil conditions of the local site and first period of the structure.

$$S\left(T\right) = 1 + 1.5\frac{T}{T_A}\left(0 \leq T \leq T_A\right) \tag{7.10}$$

$$S\left(T\right) = 2.5\left(T_A < T \leq T_B\right) \tag{7.11}$$

$$\left(T\right) = 2.5\left(\frac{T_B}{T}\right)^{0.8}\left(T_B < T\right) \tag{7.12}$$

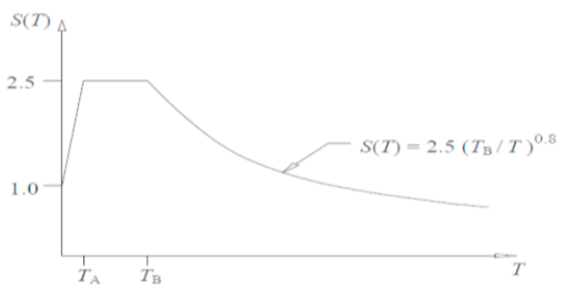

Figure 7.9. General Design Spectrum (TEC).

Table 7.6. The Importance Factor Corresponding to the Building (TEC).

Binanin kullanim amaca ve türü	Bina Önlem Katsayisi (I)
1. Deprem sonrasi kullanim gerek binalar ve thelikeli madde içeren binalar a) Deprem sonrasinda hemen kullanilmasi gerekli binalar (Hastaneler, dispansenserler, saglik ocaklari, itfaiye bina ve tesisleri, PTT ve diger haberlesme tesisleri, ulasim istasyonlari ve terminalleri, eneji üretim ve dagitim tesisleri; vilayet, kaymakamlik ve belediye yönetim binalari, ilk yardim ve afet planlama istasyonlari b) Toksik, patlayici, parlayici, vb özellikleri olan maddelerin bulundugu veya depoladigi binalar	1.5
2. Insanlarin uzun süreli ve yogun olarak bulundugu ve degerli esyanin saklandigi binalar a) Okullar, diger egitim bina ve tesisleri, yurt veya yatakhaneler, askeri kislalar, cezaevleri, vb. b) Müzeler	1.4
3. Insanlari kisa süreli ve yogun olarak bulundugu binalar Spor tesisleri, sinema, tiyatro ve konser salonlari, vb.	1.2
4. Yukaridaki tanimlara girmeyen diger binalar (Konutlar, isyerleri, oteller, bina türü endüstri yaplari, vb.	1.0

The Response Reduction Factor(R) of 8 and 6 are applied according to TDY for concentric and eccentric configurations respectively. The table below shows the response reduction factors.

Table 7.7. Characteristic Periods Corresponding to Local Site.

Çelik Binalar		
1) Deprem yüklerinin tamaminin çerçevelerle tasidigi binalar	5	8
2) Deprem yüklerinin tamaminin üstteki baglantilari mafsalli olan kolonlar tarafindantasindigi tek katli binalar	-	4
3) Deprem yüklerinin tamaminin çaprazli perdeler veya yerinde dökme betonarme perdeler tarafindan tasindigi binalar		
a) Çaprazlarin merkezi olmasi durumu	4	5
b) Çaprazlarin dismerkezi olmasi durumu	-	7
c) Betonarme perdelerin kullanimasi durum	4	6
4) Deprem yüklerinin çerçeveler ile birlikte çaprazli çelik perdeler veya yerinde dökme betonarme perdeler tarafindan birlikte tasidigi binalar		
a) Çaprazlarin merkezi olmasi durum	5	6
b) Çaprazlarin dismerkez olmasi durum	-	8
c) Betonarme perdelerin kullanilmasi durum	5	7

The characteristic periods of the soil type, T_A&T_B are given in Table 7.7 according to the specifications in TEC.

Table 7.8. Response Reduction Factors (TEC).

Local Site Class	$T_{A(S)}$	$T_{B(S)}$
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

As seismic load reduction factor, $R_a(T)$, one of the following equations can be taken into consideration and the effective ground acceleration factor is divided by this term:

$$R_a\left(T\right) = 1.5 + \left(R - 1.5\right)\frac{T}{T_A}\left(0 \leq T \leq T_A\right) \tag{7.13}$$

$$R_a\left(T\right) = R(T_A < T) \tag{7.14}$$

As a check, the obtained number should be:

$$V_t \geq 0, 10.A_0.I.W \tag{7.15}$$

7.7. Force Distribution

The distribution of the building's base shear along the height of the structure should be done via the bellow equation. By ignoring the ΔF_n the rest of the equation is as:

$$F_i = \left(V_t - F_n\right)\frac{w_i.H_i}{\sum_{j=1}^{n} w_j.H_j} \tag{7.16}$$

In which F_i is the fictitious load acting on ith floor and by using this equation the acting earthquake load can be distributed relatively.

8. ANALYSIS AND RESULTS

This part of the study is to observe the results of pushover analysis and the sequences of plastic hinge occurrence. The non linear static analysis (pushover) performed on the three mathematical models separately until the formation of the plastic hinge. In order to obtain the capacity values and the required cross sections for each of the nominated configurations the analysis performed using the nonlinear analysis program and conventional brace members. The reason for using the conventional brace members is to evaluate the proportionality and the sufficiency of the brace placements in the buildings and the maximum forces acting in those members during the analysis process. A 2D end-bay frame in X direction is used to be analyzed for the purpose of this study. The formation of plastic hinges, as illustrated in the following pictures, initiates from brace members as expected and the collapse mechanism did not reach during the analysis process. In the Figure 8.1, Figure 8.2 and Figure 8.3 the pushover analysis steps are shown for concentric, short link eccentric and long link eccentric configurations respectively. In the figures the location of plastic hinges are shown and the performance levels are indicated by different colors.

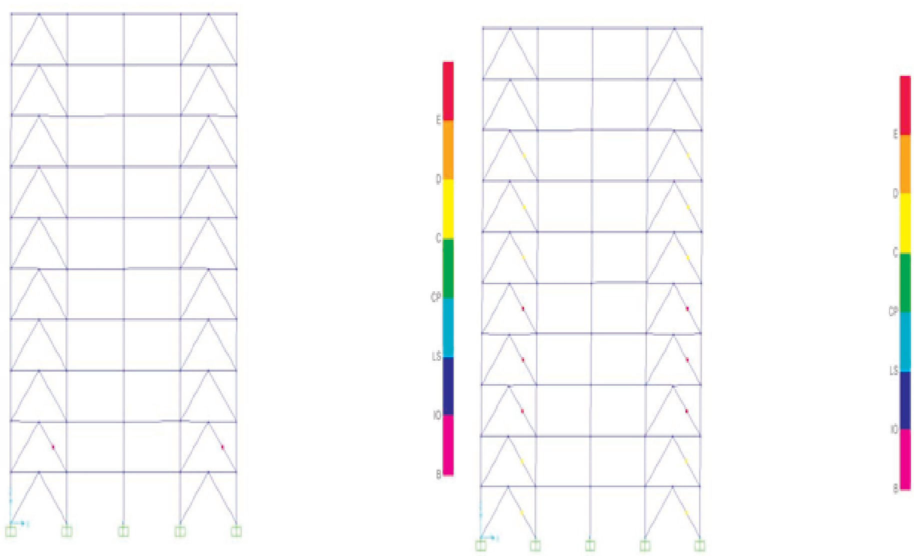

Figure 8.1. Formation of Plastic Hinges in CBF (SAP 2000).

As can be seen in Figure 8.1, the formation of plastic hinge starts from 2nd floors frame's brace members but in this step the rotation starts under the compression. As the analysis goes forth other braces reach their plastic hinges and in the second floor's brace yielding occurs and at the final step braces of seven floors exceeds the yielding level and also exceeds the life safety level. The point is that none of the structural members (columns and beams) reach their damage state level. This is truly desirable for the purpose of this study which intends to have the damage state level within the brace members prior to structural members. However, the buckling of the brace under compression forces is not assumed to be the achievement of this study which is preventing the brace members from buckling in compression and make them to continue their performance until reaching to higher level of resistance which will be evaluate in next steps by utilizing BRB members. The same procedure is done for eccentrically braced frames and in Figure 8.2 and Figure 8.3 the sequential steps are demonstrated.

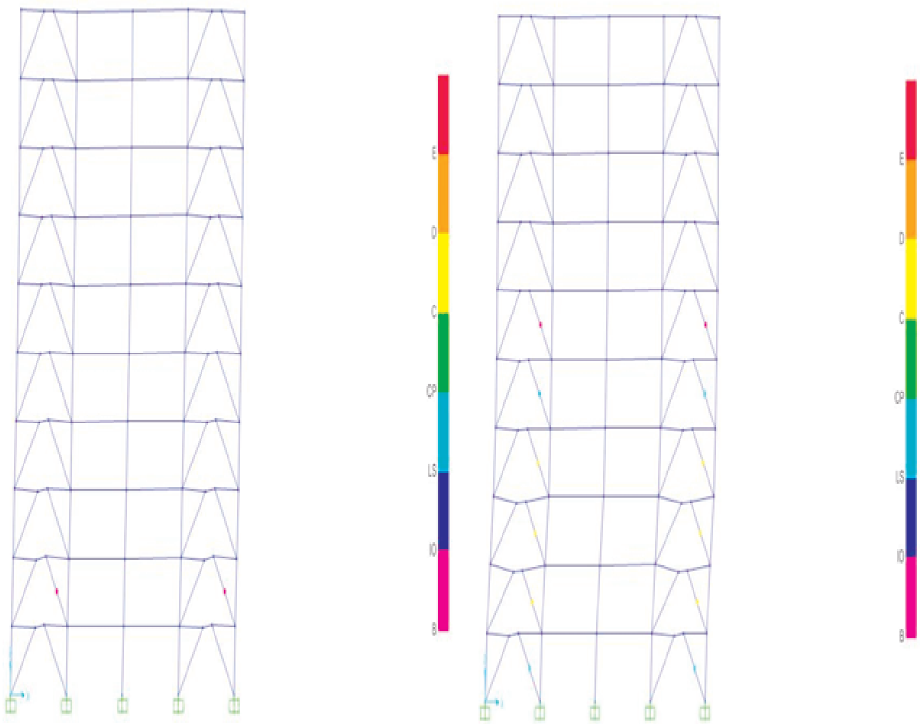

Figure 8.2. Formation of Plastic Hinges in EBF With Short Links (SAP 2000).

For this configuration result is quite different compared with concentrically braced frame. The same as the concentrically braced frame the formation of plastic hinges starts from 2^{nd} floor. However, the link members who are supposed to act as structural fuses are not work properly and the plastic hinges occur in other braces as well. This action suggested that the braces are not sufficient enough to reach the higher level of forces which is likelihood in eccentric configurations.

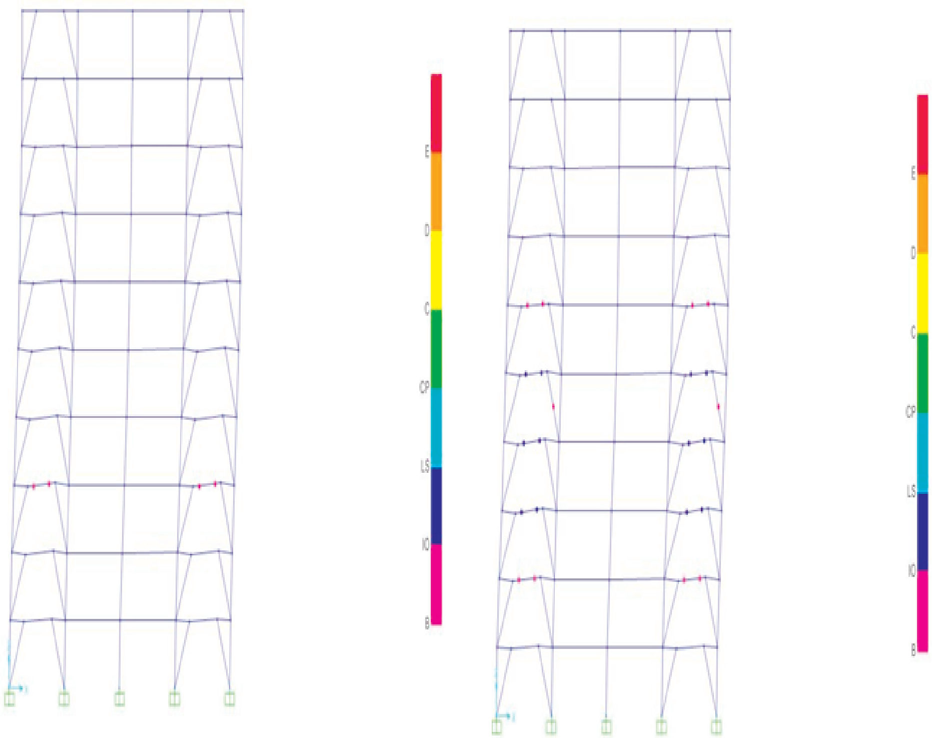

Figure 8.3. Formation of Plastic Hinge in EBF With Long Links (SAP 2000).

Regarding to eccentrically braced frames with long links, results are different from the same configuration with short link. In this type of configuration the formation of the plastic hinge starts from the link member and in shear zone. As can be seen from the figure, at the last step link members of six floors start rotating and only in one of the floors the brace member starts rotating under compression. This result suggests that link members are working quite well and the structure performs in a good manner. However, most of the floors do not respond as they expected to contribute in the resisting system of the structure under loading. This behavior is what is expecting from eccentrically braced frames, but it is not enough to rely on to get to higher load resisting capacities.

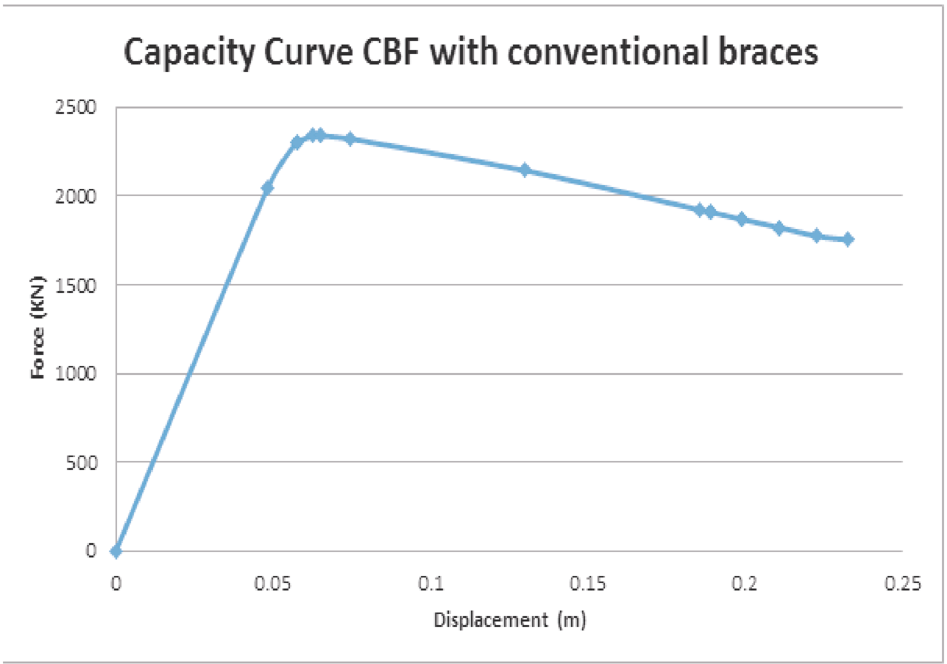

Figure 8.4. The Capacity Curve of Concentric Braced Frame.

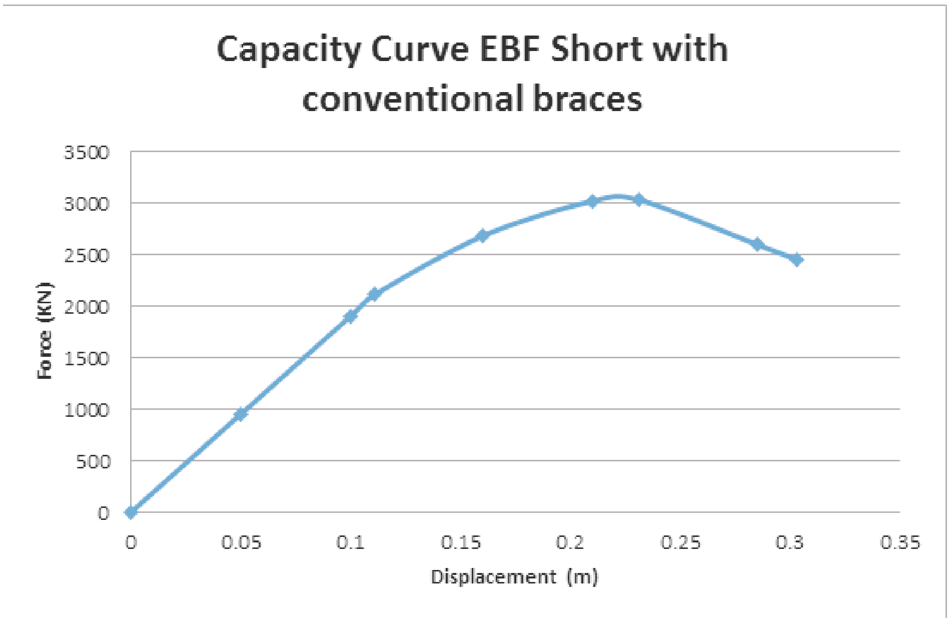

Figure 8.5. The Capacity Curve of Eccentric Braced Frame With Short Links.

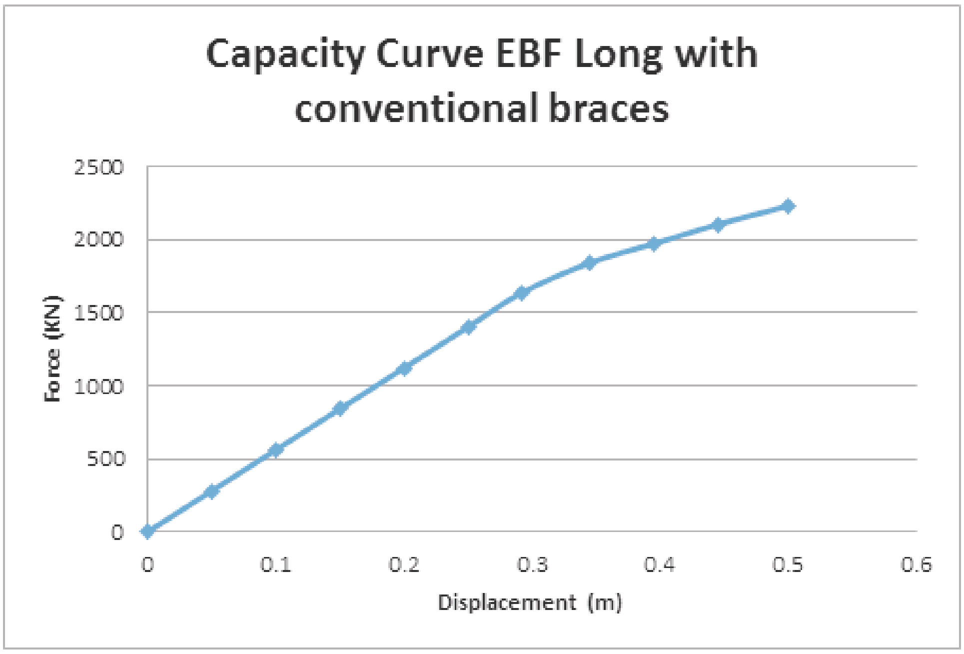

Figure 8.6. Capacity Curve for Eccentric Braced Frame With Long Links.

As shown in the Figure 8.4, Figure 8.5 and Figure 8.6 the capacity curves determined from pushover analysis performed on the concentric and eccentric braced frame works displays the target displacements and maximum base shear can be gained by the structure being analyzed. As can be seen, in the concentric braced frame the reached displacement is around 0.05 meter and the base shear of almost 2300 (KN) is gained. In the other hand the achieved displacement for eccentrically braced frame with short link is roughly 0.23 meters and the base shear is about 3000 (KN). For eccentrically braced frame with long link the maximum displacement is up to 0.5 meters and the base shear is about 2200 (KN). This result indicate that the eccentrically braced frame with long link is capable of resisting higher base shears with less displacement and the eccentrically braced frame with short link gets to higher base shear but after this point the curve starts dropping. For the concentrically braced frame the result is not as good as other configurations.

8.1. Modeling of the Brace for BRB

The buckling restrained brace aimed for the purpose of this study is modeled using the values obtained from the pre-application part by utilizing capacity values obtained from one bay one story frame analysis. The following figures show the evaluation process in order to get the maximum capacity can be reached by a steel brace and use it for correct the compression part of the steel brace behavior according to the research results conducted by Sabelli *et al.*, (2003) in which the compression capacity gained for buckling restrained brace was 10% higher that the tension capacity.

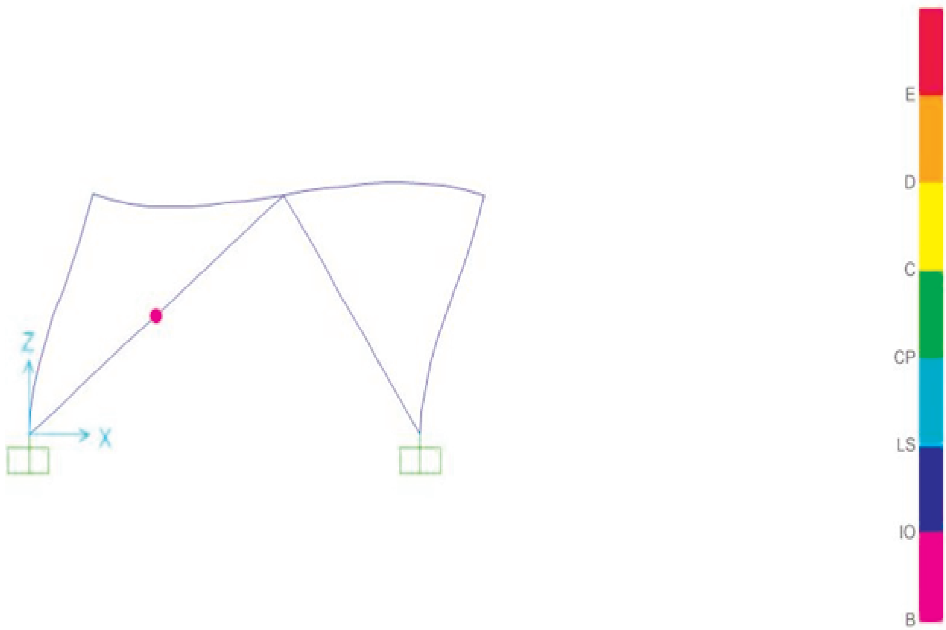

Figure 8.7. Formation of Plastic Hinge in Tension Within the Brace (SAP 2000).

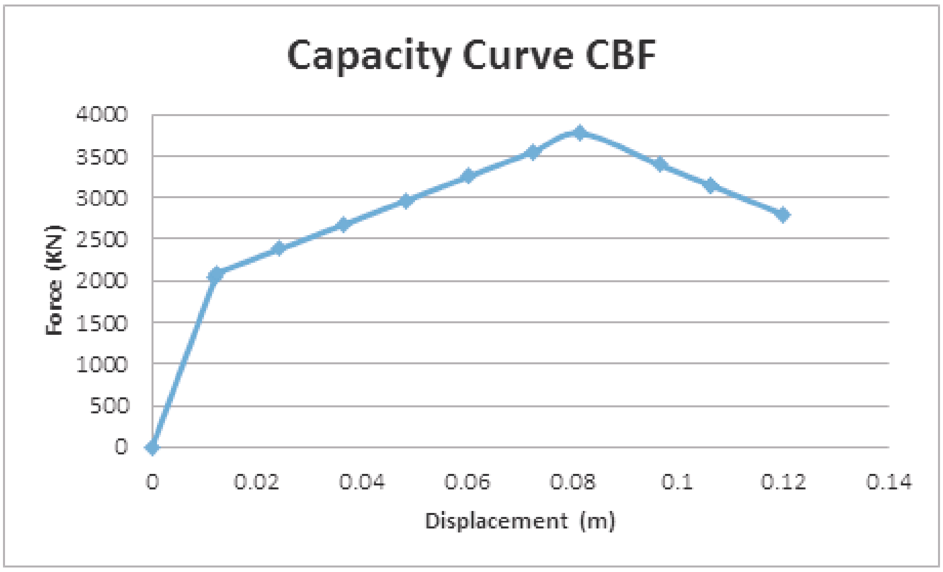

Figure 8.8. Capacity Curve for one Bay Braced Frame (SAP 2000).

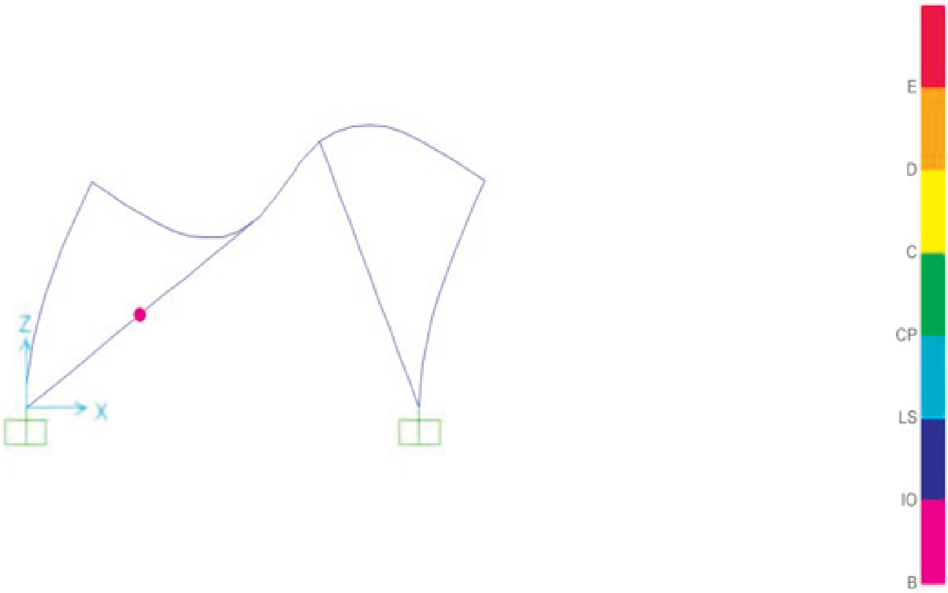

Figure 8.9. Formation of Plastic Hinge in one Bay Frame (SAP 2000).

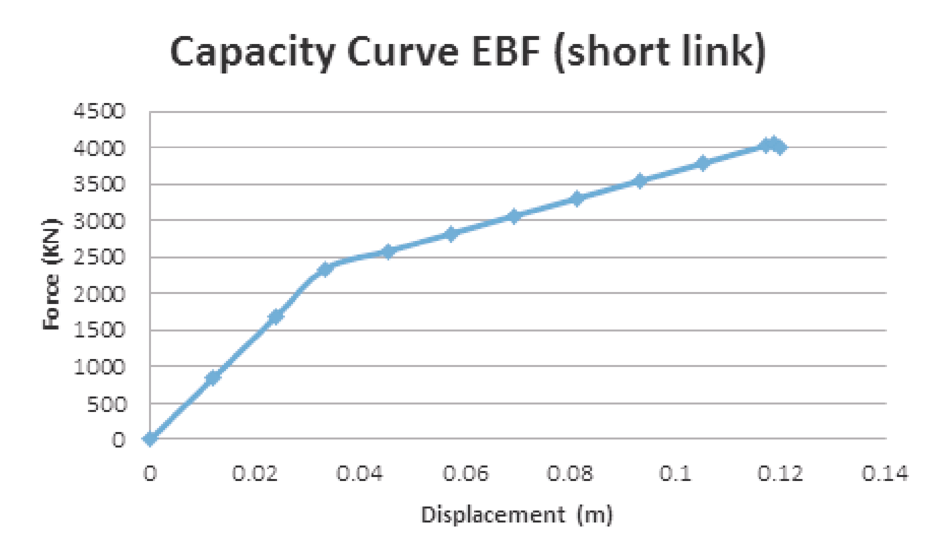

Figure 8.10. Capacity Curve for one Bay Braced Frame (SAP 2000).

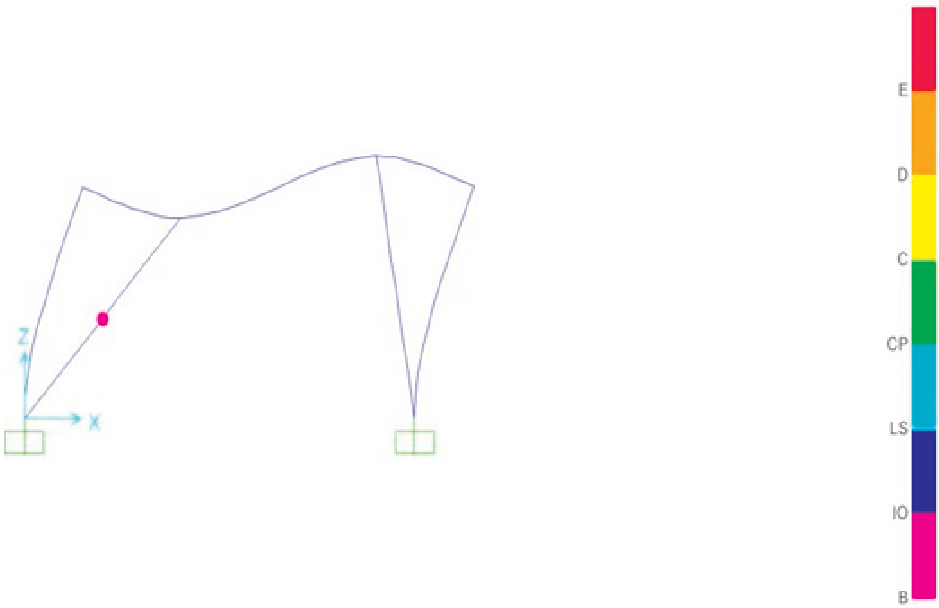

Figure 8.11. Formation of Plastic Hinge in one Bay Frame (SAP 2000).

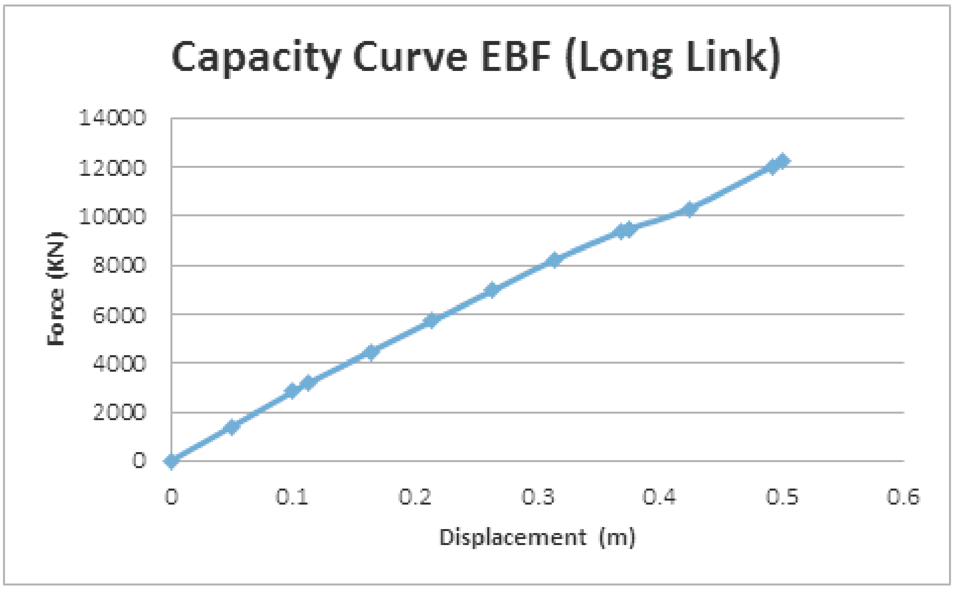

Figure 8.12. Capacity Curve for one Bay Braced Frame (SAP 2000).

As illustrated in the figures above, hinges assigned to the braces according to the FEMA 365 provisions and it can be seen in the pictures that when the hinge starts to rotate it means that the resisting capacity is reached and the next step is the yielding of the member. In the upcoming table the obtained capacities are shown according to the analysis results. The tension and compression capacities determined using SAP 2000 v 15.0.1 analysis program and checked by hand calculations and the calculated values are almost the same when compared.

Compression capacity calculated by multiplying tension capacity by 1.1 which corresponds to 10% higher in capacity. The resulted values assigned to the brace by changing the hinge properties in the analysis program. According to Sabelli *et al.*, (2003) the behavior of the brace is in a way that the brace shows its good behavior in tension whereas the compression part should show the same behavior to avoid buckling during a cyclic loading.

Table 8.1. Capacity Values for Tension and Compression for BRBs (SAP 2000).

type	story No.	cross section	tension	compression
low rise	1	HE 100A	800	880
	2	HE 100A	800	880
	3	HE 100A	800	880
mid rise	1	HE 160 A	1471	1617
	2	HE 160 A	1471	1617
	3	HE 140 A	1190	1309
	4	HE 140 A	1190	1309
	5	HE 120 A	960	1056
	6	HE 120 A	960	1056
high rise	1	HE 160 A	1471	1617
	2	HE 160 A	1471	1617
	3	HE 160 A	1471	1617
	4	HE 160 A	1471	1617
	5	HE 140 A	1190	1309
	6	HE 140 A	1190	1309
	7	HE 140 A	1190	1309
	8	HE 140 A	1190	1309
	9	HE 120 A	960	1056
	10	HE 120 A	960	1056

8.2. Nonlinear Static Analysis Process

In this step the nonlinear static (pushover) process performed and the capacity curves corresponding to each configuration plotted. Desirably brace members show good behavior during the analysis process and no buckling happened when the brace started to get its compressive load. Thus higher capacities reached when the curves are compared with those of conventional braces.

8.2.1. Capacity Curves Implementing Buckling Restrained Braces

As expected the order of formation of plastic hinges starts from braces and no damage state occurs in structural elements such as columns and beams. As mentioned before the buckling does not occur in braces in compression and the brace continues until reaching the yield point in tension zone.

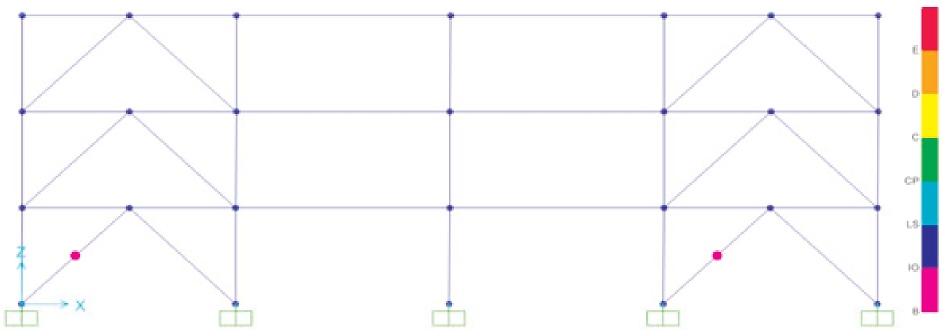

Figure 8.13. Formation of First Hinge in CBF Configuration (SAP 2000).

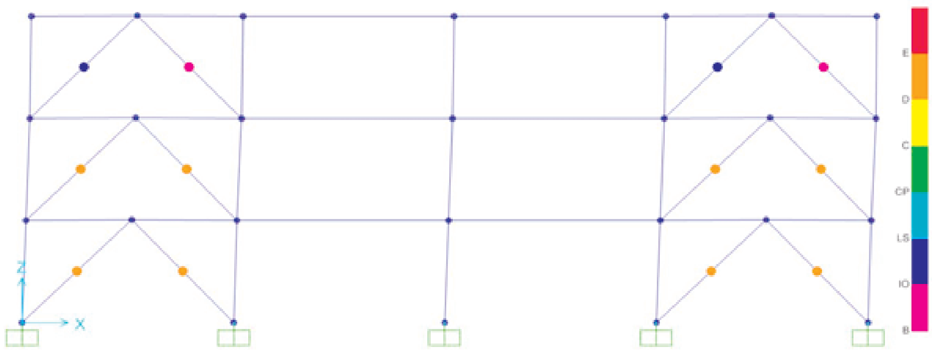

Figure 8.14. Plastic Mechanism of the Low Rise Structure with BRB.

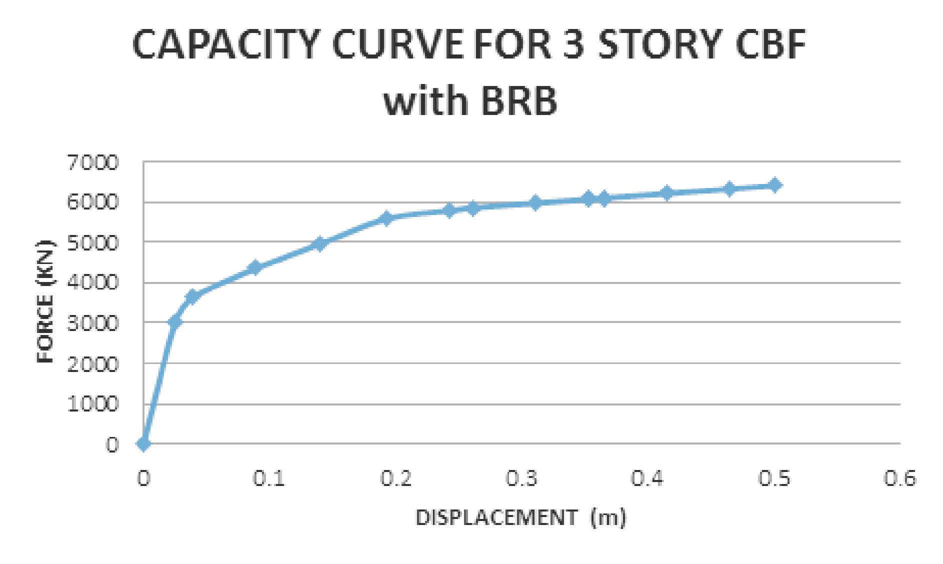

Figure 8.15. Capacity Curve for 3 Story (Low Rise) Structure with CBF Using BRBs.

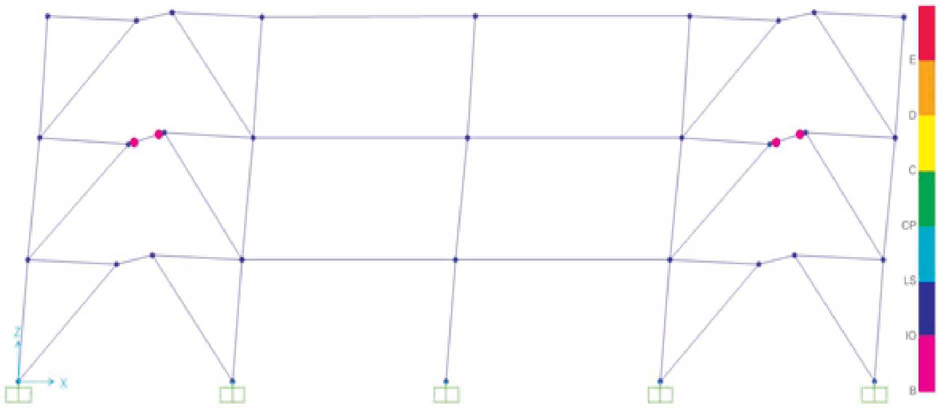

Figure 8.16. The Formation of Plastic Hinge in Link Element (SAP 2000).

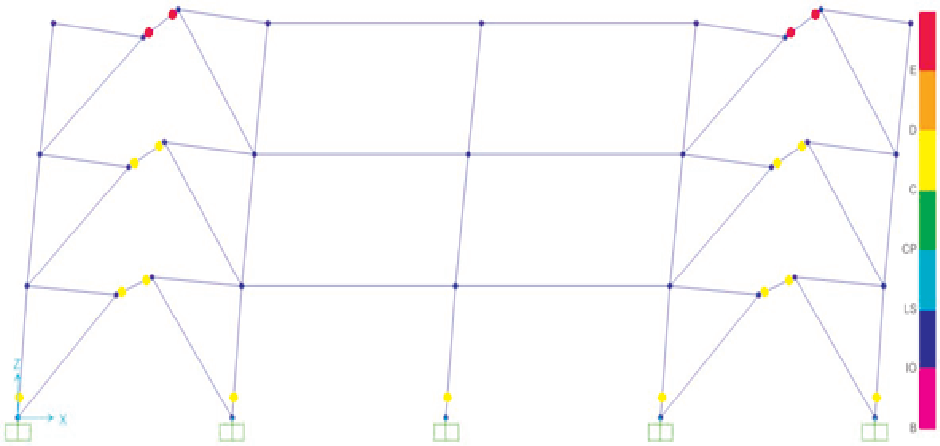

Figure 8.17. Plastic Mechanism for Low Rise EBF with Short Link Using BRBs.

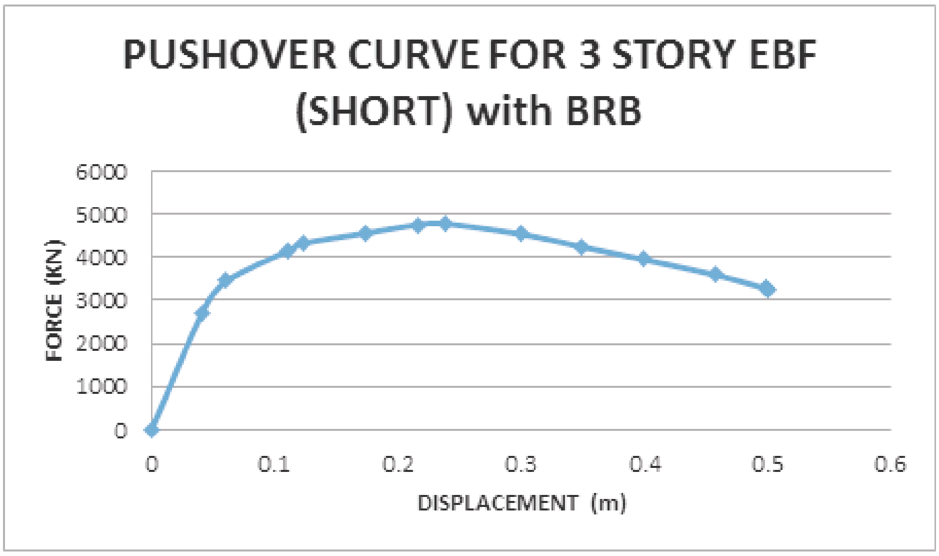

Figure 8.18. Capacity Curve for Low Rise EBF with Short Link with BRBs.

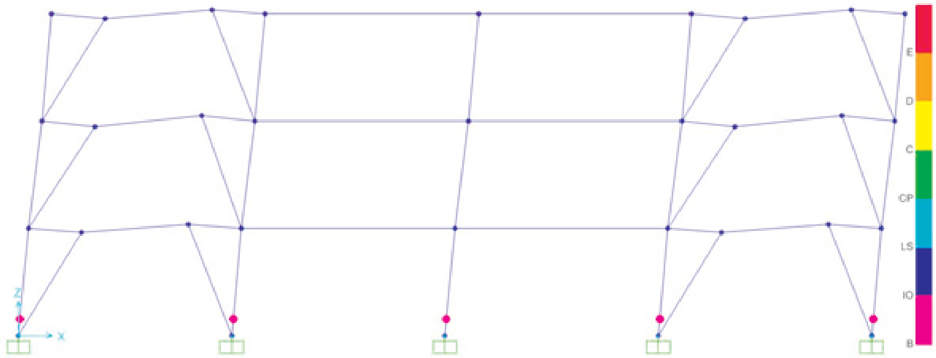

Figure 8.19. Formation of Plastic Hinge for EBF with Long Link Using BRBs (SAP 2000).

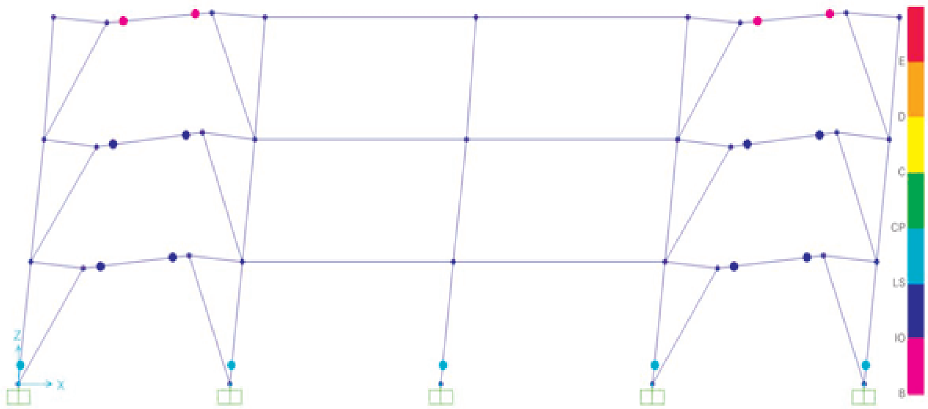

Figure 8.20. Plastic Mechanism of the Low Rise EBF with Long Link Using BRBs (SAP 2000).

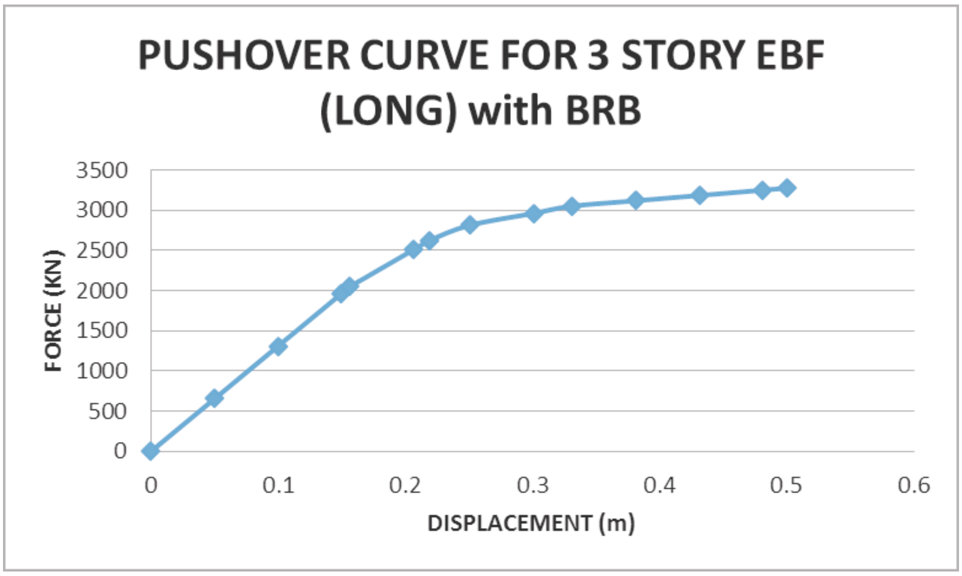

Figure 8.21. Capacity Curve for Low rise EBF with BRBs.

As shown in the capacity curves, the concentrically braced frame and eccentrically braced frame with long link demonstrate better results compared with eccentrically braced frame with short link. The concentrically braced frame shows more reliable and stable behavior than other configurations suggesting that this type of configuration is the best option for low rise buildings. The point for eccentrically braced frames is that the base columns fail during the analysis which is not expected. This problem might be stem from the fact that the brace members are oversized and need to be reduced in size to absorb the maximum energy transfers to structural members.

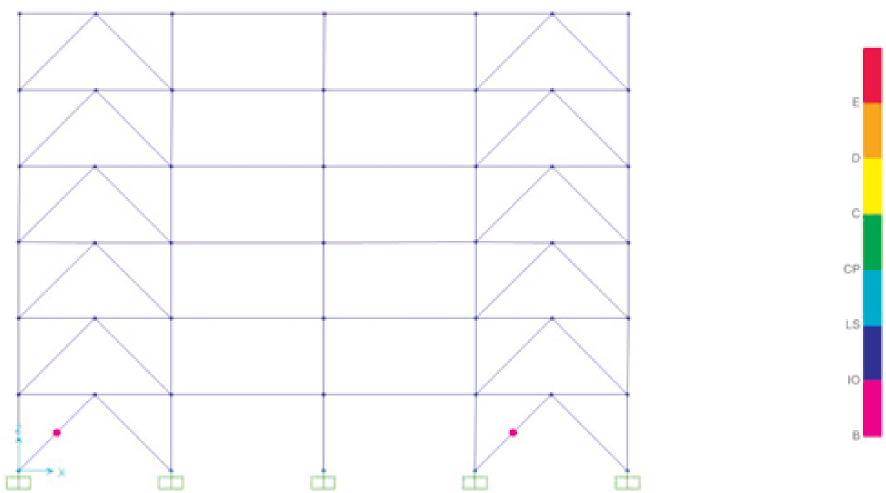

Figure 8.22. Formation of Plastic Hinge for Mid Rise CBF with BRBs.

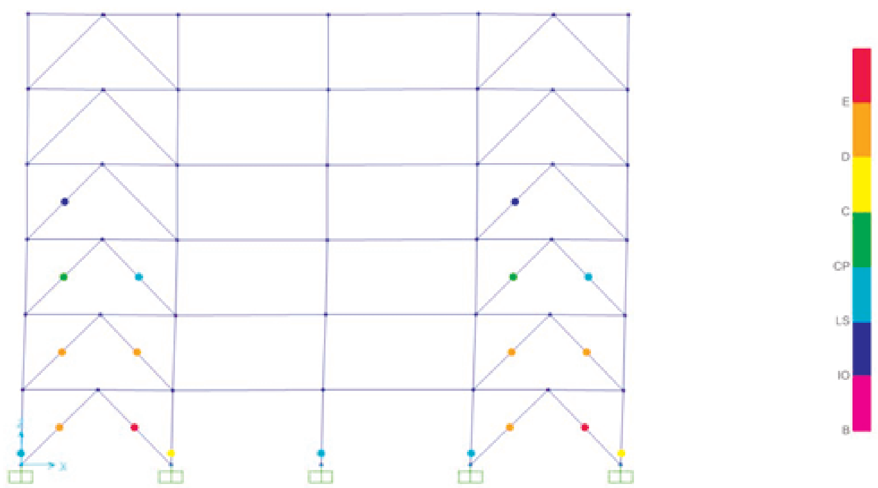

Figure 8.23. Plastic Mechanism for CBF with BRBs.

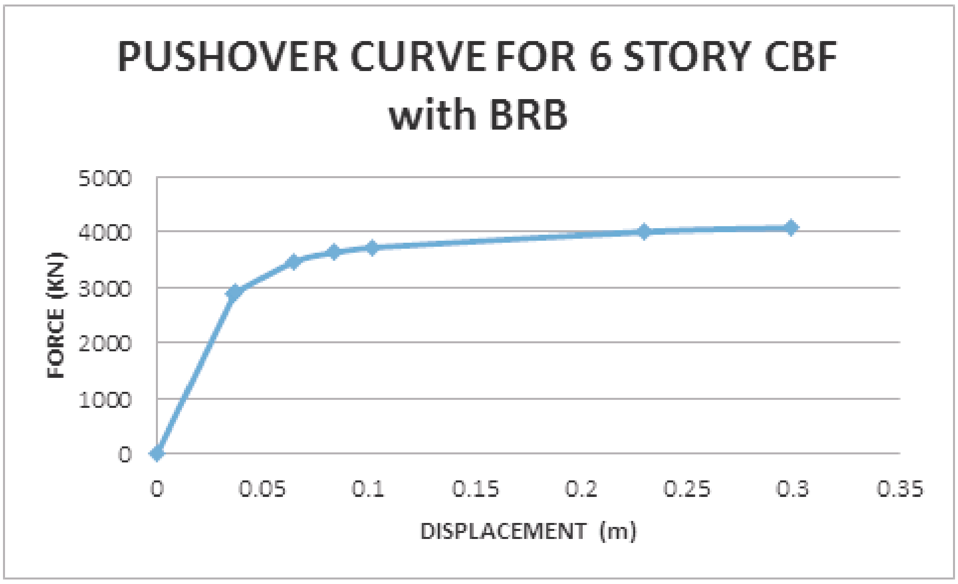

Figure 8.24. Capacity Curve for Mid Rise CBF with BRBs.

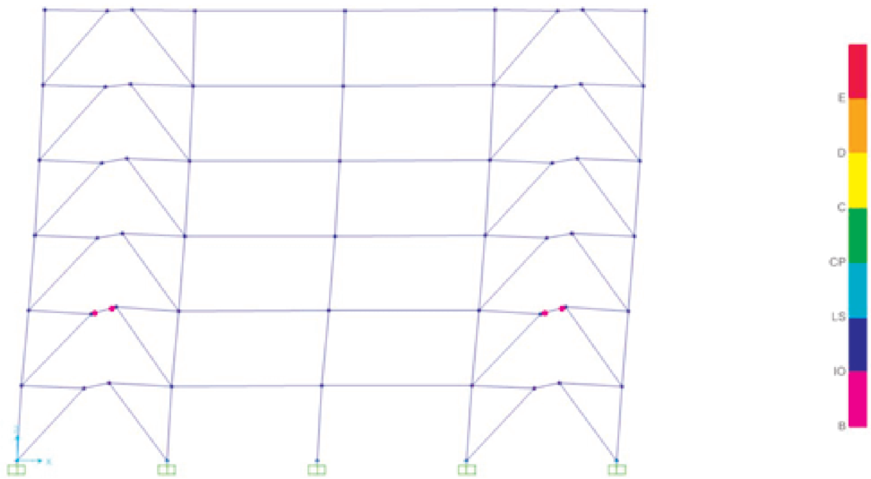

Figure 8.25. Formation of the Plastic Hinge for Mid Rise EBF with Short Link Using BRBs.

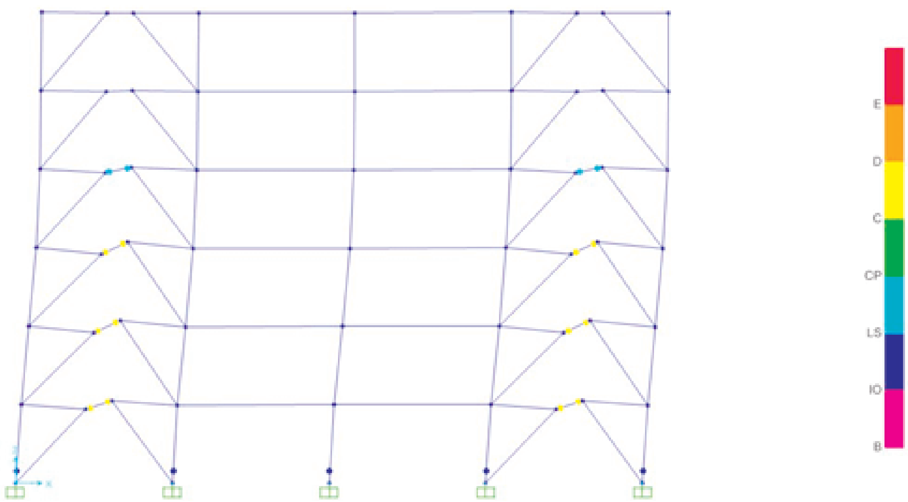

Figure 8.26. Plastic Mechanism for Mid Rise EBF with Short Link Using BRBs.

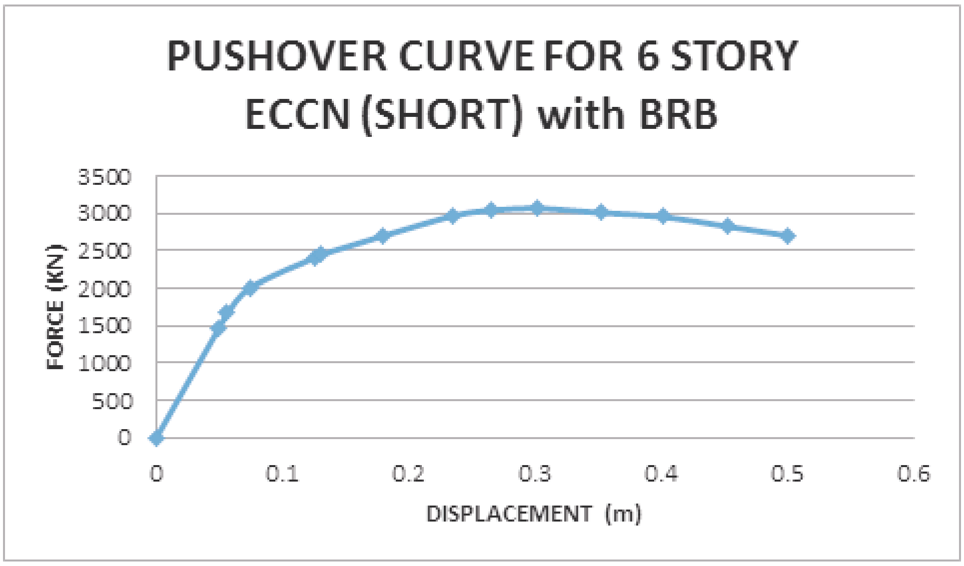

Figure 8.27. Capacity Curve for Mid Rise Building with EBF with Short Link Using BRBs.

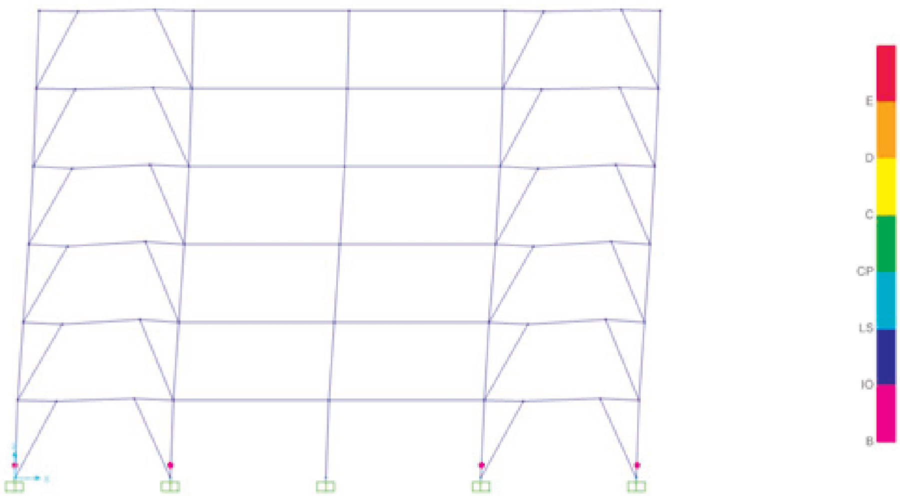

Figure 8.28. Formation of Plastic Hinges for Mid Rise EBF with Long Link Using BRBs.

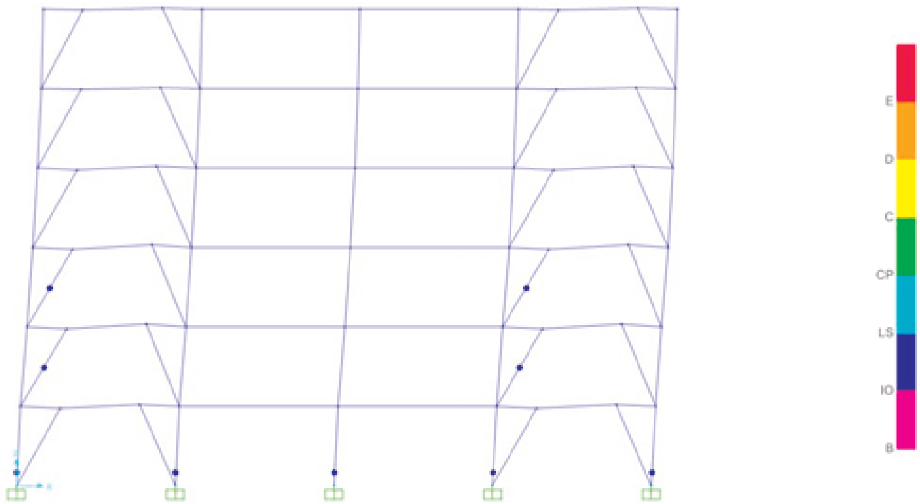

Figure 8.29. Plastic Mechanism for Mid Rise Building with EBF with Long Link Using BRBs.

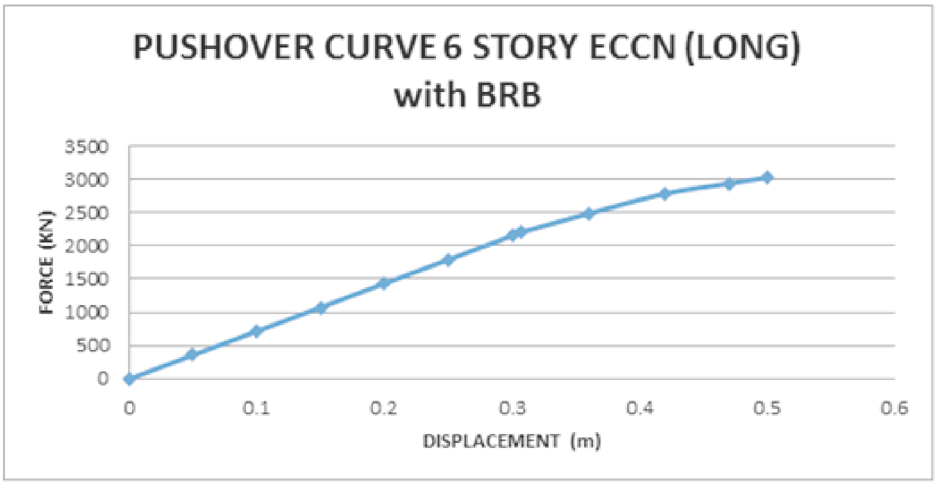

Figure 8.30. Capacity Curve for Mid Rise EBF with Long Link Using BRBs.

The results for six story building (mid rise) is interestingly close to that of three story building and capacity curves in concentrically braced frame and eccentrically braced frame with long link show reasonable increase whereas the capacity curve for eccentrically braced frame with short link ascends up to a level and after a short time starts to go down and the capacity decreases slowly. This descending is because of the shear forces form in the link element which results in shear failure and end up with the strain degradation and decrease in the global capacity of the structure. The same problem continues in this case as well and the base columns for both eccentrically braced frames fail during the pushover analysis.

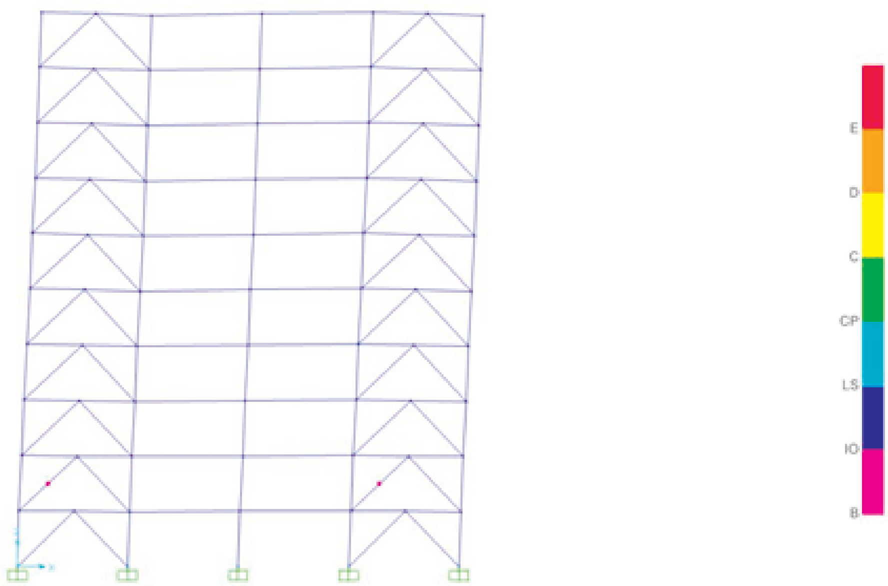

Figure 8.31. Hinge Formation of High Rise CBF with BRBs.

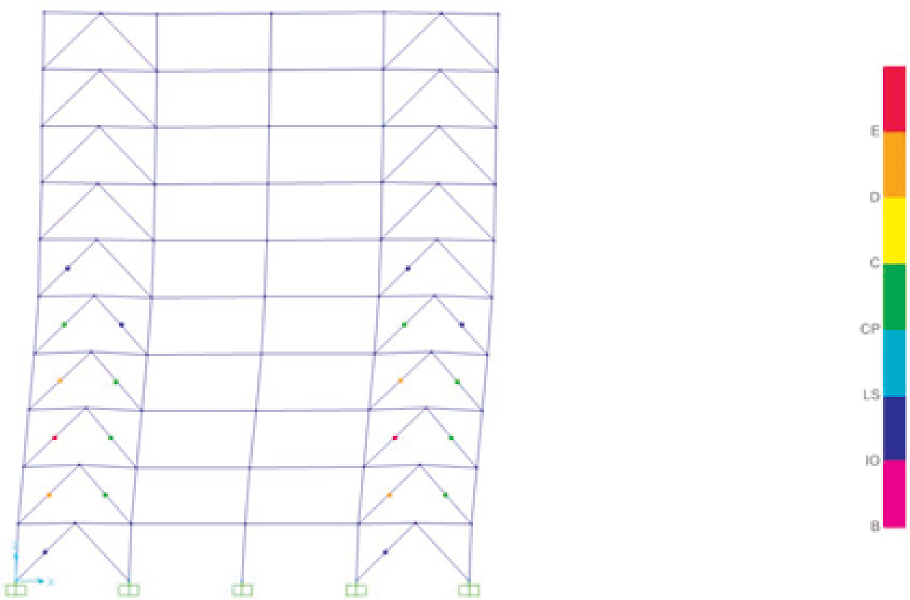

Figure 8.32. Plastic Mechanism for High Rise CBF with BRBs.

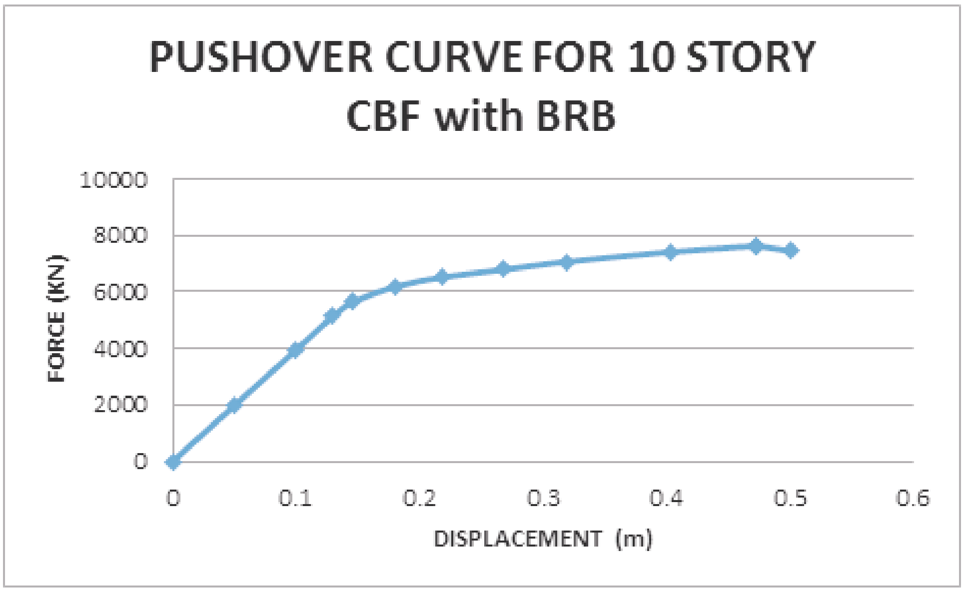

Figure 8.33. Capacity Curve for High Rise CBF Structure with BRBs.

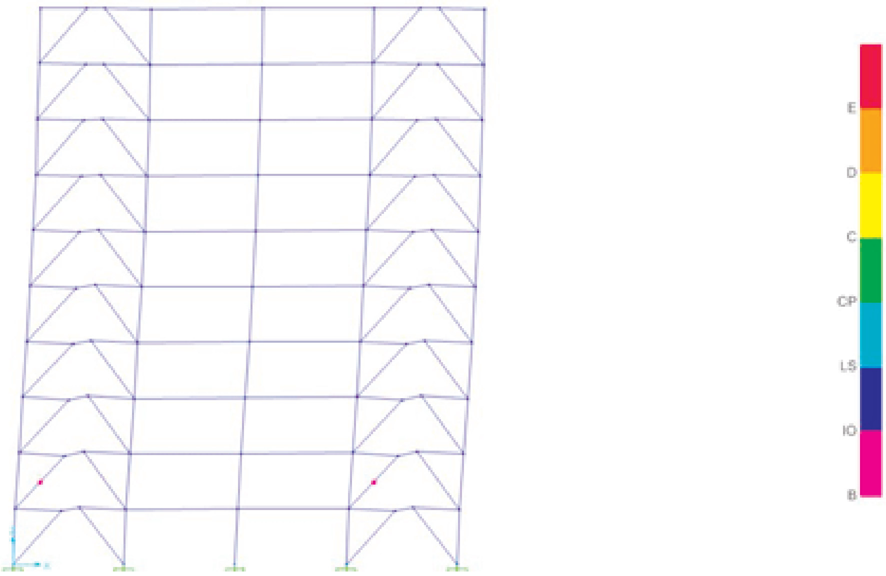

Figure 8.34. Hinge Formation for High Rise EBF with Short Link Using BRBs.

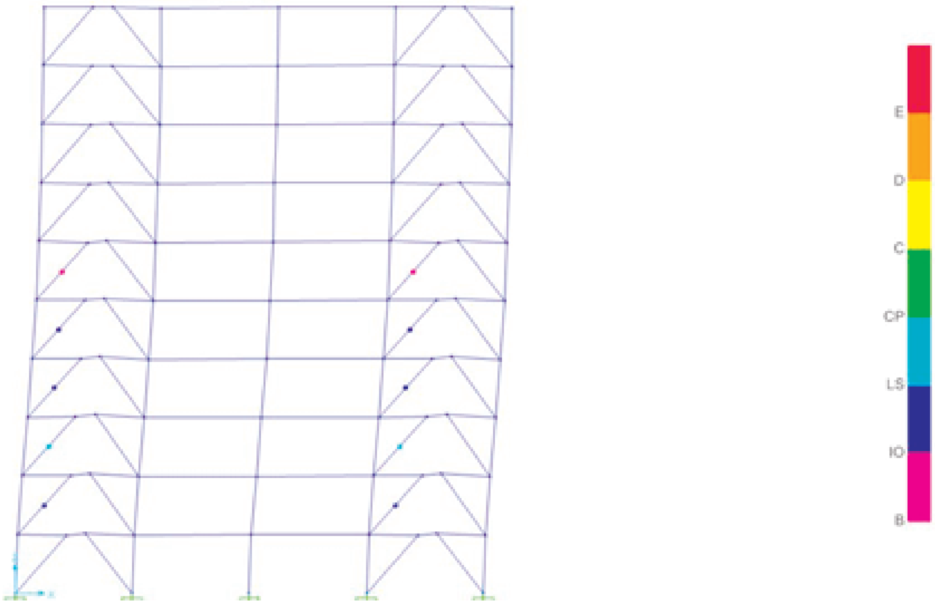

Figure 8.35. Plastic Mechanism for High Rise EBF with Short Link Using BRBs.

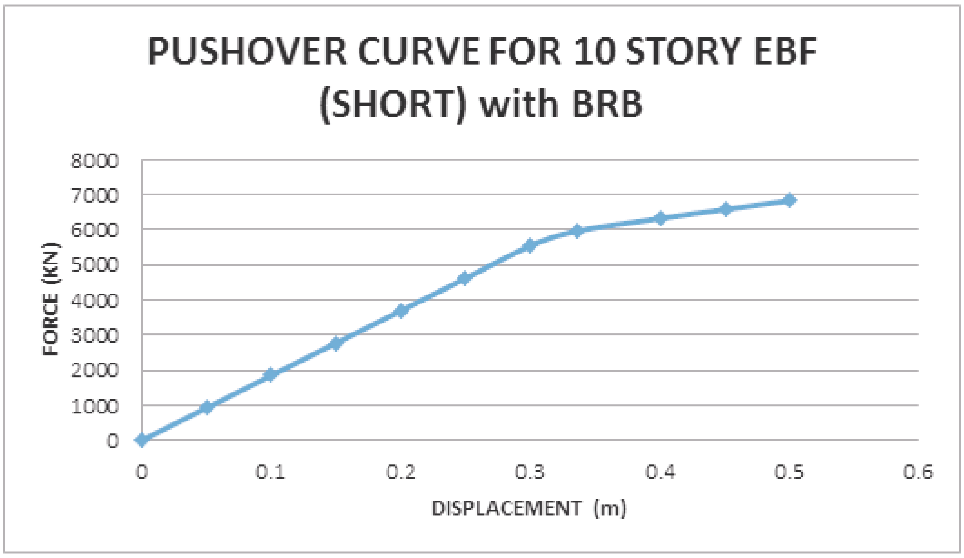

Figure 8.36. Capacity Curve for High Rise EBF with Short Link Using BRBs.

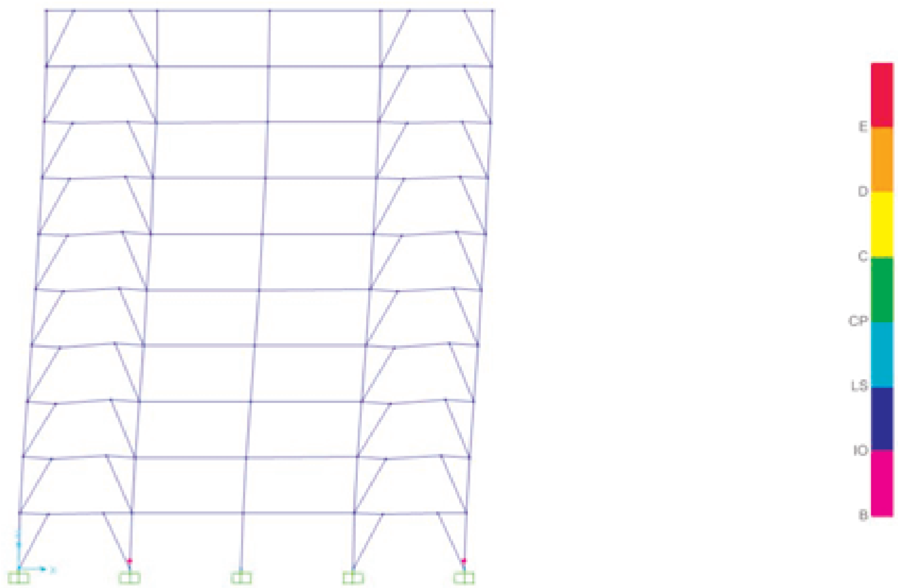

Figure 8.37. Hinge Formation for High Rise EBF with Long Link Using BRBs.

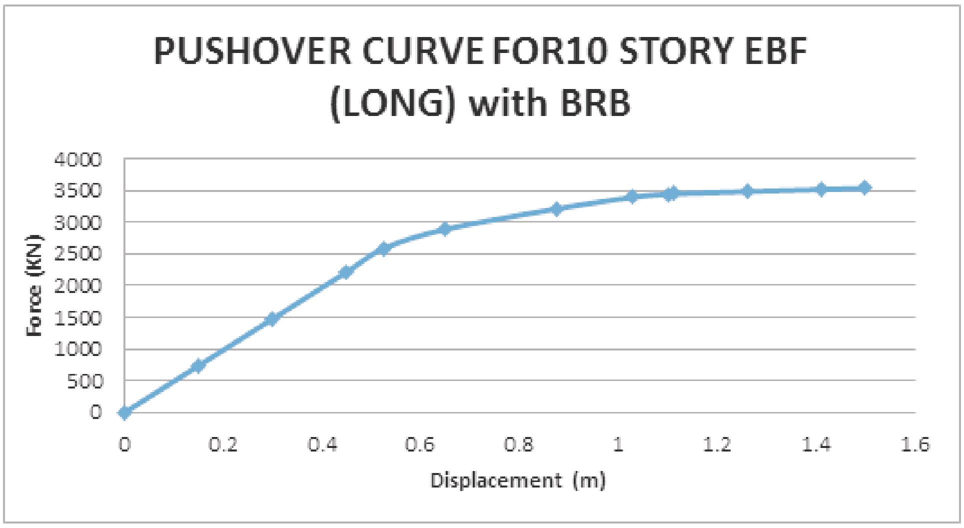

Figure 8.38. Capacity Curve for High rise EBF with Long Link Using BRBs.

As shown in the pictures the best performance is by the eccentrically braced frame with long link in which the structure does not reach its plastic state and remain elastic during the loading part. The only problem with this configuration is the formation of plastic hinges within the base columns which drives the structure to failure. The building with eccentrically braced frame with short links remains elastic until a certain point and then the plastic hinge forms within the brace member which is desired and a substantial increase happens in the global resisting system of the building. However, in the building with concentrically braced frame, the resisting system has a good behavior until the point that the hinges start to rotate and then the force resisting performance continues until the point that the curve starts dropping down and ends up with strain degradation.

The difference between the conventional bracing system and BRBF highlights when the capacity curves obtained from both configurations compared to each other. The capability of the resisting lifts up substantially by using the BRB members which makes the building to come to higher level of capacity range and longer and stable plastic phase. As another advantage, the brace as it's supposed to act is to not buckling in compression which is one of the conventional brace's negative points in seismic activities which leads the structure to a global buckling and eventually failure. In addition to what mentioned above, the usage of BRB members is cost effective as well. By replacing the conventional braces with BRB's the size of structural elements such as columns and beams reduce considerably. Accordingly, the cost of the construction of a building reduces to lower amounts.

8.3. Nonlinear Dynamic Analysis Results

In this part of the study results of the nonlinear dynamic analyses are presented and the results are compared in case of maximum displacement and inter-story drift obtained from SAP 2000 analysis program. Figure 8.39 shows the comparison of the three different configurations examined in this study according to FEMA 356 provision. The design spectrum specified in TEC is used to assess the performance of the structures with concentrically braced, eccentrically braced with short link and eccentrically braced frames. The blue colored curve represents the concentrically braced frame's displacement which indicates with CBF in this study.

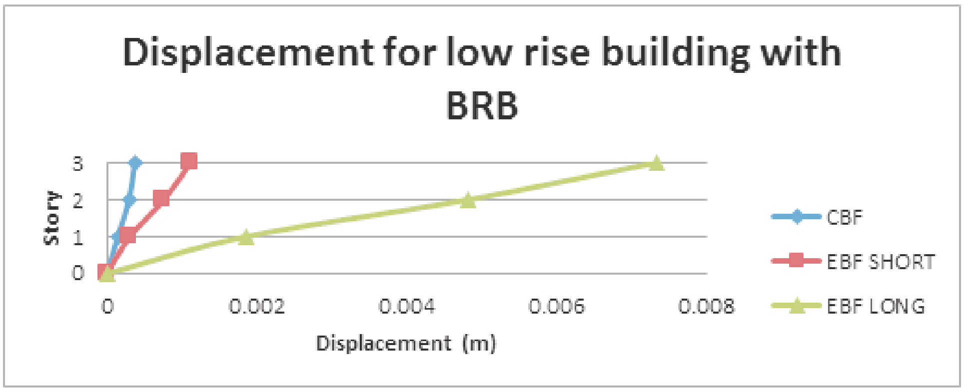

Figure 8.39. Maximum Displacement for Low Rise CBF, EBF (short link) and EBF (Long Link).

The red colored curve represents the eccentrically braced frame with short link's displacement curve which indicated with EBF SHORT and the green colored curve is for eccentrically braced frame with long link indicated with EBF LONG. The results show that the concentrically braced frame has the maximum displacement around 0.0004 m on the top roof level which is negligible and the eccentrically braced frame hast the displacement of 0.001 m which is very close to concentrically braced frame. Interestingly the eccentrically braced frame has the displacement of 0.007 m which suggests that the eccentrically braced frame with long link shows more ductility than other configurations. The reliable configuration for low rise building is eccentrically braced frame with long link. Figure 8.40 exhibits the difference between maximum inter-story drift of three considered configurations for low rise buildings. The figure shows that the drift is almost the same for first story.

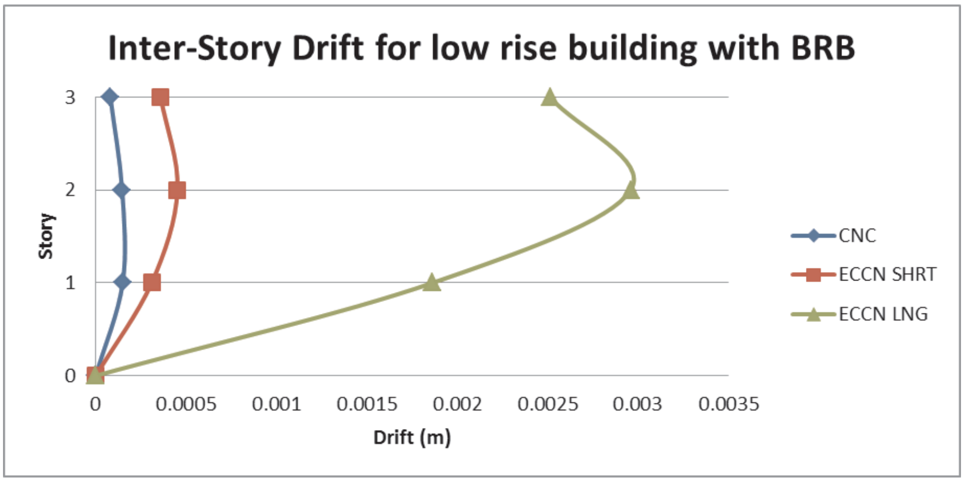

Figure 8.40. Maximum Inter-Story Drift for Low Rise CBF, EBF (Short Link) and EBF (Long Link).

However, the change starts from the second floor in which the eccentrically framed frames show the drift equal to approximately 0.001 and -0.001 for frames with short link and long link respectively. This variety in drift indicates that the eccentrically braced frames have better behavior compared with the concentrically braced frame.

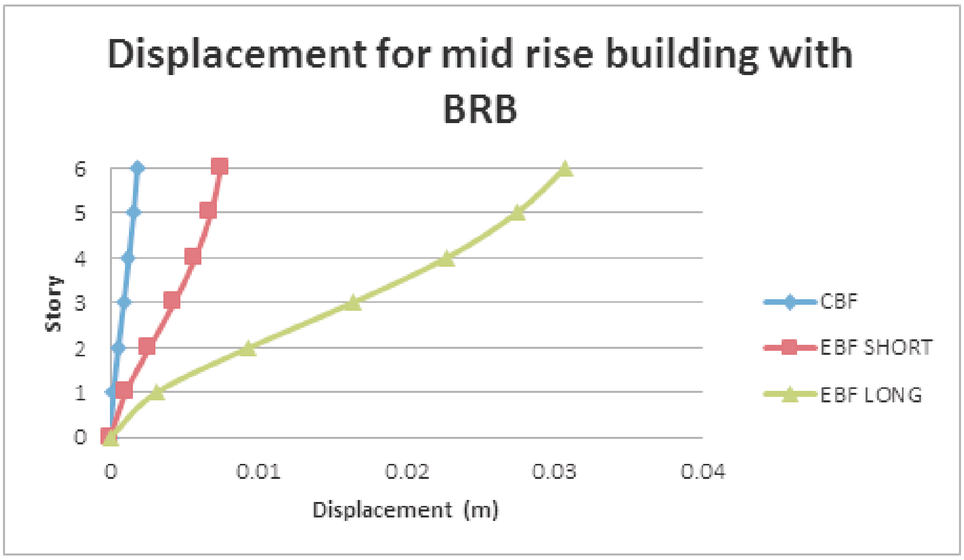

Figure 8.41. Maximum Displacement for Mid Rise Building CBF, EBF (Short) and EBF (Long).

In Figure 8.41 the maximum displacement for mid rise building in concentrically braced frame and eccentrically braced frame with short link and eccentrically braced frame with long link is shown. As illustrated in the figure the resulted displacements are the same as low rise building in which the concentrically braced frame shows the displacement very close to zero on top roof level and the eccentrically braced frame with short link shows a displacement of 0.007 m and it is 0.03 m for eccentrically braced frame with long link. As well as maximum displacement the maximum inter-story drift is exhibited in Figure 8.42 in which the concentrically braced frame drifts much more far when it is compared with eccentrically braced frames. Both configurations in eccentrically braced frame drift in an almost same path.

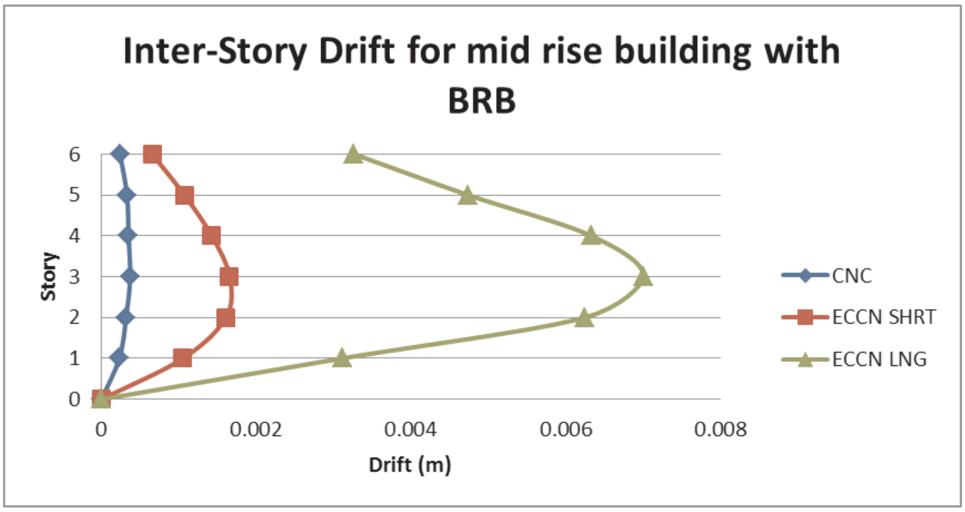

Figure 8.42. Maximum Drift for Mid Rise Building with CBF, EBF (Short) and EBF (Long).

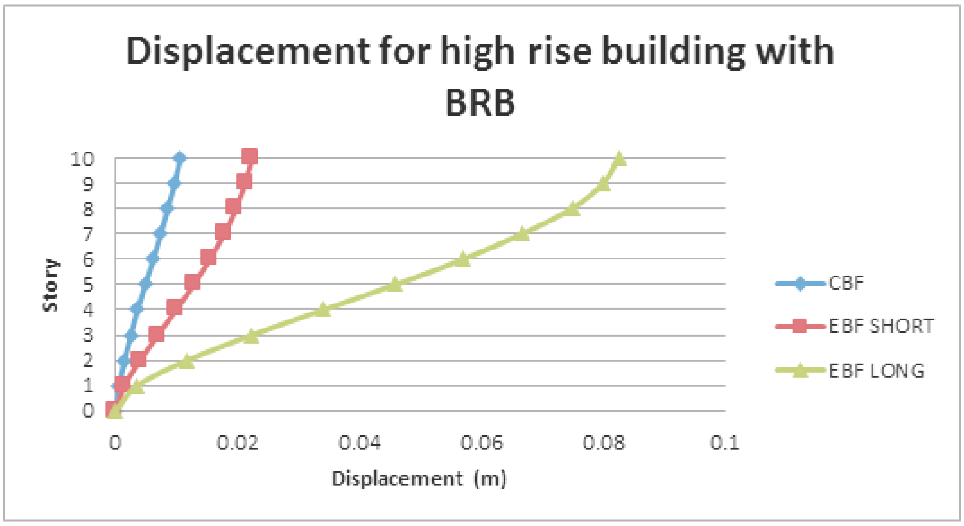

Figure 8.43. Maximum Displacement for High Rise Building with CBF, EBF (Short) and EBF (Long).

As can be seen from Figure 8.43, the displacement follows the same differences spotted in the above mentioned low and mid rise buildings. Concentrically braced frame has the lowest displacement and the eccentrically braced frame with short link come after that and the eccentrically braced frame with long link has the highest displacement on the top roof level. In the following figure the maximum drift corresponding to three different configurations are plotted considering the data obtained from nonlinear dynamic analysis. The figure 8.44 has nothing different with low and mid rise buildings in the drift ratios.

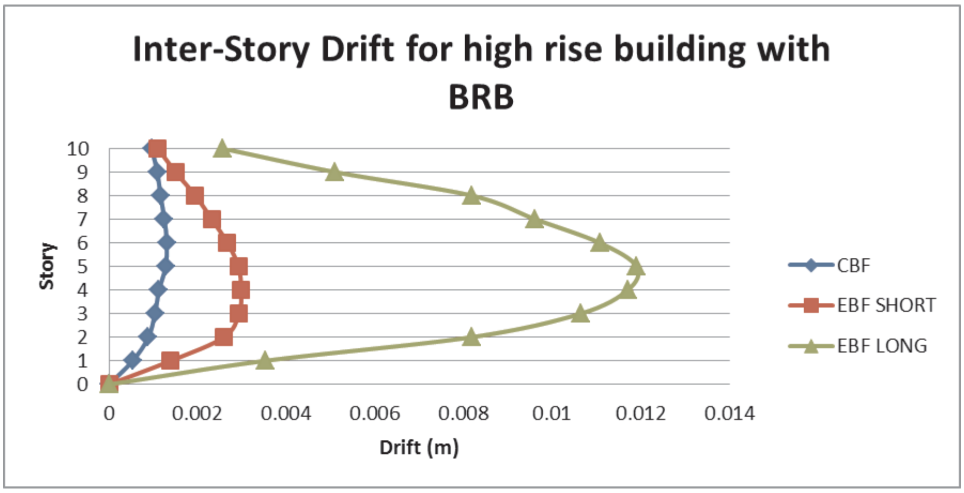

Figure 8.44. Maximum Inter-Story Drift for High Rise Building with CBF, EBF (Short) and EBF (Long).

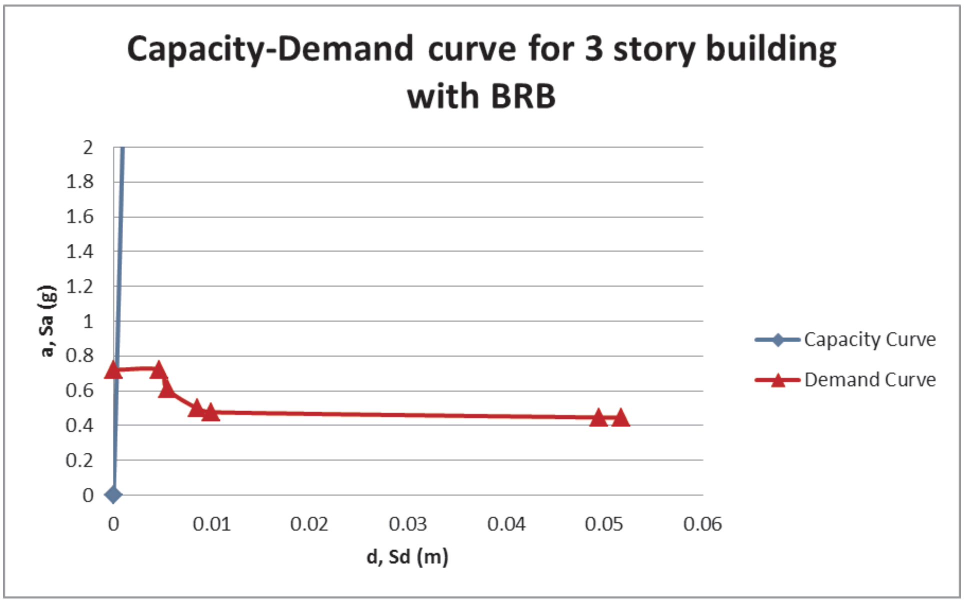

Figure 8.45. Capacity-Demand Curve for Low Rise CBF Suilding with BRBs.

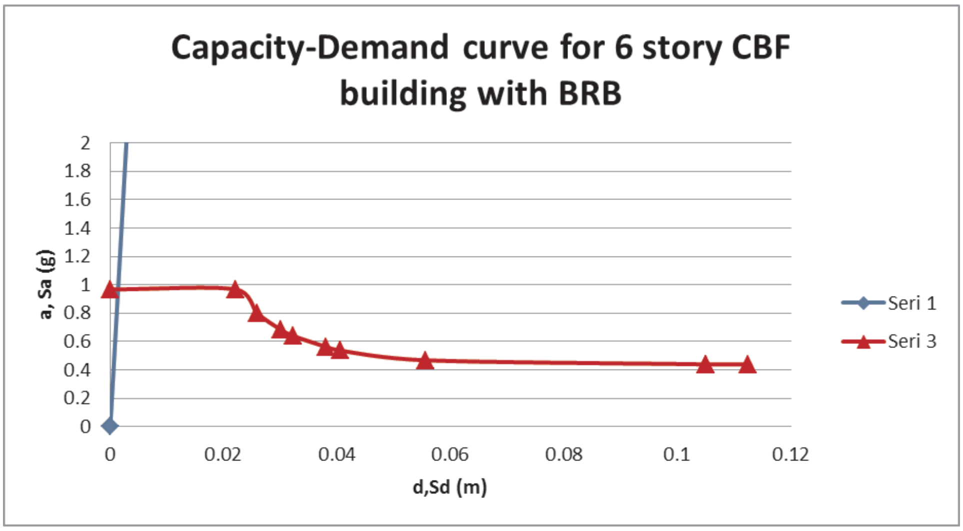

Figure 8.46. Capacity-Demand Curve for Mid Rise CBF with BRBs.

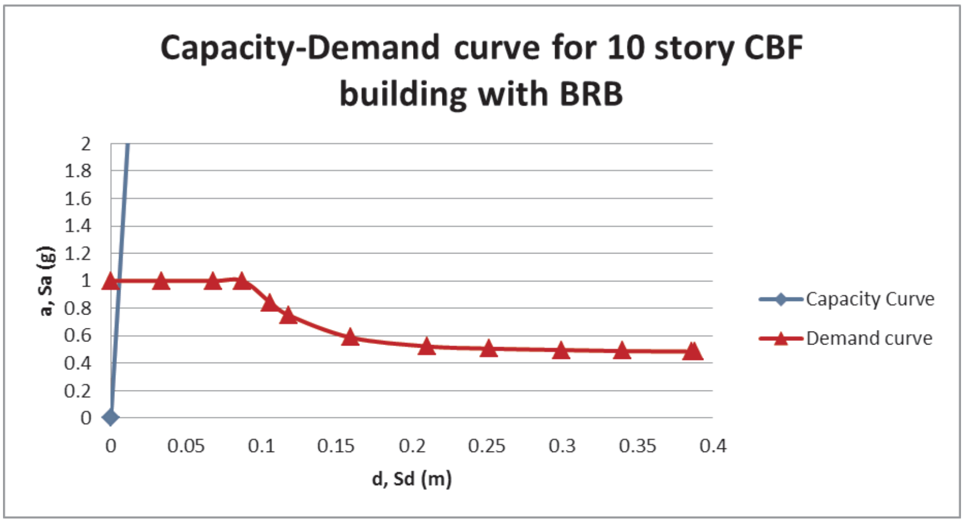

Figure 8.47. Capacity-Demand Curve for High Rise CBF with BRBs.

From figures above it can be seen that the capacity of the structure reaches to much higher values and can resist stronger loads than what specified in the earthquake code. This is one of advantageous aspects of the using of BRBs in steel structural constructions.

9. CONCLUSION AND RECOMMENDATIONS

9.1. Summary

In this study three type of buildings to have a good understanding of implementation and use of buckling restrained braces within 3 story (low rise), 6 story (mid rise) and 10 story (high rise) buildings were examined in line with FEMA 356 provisions. The nonlinear dynamic procedure was performed to assess the behavior of the structures under static (pushover) and dynamic (response spectrum) loadings. In the first step the design procedure was done according to Turkish Earthquake Code 2007. The main goal of the study is to assess the seismic behavior of the buildings with three different configurations such as concentrically braced frames, eccentrically braced frames with short link and eccentrically braced frames with long link to choice the best configuration for implementation of the buckling restrained braces. In the next step of the study the nonlinear static and dynamic procedures performed utilizing SAP 2000 analysis program and the capacity curves and maximum displacement and drift corresponding to each of the configurations obtained. As the aim of the study obtained values for displacement and drift plotted in a single graph to have a comparison among the relative results.

9.2. Conclusion

The results of this study are diverse for low, mid and high rise buildings. In three story (low rise) building the capacity curves show that the concentrically braced frame has a better and more reliable performance than other two configurations in which higher capacities can be reached with more stable curve. In the case of spectral displacement the CBF has lesser displacement than others.

For six story (mid rise) buildings the capacity curve for CBF shows a stable rise until the point that the hinge forms and then the hinges develop within other stories and the maximum displacement does not go further than 0.3 meters where the hinges develop in base columns and the structure experiences failure.

The same process happens in ten story (high rise) building. Under pushover procedure the concentrically braced frame shows good behavior and the structure reaches high values such as 7000 KN of base shear. However, in that stage the building comes to its failure point and the curve starts to dropping. The maximum displacement which was reached is 0.48 cm which is a bit lower than the target displacement which is 0.5 cm.

The eccentrically braced frame demonstrates better behavior compared with concentrically braced frame. The same values are reached by the structure but in this case the braced frame tends to mount to higher values over 7000 KN. The first problem in this case is that plastic hinges develop in base columns which is not desirable. The second problem is that the bracing system is not working properly which means that the only element which works as lateral resisting system is the beam links. The reason for this might be the size of braces which are higher than the capacity needed for acting as lateral resisting system and dissipation.

According to what discussed above, the performance of the structure under the seismic loading ends up with failure due to the formation of hinges in base columns which is not desirable in eccentrically braced frames. This action means that the braces are not contributing in resisting system and need to be reduced in size. This is the economic side of the eccentrically braced frames aside from architectural benefits because of the availability of the place for architectural designing.

APPENDIX A: STRUCTURE BEHAVIOR AND CAPACITY CURVES FOR LOW

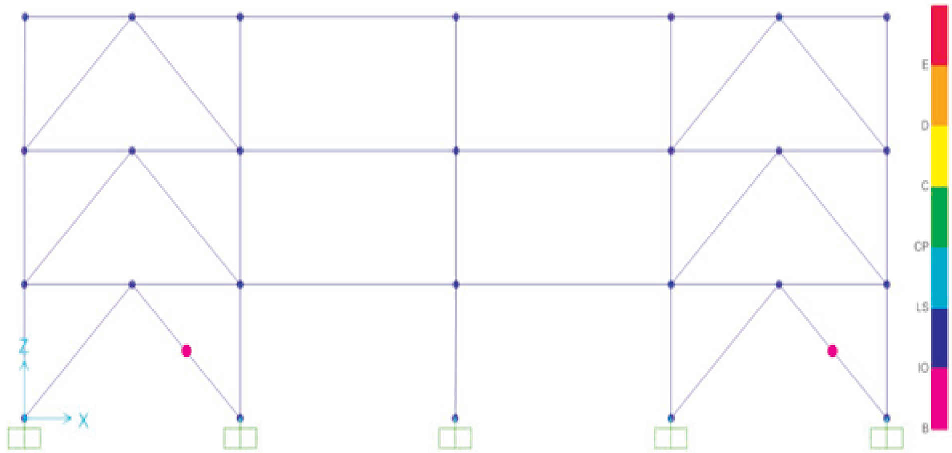

Figure A.1. Formation of the Plastic Hinge for Mid Rise EBF with Short Link Using BRBs.

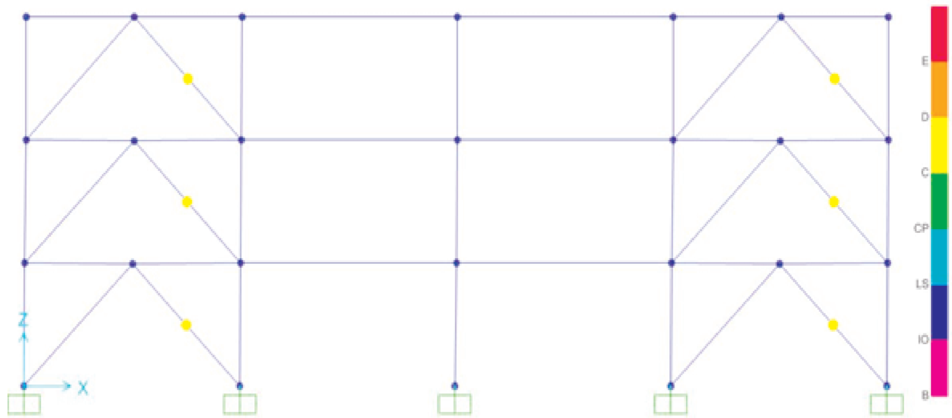

Figure A.2. Plastic Mechanism for Mid Rise EBF with Short Link Using BRBs.

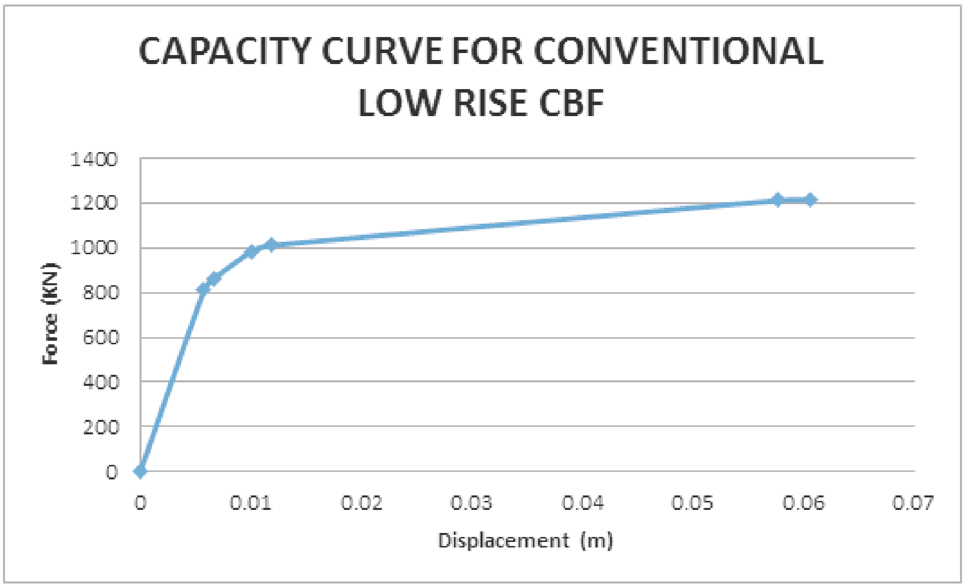

Figure A.3. Capacity Curve for Mid Rise Building with EBF with Short Link Using BRBs.

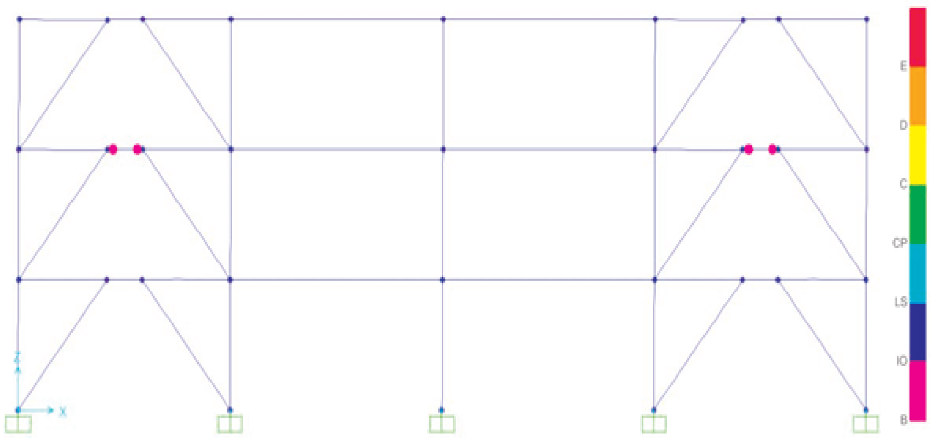

Figure A.4. Formation of the Plastic Hinge for Mid Rise EBF with Short Link Using BRBs.

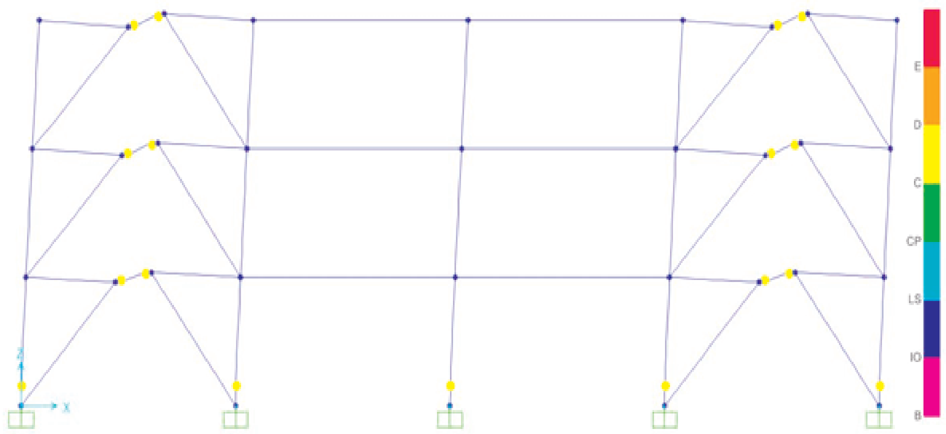

Figure A.5. Plastic Mechanism for Mid Rise EBF with Short Link Using BRBs.

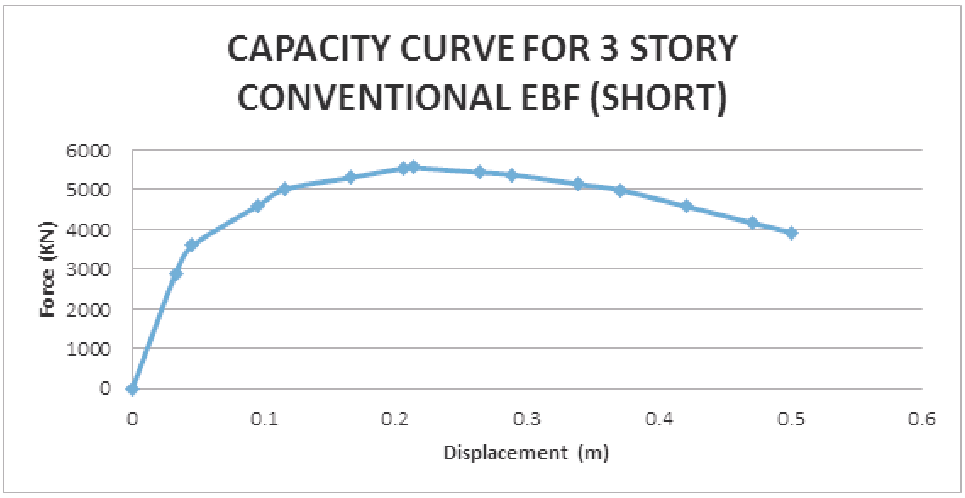

Figure A.6. Capacity Curve for Mid Rise Building with EBF with Short Link Using BRBs.

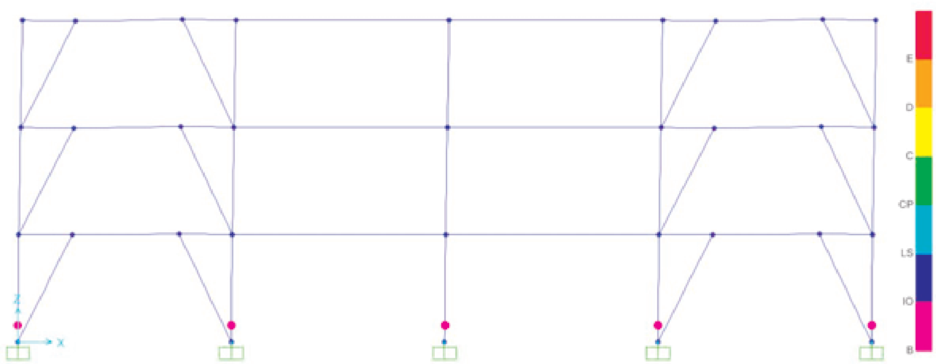

Figure A.7. Formation of the Plastic Hinge for Mid Rise EBF with Short Link Using BRBs.

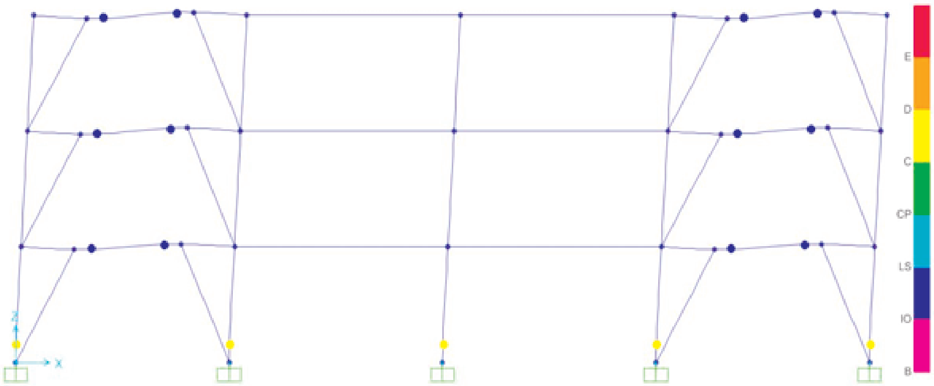

Figure A.8. Plastic Mechanism for Mid Rise EBF with Short Link Using BRBs.

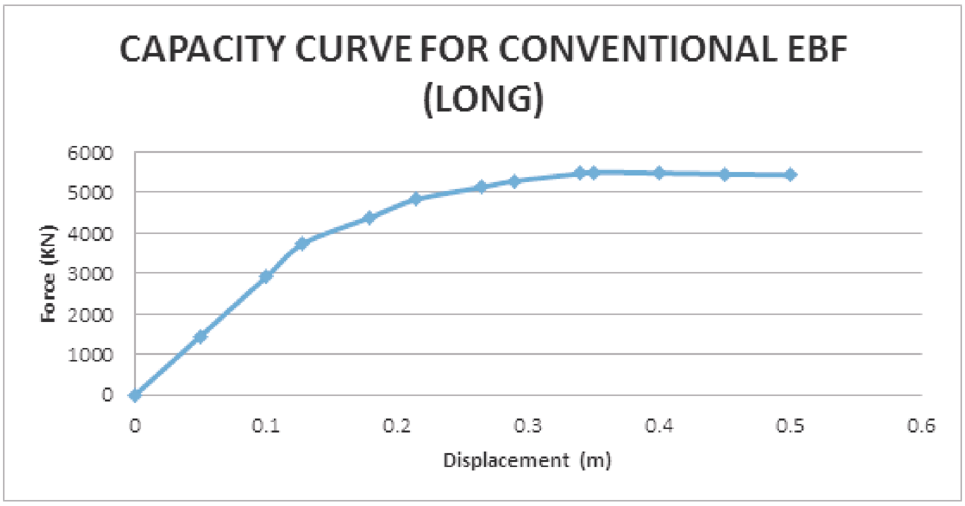

Figure A.9. Capacity Curve for Mid Rise Building with EBF with Short Link Using BRBs.

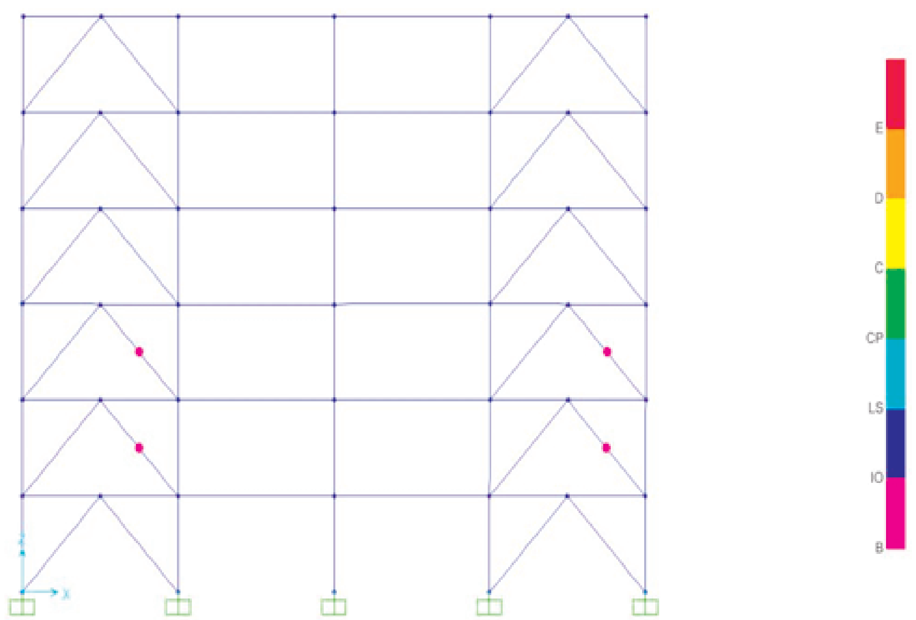

Figure A.10. Formation of the Plastic Hinge for Mid Rise EBF with Short Link Using BRBs.

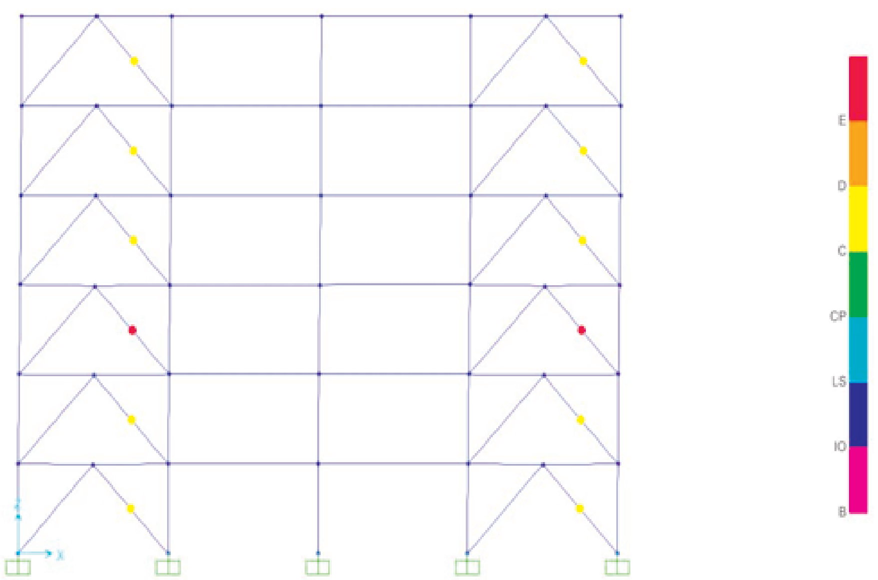

Figure A.11. Plastic Mechanism for Mid Rise EBF with Short Link Using BRBs.

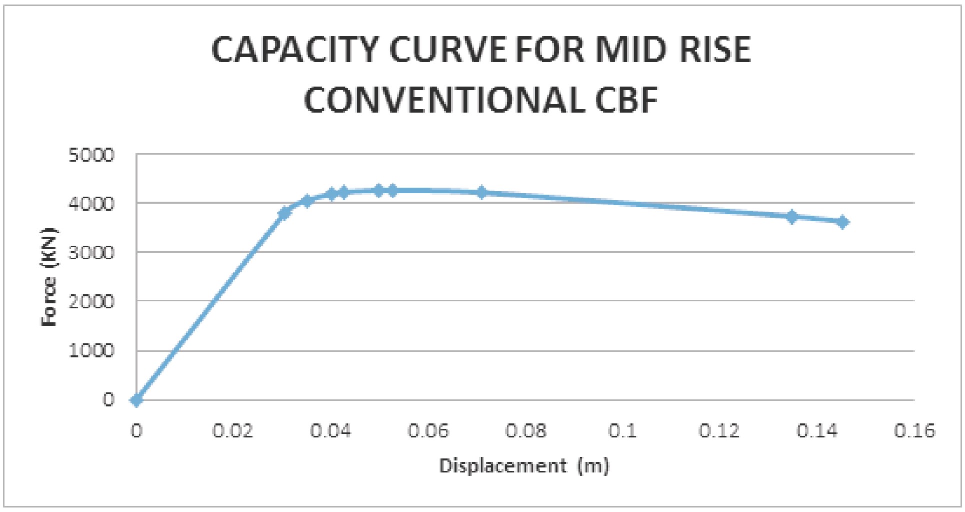

Figure A.12. Capacity Curve for Mid Rise Building with EBF with Short Link Using BRBs.

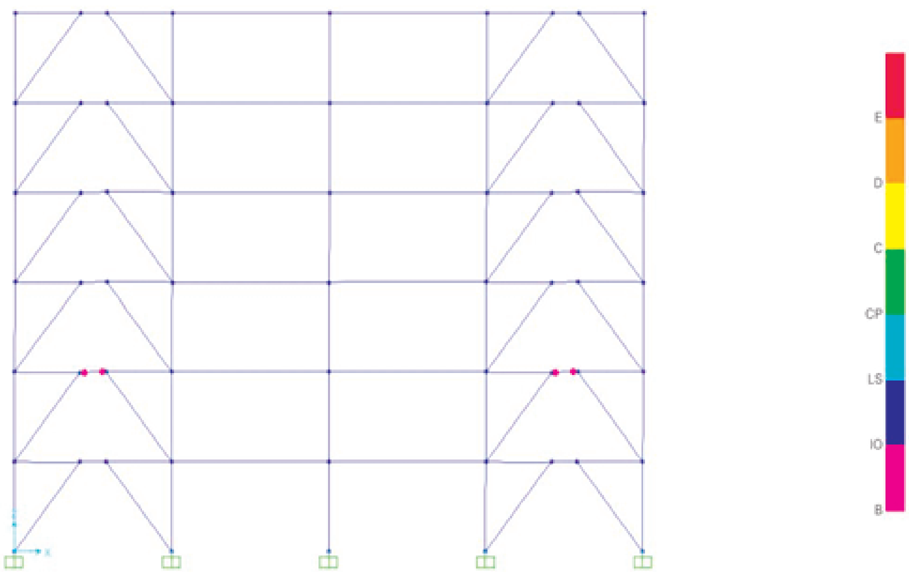

Figure A.13. Formation of the Plastic Hinge for Mid Rise EBF with Short Link Using BRBs.

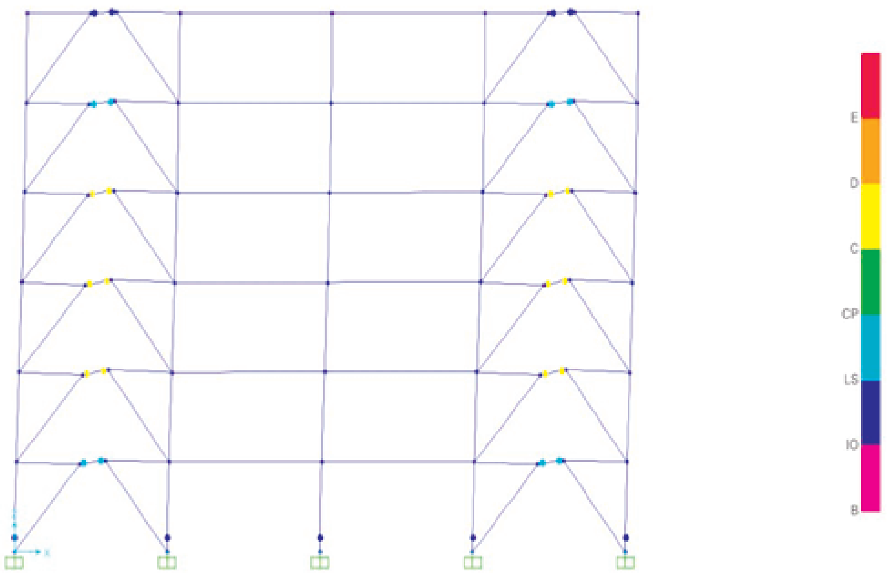

Figure A.14. Plastic Mechanism for Mid Rise EBF with Short Link Using BRBs.

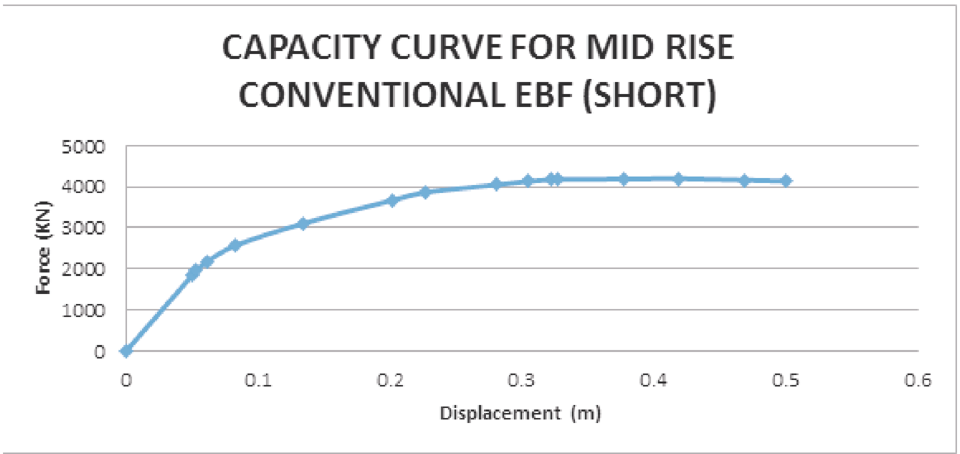

Figure A.15. Capacity Curve for Mid Rise Building with EBF with Short Link Using BRBs.

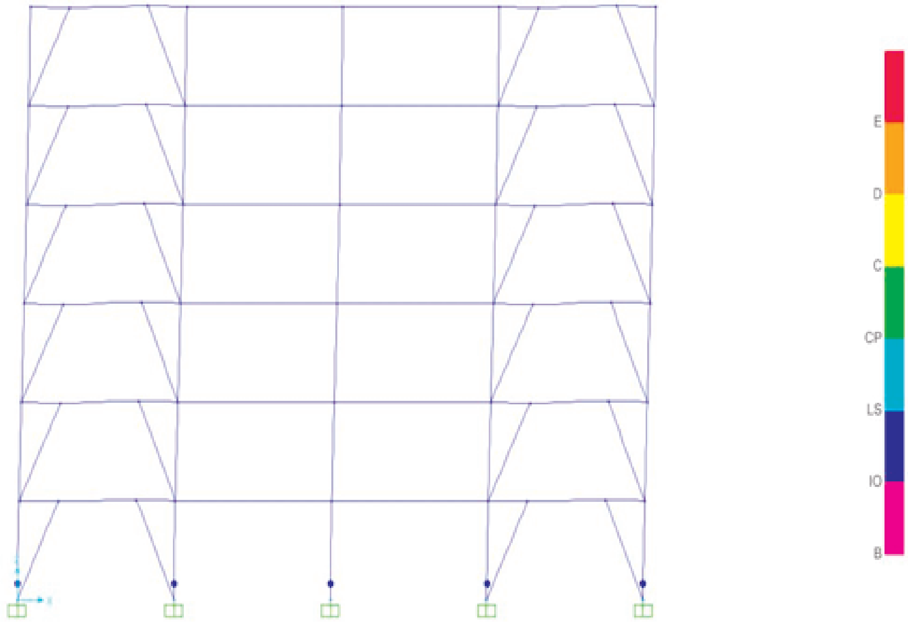

Figure A.16. Formation of the Plastic Hinge for Mid Rise EBF with Short Link Using BRBs.

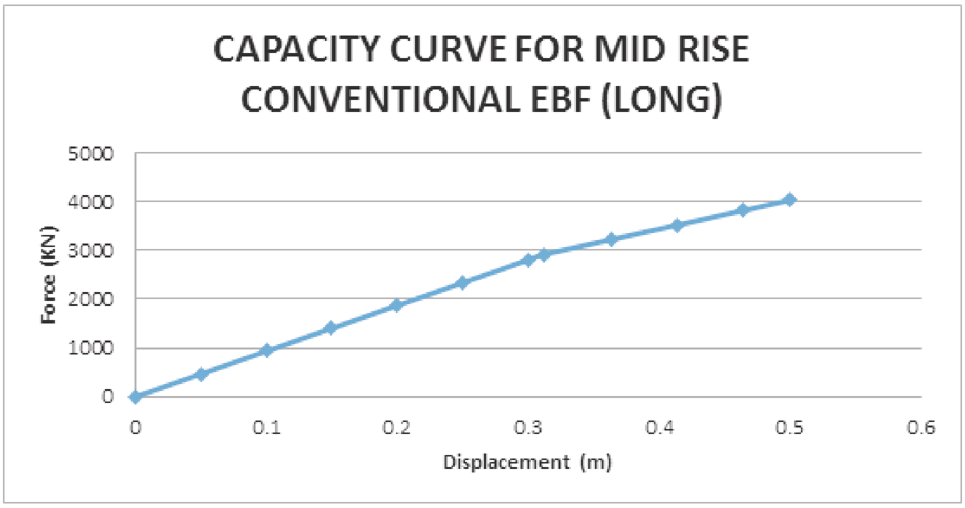

Figure A.17. Capacity Curve for Mid Rise Building with EBF with Short Link Using BRBs.

REFERENCES

Andrews, B.M., L.A. Fahnestock, J. Song, 2009, "Ductility Capacity Models for Buckling Restrained Braces", Elsevier Science Ltd. *Journal of Constructional Steel Research*, Vol. 65, pp. 1712-1720.

Andrews, B.M., J. Song, L.A. Fahnestock, 2009, "Assessment of Buckling Restrained Braced Frame Reliability Using an Experimental Limit-State Model and Stochastic Dynamic Analysis", *Journal of Earthquake Engineering and Engineering Vibration*, Vol. 8, pp. 373-385.

Balling, R.J., L.J. Balling, P.W. Richards, 2009, "Design of Buckling Restrained Braced Frames Using Nonlinear Time History Analysis and Optimization", *Journal of Structural Engineering*, Vol. 135, pp. 461-468.

Bruneau, M., C.M. Uang, A. Whittaker, 2001, "Ductile Design of Steel Structures", *Mc Graw Hill*, New York.

Choi, H., J. Kim, 2006, "Energy-Based Seismic Design of Buckling-Restrained Braced Frames Using Hysteretic Energy Spectrum", Elsevier Ltd. *Journal of Engineering Structures*, Vol. 28, pp. 304-311.

Clark, P.W., K. Kasai, I.D. Aiken, I. Kimura, 2000, "Evaluation of Design Methodologies for Structures Incorporating Steel Un-bonded Braces for Energy Dissipation", *Proceeding of Twelfth World Conference on Earthquake Engineering.*

Di Sarno, L., G. Manfredi, 2010, "Seismic Retrofitting with Buckling Restrained Braces: Application to an Existing Non-Ductile RC Framed Building", Elsevier Ltd. *Journal of Soil Dynamics and Earthquake Engineering*, Vol. 30, pp. 1279-1297.

Erdik, M., "Earthquake Vulnerability of Buildings and a Mitigation Strategy: Case of Istanbul" *International Workshop on Seismic Performance Assessment and Re-*

habilitation of Existing Buildings.

Fahnestock, L.A., R. Sause, J.M. Ricles, L.W. Lu, 2003, "Ductility Demand on Buckling Restrained Braced Frames Under Earthquake Loading", *Journal of Earthquake Engineering and Engineering Vibration*, Vol. 14, pp. 1671-3664.

Güneyisi E.M., 2011, "Seismic Reliability of Steel Moment Resisting Framed Buildings Retrofitted With Buckling Restrained Braces" Earthquake Engineering And Structural Dynamics, *John Wiley & Sons.*

Hoveidae, N., B. Rafezy, 2012, "Overal Buckling Behavior of All-Steel Buckling Restrained Braces", Elsevier Science Ltd. *Journal of Constructional Steel Research*, Vol. 79, pp. 151-158.

Jones, P., F. Zareian, 2013, "Seismic Response of a 40 Story Buckling-Restrained Braced Frame Designed for the Los Angeles Region, Wiley Online Library" *Journal of the Structural Design of Tall and Special Buildings*, Vol. 22, pp. 291-299.

Lopez-Almansa, F., J.C. Castro-Medina, S. Oller, 2012, "A Numerical Model of the Structural Behavior of Buckling Restrained Braces", Elsevier Science Ltd. *Journal of Engineering Structures*, Vol. 41, pp. 108-117.

Mahmoudi, M., M. Zaree, 2013, "Determination of Response Modification Factors of Buckling Restrained Braced Frames", Elsevier Science Ltd., *Journal of Procedia Engineering*, Vol. 54, pp. 222-231.

Mirtaheri, M., A. Gheidi, A.P. Zandi, P. Alanjari, H.R. Samani, 2011, "Experimental Optimization Studies on Steel Core Lengths in Buckling Restrained Braces", Elsevier Science Ltd. *Journal of Constructional Steel Research*, Vol. 67, pp. 1244-1253.

Piedrafita, D., X. Cahis, E. Simon, J. Comas, 2013, "A New Modular Buckling Restrained Brace for Seismic Resistant Buildings", Elsevier Ltd. *Journal of Engi-*

neering Structures, Vol. 56, pp. 1967-1975.

Prinz, G.S., P.E. Richards, 2012, “Seismic Performance of Buckling Restrained Braced Frames with Eccentric Configurations”, *Journal of Structural Engineering*, Vol. 138, pp. 345-353.

Sabelli, R., S. Mahin, C. Chang, 2003, “Seismic Demands on Steel Braced Frame Buildings with Buckling-Restrained Braces”, Elsevier Science Ltd. *Journal of Engineering Structures*, Vol. 25, pp. 655-666.

SEAOC, Vision 2000, “A Conceptual Framework for Performance Based Seismic Engineering of Buildings”, *Structural Engineers Association of California.*

Takeuchi, T., J.F. Hajjar, R. Matsui, K. Nishimoto, I.D. Aiken, 2012, “Effect of Local Buckling Core Plate Restraint in Buckling Restrained Braces”, Elsevier Science Ltd. *Journal of Engineering Structures*, Vol. 44, pp. 304-311.

TS 498, *Design Loads for Buildings*, Turkish Standards Institution, Ankara, 1987.

Turkish Earthquake Code, Specification of Buildings to be Built in Disaster Areas, The Ministry of Public Works and Settlement, Ankara, 2007.

Watanabe, A., Y. Hitomi, E. Saeki, A. Wada, M. Fujimoto, 1988, “Properties of Brace Encased in Buckling-Restraining Concrete and Steel Tube“, *Proceeding of Ninth World Conference on Earthquake Engineering*, Vol. 4.

Zhao, J., B. Wu, J. Ou, 2014, “A Practical and Unified Global Stability Design Method of Buckling-Restrained Braces: Discussion on Pinned Connections”, Elsevier Ltd. *Journal of Constructional Steel Research*, Vol. 95, pp. 106-115.

Zona, A., L. Ragni, A. Dall’Asta, 2012, “Sensitivity-Based Study of the Influence of Brace Over-Strength Distributions on the Seismic Response of Steel Frames with BRBs”, Elsevier Ltd. *Journal of Engineering Structures*, Vol. 37, pp. 179-192.