PERFORMANCE BASED ANALYSIS OF A STEEL BRACED FRAME BUILDING
WITH BUCKLING RESTRAINED BRACES

A Thesis
presented to
the Faculty of California Polytechnic State University,
San Luis Obispo

In Partial Fulfillment
of the Requirements for the Degree
Master of Science in Architecture with a Specialization in Architectural Engineering

by
Margaux Burkholder
January 2012

© 2012
Margaux Claire Burkholder
ALL RIGHTS RESERVED

COMMITTEE MEMBERSHIP

TITLE: Performance Based Analysis of a Steel Braced Frame
Building With Buckling Restrained Braces

AUTHOR: Margaux Burkholder

DATE SUBMITTED: January 2012

COMMITTEE CHAIR: Jim Guthrie, S.E.

COMMITTEE MEMBER: Kevin Dong, S.E.

COMMITTEE MEMBER: Mary Forte, M.A.

COMMITTEE MEMBER: Robert Pekelnicky, S.E.

ABSTRACT

Performance Based Analysis of a Steel Braced Frame Building With Buckling Restrained Braces Margaux Burkholder

This paper provides an assessment of the seismic performance of a code-designed buckling restrained braced frame building using the performance-based analysis procedures prescribed in ASCE 41-06. The building was designed based on the standards of the ASCE 7-05 for a typical office building located in San Francisco, CA. Nonlinear modeling parameters and acceptance criteria for buckling restrained brace components were developed to match ASCE 41-06 design standards for structural steel components, since buckling restrained braces are not currently included in ASCE 41-06. The building was evaluated using linear static, linear dynamic, nonlinear static and nonlinear dynamic analysis procedures. This study showed that the linear procedures produced more conservative results, with the building performing within the intended Life Safety limit, while the nonlinear procedures predicted that the building performed closer to the Immediate Occupancy limit for the 2/3 maximum considered earthquake hazard. These results apply to the full maximum considered earthquake hazard as well, under which the building performed within the Collapse Prevention limit in the linear analysis results and within the Life Safety limit in the nonlinear analysis results. The results of this paper will provide data for the engineering profession on the behavior of buckling restrained braced frames as well as performance based engineering as it continues to evolve.

ACKNOWLEDGEMENTS

I would like to thank Degenkolb Engineers for their involvement in this project and for providing the topic for research. Specifically, I would like to thank committee members Robert Pekelnicky and Jim Guthrie for their advice and support of this project.

TABLE OF CONTENTS

List of Tables	viii
List of Figures	ix
List of Nomenclature	xi
1.0 Introduction.....	1
2.0 Background.....	3
2.1 Performance-Based Design.....	3
2.2 ASCE 41-06.....	4
2.3 Buckling Restrained Braced Frames (BRBF).....	5
2.3.1 History of BRBFs	5
2.3.2 Applicable Codes and Provisions	7
3.0 Analytical Procedures	8
3.1 Linear Static Analysis	10
3.2 Linear Dynamic Analysis	12
3.3 Nonlinear Static Analysis	13
3.4 Nonlinear Dynamic Analysis	14
4.0 Building Description and Models	17
4.1 Building Description.....	17
4.2 ASCE 7-05 Design.....	19
4.3 ETABS Model	25
4.4 PERFORM 3D Model.....	28
4.5 Partially Restrained Moment Connections	29
4.6 ASCE 41-06 Acceptance Criteria for BRBF	31
4.7 Design Checks	33
5.0 Results.....	36
5.1 ASCE 41-06 BSE-1 Results	37
5.1.1 Brace Results	38
5.1.2 Drift Results.....	41

5.2 ASCE 41-06 BSE-2 Results	45
5.2.1 Brace Results	45
5.2.2 Drift Results	48
5.3 Pushover Curves	51
5.4 Performance Comparison with Different Building Types	53
6.0 Conclusions	55
7.0 References	58
List of Acronyms	60
Appendix	61

LIST OF TABLES

Table 1: Earthquake Hazard Levels	9
Table 2: Site Parameters	18
Table 3: Gravity Framing Members	19
Table 4: BRB Steel Core Design	24
Table 5: Modal Periods and Mass Participation	24
Table 6: Developed BRB Nonlinear Acceptance Criteria	33
Table 7: Modal Periods and Mass Participation	34

LIST OF FIGURES

Figure A: Typical BRBF Element	7
Figure B: BSE-1 Hazard Level Response Spectrum	11
Figure C: BSE-2 Hazard Level Response Spectrum	11
Figure D: Sample Inertia Force Distributions.....	12
Figure E: Scaled BSE-1 Response Spectrums for 7 Ground Motions.....	15
Figure F: Scaled Ground Motions [Acceleration,g]	16
Figure G: 3D View of Building Model.....	18
Figure H: Plan View of Brace Layout	21
Figure I: Brace Elevation in the X-Direction.....	1
Figure J: Brace Elevation in the Y-Direction	1
Figure K: Illustration of BRB Yield Lengths	27
Figure L: Generalized Force-Displacement Curve	28
Figure M: PERFORM 3D Rayleigh Damping.....	29
Figure N: Model of Moment-Rotation Behavior of Shear Tabs.....	30
Figure O: Nippon Test Data and Corresponding Backbone Curve	32
Figure P: Pushover Curve of BRBF Test Frame	35
Figure Q: Brace DCR in X Direction	40
Figure R: Brace DCR in Y Direction.....	40
Figure S: Story Drift in X Direction	42
Figure T: Story Drift in Y Direction	43
Figure U: Maximum Roof Displacements in X Direction.....	43
Figure V: Maximum Roof Displacements in Y Direction.....	44
Figure W: Brace DCR in X Direction.....	47
Figure X: Brace DCR in Y Direction	47
Figure Y: Story Drift in X Direction.....	49
Figure Z: Story Drift in Y Direction	49
Figure AA: Maximum Roof Displacements in X Direction.....	50

Figure BB: Maximum Roof Displacements in Y Direction	51
Figure CC: Pushover Curve in X Direction.....	52
Figure DD: Pushover Curve in Y Direction	53
Figure EE: Scaled Ground Motion 1	1
Figure FF: Scaled Ground Motion 2.....	1
Figure GG: Scaled Ground Motion 3.....	1
Figure HH: Scaled Ground Motion 4.....	1
Figure II: Scaled Ground Motion 5.....	1
Figure JJ: Scaled Ground Motion 6	1
Figure KK: Scaled Ground Motion 7.....	1
Figure LL: Nippon Test Data and Design Information (2006).....	69

LIST OF NOMENCLATURE

C_u – Coefficient for upper limit on calculated period per ASCE 7-05 Table 12.8-1

T_a – Approximate period calculated in accordance to ASCE 7-05 § 12.8.2

1.0 INTRODUCTION

In recent years, the seismic design of buildings has undergone significant changes due to the increased demand for optimization of the structural capabilities of buildings in order to minimize the level of damage, economic loss, and structure repair costs after an earthquake. The current 2009 International Building Code (IBC) provides a force-based method of design that prescribes minimum strength and ductility levels to provide for life safety. However, the seismic demand and capacity of buildings designed with these minimum levels can be uncertain, which leads to uncertain levels of damage. For this reason, engineers are beginning to use performance-based design as an alternative seismic design method. Performance based design is a more comprehensive design method in which the design criteria is based on achieving certain performance goals when the structure is subjected to certain levels of seismic hazard. These goals may be a certain level of stress not to be exceeded, a lateral displacement or deformation, a load, a limit state, or a target damage state (Ghobarah 2001). The use of performance based design has enabled increased reliability of the structure in achieving the desired objectives.

The purpose of this project is to evaluate the performance of a steel braced frame building with Buckling Restrained Braces (BRB) designed according to the current standard, American Society of Civil Engineers (ASCE) Standard 7-05, Minimum Design Loads for Buildings and Other Structures, when analyzed using the four prescribed performance-based analysis procedures in ASCE 41-06, Seismic Rehabilitation of Existing Structures. The project also investigates the differing results from the pushover

analyses completed using ETABS and PERFORM 3D, two commonly used structural analysis programs. Lastly, the results of the comparison between the ASCE 7-05 and the ASCE 41-06 will be compared to the performance of two similar projects using Special Concentrically Braced Frames (SCBF) and Special Moment Frames (SMF) to assess how the lateral system chosen affects building performance. The results of the comparison will provide data for the engineering profession to assess performance-based engineering as it continues to evolve and help engineers choose a building type to use based on the desired performance.

2.0 BACKGROUND

This section will look at the evolution of performance-based design and the development of the American Society of Civil Engineers (ASCE) Standard 41-06 as a set of guidelines for performance-based design. The history of Buckling Restrained Brace Frames and their application in lateral force resisting systems will also be discussed.

2.1 Performance-Based Design

Initial performance-based procedures and provisions were developed under three separate agencies: the Structural Engineers Association of California's (SEAOC) *Vision 2000*, Advanced Technology Council (ATC) 40, and FEMA 273/274 documents.

SEAOC *Vision 2000* was the first of these papers, published in 1995. The goal of this document was to define procedures for producing structures with predictable seismic performance while accommodating multiple performance objectives such as a certain level of drift or acceleration. Along with these design guidelines and performance objectives, performance levels and a series of reference earthquake hazard and design levels were established.

The ATC 40 document published in 1996 provided another definition of performance-based design. This goal of this paper was to describe a method of expressing structural criteria in terms of achieving a performance objective. This goal was accomplished by determining the capacity spectrum of a building and matching it to the demand spectrum of the earthquake. However, the document was limited to concrete buildings and not all components of the procedure were well established. At this stage, methods of implementation of performance-based design were still undefined.

Research on methods of implementation came in the form of FEMA 273/274, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, published in 1997. FEMA 273/274 built on SEAOC's *Vision 2000* by providing analysis procedures to assess the probable seismic performance of a building. The document also defined performance levels for nonstructural elements and systems and proposed drift limitations for various lateral force resisting systems, such as steel and concrete moment and braced frames and timber and masonry structures at different performance levels (Ghobarah 2001).

2.2 ASCE 41-06

As knowledge of performance-based design expanded, so did the need to set one standard of design that industry could follow. In 2000, FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Structures*, was published based on FEMA 273/274 by a team of specialists in earthquake engineering to incorporate the most advanced analytical techniques considered for practical use. These techniques and acceptance criteria were tested by a program of case studies in 1998 to ensure the accuracy of the document. The intention was to create a basis for a national consensus standard that could be adopted by design professionals to create buildings that were more resistant to earthquakes.

In 2007, the ASCE produced ASCE Standard 41-06, *Seismic Rehabilitation of Existing Buildings*, with the goal of providing a set of rules, regulations, and procedures dealing with assessment and strengthening of existing structures, with each set of rules

and regulations organized by building type. The provisions of this document are based on FEMA 356 with limited material included from FEMA 274.

ASCE 41-06 superseded FEMA 356 as the national standard for performance-based design procedures to be used in the seismic assessment and rehabilitation of structures.

2.3 Buckling Restrained Braced Frames (BRBF)

The analysis procedures prescribed in ASCE 41-06 will be used to analyze the performance of the Buckling Restrained Braced Frame building designed for this project. BRBFs are a special class of concentrically braced frame that are currently used as primary lateral force resisting elements in new construction and seismic retrofit projects. The main component of the BRBF is the Buckling Restrained Brace (BRB), which consists of a buckling restraining mechanism that is separate from the load resisting steel core, effectively reducing the un-braced length of the compression member to zero and eliminating the buckling failure mode. The core is then able to yield in both compression and tension as well as sustain compressive strains relatively uniform to the tensile strains.

2.3.1 History of BRBFs

The concept of the BRBs was first explored in the 1980s at the Tokyo Institute of Technology in Japan. The first BRBF/ Moment Resisting Frame was built in Japan in 1988 with the use of the Unbonded BraceTM manufactured by Nippon Steel Corporation. The first testing and application of BRBs in the United States was in 1999 for the Plant and Environmental Sciences Building at the University of California, Davis. To date, there are an estimated 150 structures in the United States utilizing BRBFs with 20,000

BRBs (Lopez 2008). BRBF systems have been approved for use by the Division of the State Architect (DSA), the Office of Statewide Health Planning and Development (OSHPD), and the University of California. Currently there are three proprietary manufacturers of BRBs: Nippon Steel Corporation, Star Seismic, and CoreBrace.

A typical BRB consists of a yielding steel core that provides axial resistance confined by a concrete-filled steel casing that provides flexural and buckling resistance. BRBs are able to achieve stable, balanced hysteresis loops and considerable ductility due to the gap between the steel core and surrounding concrete, which decouples the stress resisting and buckling resisting aspects of the compression strength (Hussain, et al. 2005). Figure A on page 7 shows the typical detail of a BRB member. This system provides sufficient strength and stiffness to prevent global and local buckling as well as plastic hinges associated with buckling. Prevention of buckling results in minimal degradation of the strength and stiffness of the BRB as well as equal tension and compression capacities (Asgarian and Amirhesari 2008). The equal capacities result in significantly improved performance compared with conventional concentric braced frames (CBF) which have unequal compression and tension capacities. With conventional CBFs, the compression brace will most likely buckle, leaving the tension brace to resist the majority of the demand during a major seismic event.

Figure A: Typical BRBF Element

Source: Hussain et al 2005

As BRBFs gained increased popularity, so did the need for a better understanding of the member and system behavior. Extensive research, analytical studies (e.g., Fahnestock, *et al.* 2003; Sabelli, Mahin, and Chang 2003), and experimental studies (e.g., Asgarian and Amirhesari 2008) have been performed in order to help establish design guidelines and appropriate testing procedures for BRBFs.

2.3.2 Applicable Codes and Provisions

The design of BRBF systems is currently governed by the 2003 National Earthquake Hazards Reduction Program (NEHRP), Recommended Provisions for New Buildings and Other Structures (FEMA 450), and the 2005 AISC Seismic Provisions for Structural Steel Buildings. These documents have been incorporated into the current 2010 California Building Code (CBC), which is based on the 2009 IBC. The 2009 IBC references the Minimum Design Loads for Buildings and Other Structures (ASCE 7-05) and the 2005 AISC Seismic Provisions. Currently there are two BRBF systems defined by these provisions. The first system utilizes a moment-resisting beam-to-column connection. The second system utilizes pinned beam-to-column connections. For this

project, a BRBF system with moment-resisting beam-to-column connections will be analyzed since this the most common type of connection used in existing BRB structures.

3.0 ANALYTICAL PROCEDURES

As stated in section 1.1 of this paper, the purpose of this project is to evaluate the performance of a Buckling Restrained Braced Frame (BRBF) structure using the four prescribed seismic analysis procedures stated in the ASCE 41-06. This evaluation is performed to determine whether the building, in its existing condition, has the desired seismic performance capabilities. Prior to using these procedures, a seismic Rehabilitation Objective, Building Performance Level, and Earthquake Hazard Level must be selected.

The Rehabilitation Objective is selected as a basis for design based on the cost and feasibility of a rehabilitation project and the benefits to be obtained in terms of improved safety. The three specific Rehabilitation Objectives that may be used are described in ASCE 41-06 §1.4 as the Basic Safety Objective, Limited Rehabilitation Objectives, and Enhanced Safety Objectives. For this project the Basic Safety Objective (BSO) was selected. The BSO is intended to approximate the earthquake risk to life safety traditionally considered acceptable in the United States. Buildings meeting the BSO are expected to experience little damage from moderate and frequent earthquakes and significant damage from more severe and infrequent earthquakes (ASCE 41-06, 8).

Once a Rehabilitation Objective is selected, a Building Performance Level is chosen. The performance levels are categorized in the ASCE 41-06 based on the extent of damage that would be sustained by the building. Tables C1-1 through C1-7 list these

performance levels as Collapse Prevention, Life Safety, Immediate Occupancy, and Operational Performance and describe the amount of structural and nonstructural damage a building will experience at each level. The desired Performance Level of a building is chosen by the owner based on the importance and operational need of the building.

Lastly, a Probabilistic Earthquake Hazard Level is selected. The earthquakes corresponding with the BSO chosen as defined by ASCE 41-06 are Basic Safety Earthquake 1 (BSE-1) and BSE-2. These earthquakes and their corresponding mean return periods are shown below in Table 1. The mean return period relates to the average number of years between two earthquakes of the same level of severity.

Earthquake Having Probability of Exceedance	Mean Return Period [years]
BSE 1 - 10%/50 year	500
BSE 2 - 2%/50 year	2500

Table 1: Earthquake Hazard Levels

Buildings designed to meet the BSO are expected to achieve a Life Safety Building Performance Level for the BSE-1 Earthquake Hazard Level and a Collapse Prevention Performance Building Performance Level for the BSE-2 Earthquake Hazard Level.

Before any of the analysis procedures can be used, the building must comply with the analysis requirements listed in ASCE 41-06 §2.6 and 3.2. Due to the lack of irregularities in the building used for this project and the low level of expected nonlinearity, all the requirements were met allowing the two linear and two nonlinear procedures to be used.

3.1 Linear Static Analysis

The Linear Static Procedure (LSP) uses a linearly elastic, static analysis to design seismic forces that are evenly distributed over the height of the building that develop corresponding internal forces and system displacements. The LSP model represents the building with a linear-elastic stiffness that corresponds to the building's stiffness before yield occurs. Although the procedure is described as linear, geometric nonlinearity such as P-delta effects are considered.

According to ASCE 41-06, 62, "The magnitude of the pseudo-lateral force applied to the structure has been selected with the intention that, when applied to the linearly elastic model of the building, it will result in design displacement amplitudes approximating maximum displacements expected during the design earthquake". This pseudo-lateral force is typically much larger than the ASCE 7-05 design lateral force and creates internal forces that exceed the capacity of the members. These high Demand to Capacity Ratios (DCR) are evaluated through the acceptance criteria of ASCE 41-06 §3.4.2, which includes demand modification or m -factors that reduce the demand on members of the structure. The m -factors also account for the anticipated nonlinear response of the structure and are an indirect measure of the nonlinear deformation capacity of the component (ASCE 41-06, 72). These m -factors are listed in Chapter 5 of the ASCE 41-06 for steel structures.

The pseudo-lateral force is calculated using the equation in ASCE 41-06 §3.3.1.3.1, which incorporates the seismic building weight, fundamental building period, and response spectrum acceleration associated with the structure. The response spectrum

acceleration values used are taken from the 5% damped general response spectrum provided by ASCE 41-06 § 1.6.1.5 for BSE-1 and BSE-2. The response spectrum for both Hazard Level Earthquakes can be seen in Figures B and C below.

Figure B: BSE-1 Hazard Level Response Spectrum

Figure C: BSE-2 Hazard Level Response Spectrum

3.2 Linear Dynamic Analysis

The Linear Dynamic Procedure (LDP) uses a linearly elastic, dynamic analysis to design seismic forces that are evenly distributed over the height of the building that develops corresponding internal forces and system displacements. The LDP model is the same as that used for the LSP.

The LDP uses a modal spectral analysis with the same unmodified response spectra used in the LSP to account for anticipated nonlinear response. As with the LSP, the LDP produces displacements that approximate maximum displacements expected during the design earthquake and internal forces that exceed the yield capacity of the members in the structure. The same m-factors used for the LSP are applied to accommodate the high demands on the structure. However, the demand due to the LDP is smaller than the demand due to the LSP because an increased number of modes are considered in order to capture the required 90% of the participating mass of the building. Figure D below shows how an increased modal analysis reduces the forces applied to the structure.

Figure D: Sample Inertia Force Distributions
Source: FEMA 274, 1997

3.3 Nonlinear Static Analysis

For the Nonlinear Static Procedure (NSP) “a model of the structure incorporating the nonlinear load-deformation characteristics of individual components of the building is subjected to monotonically increasing lateral loads until a target displacement is exceeded. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake” (ASCE 41-06, 67). Unlike the internal forces calculated in the linear procedures, the internal forces of the NSP are reasonable approximations of those expected during the design earthquake. This target displacement is calculated using the procedures described in ASCE 41-06 § 3.3.3.3.2. It is required that the analysis be carried out to at least 150% of the target displacement to encourage investigation of the model under extreme load conditions that exceed the design values. This is due to the fact that the target displacement represents a mean displacement value and that there is considerable scatter about the mean (ASCE 41-06, 68).

ASCE 41-06 states that the lateral forces applied to the building should be proportional to the shape of the fundamental mode in the direction under consideration and that this single load pattern is sufficient. Earlier documents such as FEMA 356 recommended the use of more than one lateral load pattern as a way to bind the range of design actions that may occur during actual dynamic response. However, recent research published in FEMA 440 has shown that inclusion of multiple load patterns do little to improve the accuracy of NSP when higher modes are not significant. Because the building for this project is not significantly affected by higher modes, a triangular load

pattern based on the shape of the dominant mode would have been sufficient to perform the analysis. However, a uniform load pattern is also included for this project.

The acceptance criteria used for the NSP are listed in Chapter 5 of ASCE 41-06 and are based on the deformation capacities of separate members of the structure in terms of rotation and axial deformation.

3.4 Nonlinear Dynamic Analysis

For the Nonlinear Dynamic Procedure (NDP), a time-history analysis is performed in which a model that incorporates the nonlinear load-deformation capacities of individual members in the structure is subjected to earthquake ground motions to obtain forces and displacements. The basis, modeling approaches, and acceptance criteria of the NDP are similar to those for the NSP. The main exception is that the design displacement is not established using a target displacement but instead is determined directly through dynamic analysis using the ground motion time histories. Because calculated response can be highly sensitive to the characteristics of one ground motion, it is required that the analysis be carried out with a minimum of three ground motions. If fewer than seven ground motions are used, the maximum demand in any member may be used. If seven ground motions or more are used, an average value of the maximum demands is allowed to be used. For this project seven ground motions were used. Plots of the acceleration time histories for the seven ground motions are shown on the next page in Figure F. The plots have been truncated to 45 seconds for clarity; however, full versions of the plots can be seen in the Appendix on page 62. The ground motions chosen have duration and spectral content equivalent to expected ground motions for the San

Francisco Financial District. All seven ground motions were scaled to the ASCE 41-06 response spectrum to match the spectral acceleration at a period of 1.0 seconds for both the BSE-1 and BSE-2 Hazard Levels. The time histories were recorded with 0.01 second time intervals. The response spectra are shown in Figure E below.

The lateral forces applied to the structure are determined using the same criteria as the NSP. Because the model accounts directly for effects of material inelastic response, the resulting internal forces are reasonable approximations of those expected during the design earthquake (ASCE 41-06, 72).

Figure E: Scaled BSE-1 Response Spectrums for 7 Ground Motions

Figure F: Scaled Ground Motions [Acceleration,g]

4.0 BUILDING DESCRIPTION AND MODELS

This section illustrates the procedure followed to design the Buckling Restrained Braced Frame (BRBF) building for this study. The following documents were used to calculate the loading demands and perform the design checks:

- ASCE 7-05, Minimum Design Loads for Buildings and Other Structures
- American Institute of Steel Construction (AISC) Steel Construction and Seismic Design manuals
- Chapter 8, Section 8.6 of FEMA 450.

The resulting design was then analyzed using the four previously described analytical procedures included in the ASCE 41-06. Two computer models of the building were created using the structural analysis programs ETABS v.9.2.0 and PERFORM 3D v.4. ETABS was used to perform the linear static, linear dynamic, and nonlinear static analyses, while PERFORM 3D was used to complete a comparative nonlinear static analysis as well as the nonlinear dynamic analyses.

4.1 Building Description

The building is a six-story structure with a rectangular floor plan that is 180'-0" in the east-west direction and 150'-0" in the north-south direction. Bays span 30'-0" in each direction. The total height of the building is 83'-2" with two subgrade levels totaling a depth of 27'-6". A 3D perspective of the building is shown in Figure G on the next page. The floors and roof are comprised of 3" steel decking with 3 1/4" lightweight concrete fill. The weight due to the mechanical loading on the roof was incorporated with a reducible live load placed on the center bays of the roof. The first floor has a glass curtain wall and

all the floors above have a precast concrete cladding. All other loads were calculated using the load combinations described in Chapters 2 and 12 of the ASCE 7-05. The total seismic weight of the structure is 17,142 kips. The subgrade structure is comprised of 18" concrete walls along the perimeter of the building at the lower basement level and 14" walls at the upper basement level. The building will be used as an office and is in Occupancy Category II per ASCE 7-05 Table 1-1 with an importance factor of 1.0. The building site is located west of the Financial District of San Francisco, CA. The site seismicity information is listed below in Table 2. The site is classified as soil type D due to lack of soil information provided for this project. For this reason the effects of soil-structure interaction were not evaluated for this building.

Figure G: 3D View of Building Model

Period	S_a (g)	S_{DS} (g)	S_{D1} (g)
$T = 0.2s$	1.5	1.0	-
$T = 0.1s$	0.667	-	0.65

Table 2: Site Parameters

4.2 ASCE 7-05 Design

The building was designed to meet the standards and parameters of the ASCE 7-05. The gravity framing beams and girders were designed as composite members per the AISC Steel Construction Manual. Table 3 provides a summary of the gravity framing members. The lateral force resisting system is comprised of Buckling Restrained Braces

Location		Beam	Girder
Roof	Interior Bays	W16x26	W18x40
	Exterior Bays	W14x22	W18x65
Floors 2-6	All Bays	W14x22	W18x50
Floor 1	All Bays	W14x30	W21x45

Table 3: Gravity Framing Members

(BRBs) as well as all beams and columns the braces frame in to, which were designed to meet the requirements for a BRBF per § 8.6 of FEMA 450 and the AISC Seismic Design Manual. The braces were first designed for adequate strength using an equivalent-lateral-force method. For such an elastic analysis to be valid, the behavior of the brace elements used corresponds to tested brace behavior. Nippon Steel was chosen as the brace manufacturer and provided brace length and stiffness estimates, adjusted brace strength factors, and resulting hysteretic loops from brace testing. The data provided by Nippon based on testing completed in 2006 is located in the Appendix on page 69. The area of the steel core of each brace was designed to have a Demand Capacity Ratio (DCR) between 0.8 and 1.00. The adjoining frame elements were designed for the adjusted brace strengths corresponding to two times the design story drift deformations. The adjusted brace strengths are significantly higher than the brace design forces because of the compression overstrength and strain hardening of the brace at large deformations and

under repeated cyclic inelastic loading. The adjusted brace strength factors ω and β provided by Nippon represent the maximum tension/compression ratios based on tested brace results. These factors were used to determine the adjusted brace strengths.

A plan of the building and the brace layout is shown in Figure H on page 21 and the lateral framing design is shown in elevations in Figures I and J on pages 22 and 23. The elevation in the X-direction depicts braces on grid lines B and F while the elevation in the Y-direction depicts braces on grid lines 2 and 5. Table 4 on page 24 summarizes the steel core brace design per floor.

Figure H: Plan View of Brace Layout

Figure I: Brace Elevation in the X-Direction

Level	A _{SC}	ϕP_n	ω	β	C _{MAX}	T _{MAX}	DCR-X	DCR-Y
Roof	3.00 in ²	113 k	1.22	1.02	157 k	154 k	0.95	0.72
6	5.50 in ²	208 k	1.20	1.02	282 k	278 k	0.88	0.73
5	7.25 in ²	274 k	1.20	1.01	369 k	364 k	0.85	0.74
4	8.750 in ²	331 k	1.14	1.04	437 k	420 k	0.82	0.73
3	10.00 in ²	378 k	1.14	1.04	499 k	480 k	0.82	0.72
2	12.00 in ²	454 k	1.12	1.04	585 k	562 k	0.74	0.75

Table 4: BRB Steel Core Design

The fundamental period of the building provided by ETABS of 1.04 seconds in the X-direction and 1.00 seconds in the Y-direction was less than the maximum allowable period per ASCE 7-05 § 12.8.2 of 1.16 seconds. The initial lateral forces used to design the BRBF as well as determine story drift were calculated using the approximate building period, T_a , in ASCE 7-05 § 12.8.2.1; however, the actual building periods stated above were used for the final design. The resulting ASCE 7-05 base shear was 1,730 kips. The building experiences translation in the X-direction for the first mode, torsion for the second mode, and translation in the Y-direction for the third mode. Although the torsion mode shape occurred second, the dominant modal frequencies and mass participation were in the X and Y-direction and were the periods used for the analysis of the building. Table 5 below summarizes the first three mode shapes and their corresponding mass participation ratios. A total of 20 building modes were included in order to achieve the 99.5% mass participation required for the linear dynamic procedure.

Mode	Period	% Mass Participation
1	1.04	72.16
2	1.01	0
3	1.00	72.58

Table 5: Modal Periods and Mass Participation

4.3 ETABS Model

The structural analysis program ETABS v.9.2.0 was used to model the structure for the linear static, linear dynamic, and nonlinear static analysis. Following is a description of the computer model as well as the modeling assumptions made:

- Dead and live loads have been applied as element loads based on what is tributary to that member.
- A yield strength of 38 ksi was used for all braces with an ultimate strength of 42 ksi. All other steel was assumed to be 50 ksi with assumed yield strength of 55 ksi.
- Self-weight is not calculated by the computer program.
- Seismic forces were applied at the center of mass at each diaphragm as point loads with an additional moment applied to account for accidental torsion (5% eccentricity).
- Five percent modal damping was applied to the structure for parity with other models, although damping of 2% would be more realistic for a BRBF structure since damping of the structure is accounted for in the hysteretic behavior of the braces.
- Braces are modeled as pinned end conditions.
- Frame beam-to-column connections were modeled as Welded Unreinforced Flange (WUF) connections along lines B, F, 2, and 5 where the braces are located. All other connections were modeled as partially restrained moment connections as described in section 4.5.

- Braced frame beams are segmented and begin at the column and end at the apex brace connection.
- All beam and column end zones were assumed to be fully rigid to provide parity with the PERFORM 3D model. Whereas ETABS provides a rigid end zone factor in which the user can input the ratio of the rigid versus elastic portion of the end zone, PERFORM 3D uses a multiple of the beam stiffness over the entire length of the end zone. For the ETABS model, a rigid end zone factor of 1.0 was used. For the PERFORM 3D model, the default end zone which uses a multiple of 10 times the element stiffness was used.
- The default rigid end zone length of one half the element depths was used for all non-frame beam and column end zones. A custom end zone length equivalent to the length of the gusset plate was used in all frame beam and column end zones.
- Floor diaphragms are modeled as rigid.
- As shown in Figure K, the actual length of the steel core is smaller than the work-point-to-work-point length of the brace. As a result, the actual stiffness of the brace is greater than that computed using only the steel core area. For this project, the effective stiffness of the BRB is defined as 1.66 times the stiffness computed using only the steel core.

Figure K: Illustration of BRB Yield Lengths

Source: Lopez and Sabelli 2004

- Frame columns are modeled as pinned at their bases.
- Columns in plane of the basement wall were modeled as continuous to the ground and encased by concrete walls.
- No mass is included in the basement floors or shear walls.
- Element and connection force-displacement curves were calculated using ASCE 41-06 Table 5-6 and Table 5-7 in order to perform the nonlinear static procedure. The generalized force-displacement relation for steel components used for all non BRB primary components is shown in Figure L below. All beams and columns surrounding brace components were assumed to be primary components due to the significant frame action behavior caused by the fixed beam-to-column connections. Adaptations of the ASCE 41 procedure were used to develop the force-displacement curves for the BRBs and are explained in Section 4.6.

Figure L: Generalized Force-Displacement Curve

4.4 PERFORM 3D Model

The nonlinear dynamic procedure as well as a second nonlinear static procedure were completed using PERFORM 3D v. 4, a nonlinear structural analysis program distributed by CSI. The brace, beam, and column hinges were defined with the same location and properties as in the ETABS model. The same modeling assumptions made for the ETABS model applied to the PERFORM 3D model with the following changes. The seismic mass was not calculated by the program based on distributed floor load as in ETABS. Instead the seismic mass was defined as an applied nodal load placed at the center of rigidity of each level. The nodal loads used were equivalent to the seismic mass output from the ETABS model.

In addition to 5% modal damping, 0.2% Raleigh damping was applied. Modal damping only applies to the displaced shapes that correspond with the elastic mode shapes. Applying a small amount of Raleigh damping ensures that higher mode displacements are damped. Since this small amount of damping is placed at the highest calculated period of the structure, it has little effect on the behavior of the lower modes.

A graph displaying the Rayleigh damping for the building can be seen in Figure M below.

Figure M: PERFORM 3D Rayleigh Damping
Source: PERFORM 3D

4.5 Partially Restrained Moment Connections

All frame beams and drag beams were modeled as moment resisting connections while all gravity load connections were modeled as shear tab connections. Shear tab connections are idealized as pinned connections that have shear and axial strength but no rotational stiffness or moment capacity. However research done in *Design of Shear Tab Connections for Gravity and Seismic Loads* by Abolhassan Astaneh-Asl (2005) shows that deep shear tab connections with more than 5 bolts can have considerable initial rotational stiffness and can develop significant bending moment in the connection especially where the floor slab is present. Figure N below shows the simplified model of the moment-rotation behavior of typical shear tab connections used for this project.

Figure N: Model of Moment-Rotation Behavior of Shear Tabs
Source: Astaneh-Asl 2005

Astaneh-Asl's model shows that the shear tab connection has a higher positive bending moment capacity than negative moment capacity which corresponds to the bending moment reached before the concrete floor slab is crushed. This means the connection has a different rotational stiffness for positive and negative bending. The nonlinear hinge properties in ETABS allow for different positive and negative stiffness properties while PERFORM 3D requires a single value for rotational stiffness. The rotational stiffness used for the PERFORM 3D hinges was taken as the average of the positive and negative values at each shear tab connection. The positive and negative moment capacities and corresponding maximum rotations of each hinge were calculated using the method described in Astaneh-Asl's paper.

4.6 ASCE 41-06 Acceptance Criteria for BRBF

Adaptations were made to the linear and nonlinear acceptance criteria provided in the ASCE 41-06 where applicable to better suit the behavior of the structure. For the linear analyses, the demand modification or m -factors for braces in tension were used to compare the demand capacity ratios for both the tension and compression braces due to the relatively equal capacities of BRBs in tension and compression. The ASCE 41 m -factors for braces in tension are conservative for BRBs since they are based on components such as concentrically braced frames with a lower ductility capacity than BRBs; however, for modification factors that accurately quantify the behavior of BRBs to be produced, a series of analytical studies and experimental programs must be conducted. Such studies are not included in the scope of this thesis. For this thesis, the use of the ASCE 41 m -factors for braces in tension to assess the performance of the BRBs was deemed acceptable.

For the nonlinear analysis, Nippon test results listed in the Appendix and hysteretic loops from 5 different specimens with varying steel core areas were used to develop full backbone curves that capture the behavior of the BRBs. Figure NO below is an example force-displacement loop for a tested BRB provided by Nippon. It is important to note that although the actual forces and displacements are dependent on the size of the steel core, the behavior of the BRBs is consistent for all test specimens regardless of the brace capacity. Each of the tested BRBs obtained a ductility capacity of 20 times the brace axial yield deformation. This ductility capacity does not represent the maximum capacity of the braces since the tests did not push the BRBs to failure. However, the

obtained ductility capacity was used as the maximum brace capacity, providing a conservative assumption of the brace capacity.

Figure O: Nippon Test Data and Corresponding Backbone Curve
Source: Nippon Steel Engineering Unbonded Brace Design Information Packet (2006)

Per § 3.4.3.2.1 of ASCE 41-06, use of secondary component acceptance criteria for components modeled explicitly with full backbone curves is allowed. Use of the corresponding nonlinear acceptance criteria for braces in tension would limit the ductility demand of the BRBs to 0.25, 11, and 13 times the yield deformation of the brace for Immediate Occupancy, Life Safe, and Collapse Prevention respectively. Based on the limited test data provided by Nippon as well as previous experimental studies summarized by Fahnestock, *et al.* (2003), it can be concluded that BRBs are capable of

significant maximum ductility capacities of up to 20 times the yield deformation. From this research, it may be concluded that use of the acceptance criteria listed above would be too conservative for BRBs. In order to more accurately assess the performance of the BRBS, acceptance criteria for axial force and brace elongation were derived from the experimental research and recommendations from Nippon. The resulting nonlinear acceptance criteria used for BRBs for this project are listed below in Table 6 alongside the existing nonlinear acceptance criteria for braces in tension as stated in ASCE 41-06.

	Acceptance Criteria	
	ASCE 41-06: Braces in Tension	Developed Criteria For BRB
Immediate Occupancy	$0.25\Delta y$	$5\Delta y$
Life Safety	$11\Delta y$	$15\Delta y$
Collapse Prevention	$13\Delta y$	$20\Delta y$

Table 6: Developed BRB Nonlinear Acceptance Criteria

4.7 Design Checks

Several design checks were completed in order to validate the accuracy of the computer models. For the first check, the Rayleigh -Ritz method was used to verify the fundamental period of the building. The building period was calculated using Eq. 1 from ASCE 7-05 §15.4.4,

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^n w_i \delta_i^2}{g \sum_{i=1}^n f_i \delta_i}}, \quad \text{Eq. 1}$$

where w_i is the portion of the seismic weight at floor i ,
 δ_i is the displacement at floor i due to lateral force, f_i ,
 f_i is the lateral load applied at level i
 n is the total number of stories in the vertical seismic framing; and
 g is the acceleration due to gravity.

The results of this equation are listed below in Table 7 in comparison to the fundamental periods of both structural models. The ETABS values in both directions match the calculated values, and the PERFORM 3D values are within 5%, which is deemed to be acceptable for this thesis.

X-Direction		Y-Direction	
Method	Period (sec.)	Method	Period (sec.)
ETABS	1.04	ETABS	1.00
PERFORM	1.00	PERFORM	0.94
RR Method	1.04	RR Method	1.00

Table 7: Modal Periods and Mass Participation

The second check performed was to ensure the nonlinear hinges in the seismic force resisting system were modeled correctly. To do this, the test results of a full-scale BRBF subjected to cyclic loading were obtained from an experimental study in *Lessons Learned from Large-Scale Tests of Unbonded Brace Frame Subassemblages* by Lopez, et al. (2002). A replicate of the test frame was modeled in ETABS and PERFORM 3D with nonlinear hinges in the brace and beam elements. A pushover analysis was performed in both programs. Figure P below shows the results of the pushover analyses in comparison with the tabulated test results. It may be concluded from these results that the nonlinear hinge properties for the braced frame beams and braces used in ETABS and PERFORM 3D accurately represent the nonlinear behavior of a BRBF.

Figure P: Pushover Curve of BRBF Test Frame

5.0 RESULTS

This section provides a summary of the results from the four previously described analytical procedures included in the ASCE 41-06. As stated before in this paper, two computer models of the building were created using the structural analysis programs ETABS v.9.2.0 and PERFORM 3D v.4. ETABS was used to perform the linear static, linear dynamic, and nonlinear static analyses, while PERFORM 3D was used to complete a comparative nonlinear static analysis as well as the nonlinear dynamic analyses.

Data was collected from the individual components of the structure and compared to acceptance criteria to assess their performance, from which the overall performance of the building may be concluded. For the braces, all of the data was recorded at the brace hinge, or the midspan of the member. For the frame beams and columns, the data was recorded in the portion of the member just outside of the gusset plate connection where hinges would be expected to form. For each procedure, the maximum axial brace force and axial deformation were recorded along with the maximum demand and hinge rotations in the beams and columns. The maximum interstory drift at each level was also recorded for each procedure.

For the linear procedures, the performance of each member was assessed by comparing the component demand-capacity ratio (DCR) to m -factors associated with a specific performance level per ASCE 41-06 Table 5-5.

For the nonlinear procedures, the performance of each member was assessed by comparing the inelastic displacement of the component to acceptance criteria for a given performance level. The acceptance criteria for primary members per ASCE 41-06 Table

5-6 were used for the frame beams and columns. The derived acceptance criteria based on experimental data and research as described in section 4.6 were used for the braces.

5.1 ASCE 41-06 BSE-1 Results

As stated in section 3.0 of this paper, the Basic Safety Objective (BSO) was selected as the design basis of the structure. Buildings meeting the BSO are expected to achieve a Life Safety Building Performance Level for the BSE-1 Earthquake Hazard Level. The following graphs compare the demand from the analysis procedures with the maximum allowable demand associated with the Life Safety Performance Level.

Due to the equal capacity of BRBs in tension and compression, the performance of the braces in tension and compression were very similar with a slightly higher demand occurring in the compression braces. For this reason the graphs below depict the averaged results for the compression braces only and do not include the tension brace results. The performance of the frame beams and columns are not included in the tabulated results because both experienced only minor flexural yielding in all procedures. This is because the beams and columns were designed for the axial capacity of the braces including overstrength and for the induced flexural demand when the braces first begin to yield.

For this project, the inelastic axial displacements from the analyses have been divided by the brace yield displacement to produce a unitless DCR similar to the linear procedures. The acceptance criteria have also been divided by the yield displacement to create an inelastic equivalent to the m -factors. This conversion was done to provide consistency in the assessment of the linear and nonlinear results.

5.1.1 Brace Results

The figures on the following pages show the results for the compression braces in the BRBFs at each floor for all analyses. The demands at each level include an average of the demands from all the braces in compression at each floor and are tabulated for each direction. Because all of the compression braces per floor experienced similar demands, an average value is a good representation of the brace behavior. The demand in the braces for the linear procedures is equal to the axial brace force and the demand in the braces for the nonlinear procedures is equal to the axial brace deformation. The results in Figure Q and R depict the demands in the braces as a percentage of the Life Safety Performance Level capacity.

As illustrated by Figure Q, the axial force and deformation demands in the braces in the X-direction are within the Life Safety performance level criteria. The maximum demand versus the allowable Life Safety capacity is approximately 78% for the linear procedures and 61% for the nonlinear procedures.

Likewise, Figure R shows that the axial force and deformation demands in the braces in the Y-direction are also well within the Life Safety performance level criteria with maximum demand-capacity ratios of approximately 83% for the linear procedures and 63% for the nonlinear procedures. The demand-capacity ratios from the ASCE 41-06 analyses are smaller in the X-direction than in the Y-direction because the yield strength of the building is greater in the X-direction.

Overall, the linear procedures produced the highest brace demands at each level with uniform demands along the height of the building. The nonlinear static procedures

had more variation depending on the level but resulted in average brace demands below 50% of their Life Safety capacity. The tendency for force to concentrate in the lower floors is noticeable, although no pronounced soft story behavior was observed. The nonlinear dynamic procedure resulted in the lowest and most uniform demands at each level with demands equal to 14% of the brace Life Safety capacities, which is equal to 50% of Immediate Occupancy capacity.

In order to achieve a more uniform distribution of deformation and ductility, the BRB properties could be “fine tuned” more in the design process. However, this can be an iterative process since the demands on the structure depend on the structural properties of the building and the ground characteristics used. While this process is recommended, the basic observation from the analysis summarized here is that the equivalent lateral force design approach produces a reasonable design with better than expected overall BRBF performance that exceeds the expected Life Safety performance level for a BSE-1 Earthquake Hazard Level.

Figure Q: Brace DCR in X Direction

Figure R: Brace DCR in Y Direction

5.1.2 Drift Results

The lateral displacements of each story measured at the frame beam were recorded. The maximum roof displacement for each analysis was also recorded. As illustrated by Figures S and T on the following pages, the linear procedure drift results are similar at each floor for both the X and Y-direction with fairly uniform interstory drift ratios over the height of the building. The nonlinear static procedure drift results vary widely for the uniform loading profile but are more uniform for the triangular loading profile. The BRBs in all stories yield almost simultaneously under the triangular loading profile while the uniform loading profile causes the BRBs in the lower stories to yield first, leading to much larger drifts in the first and second story. The nonlinear static procedures performed using PERFORM 3D indicated similar interstory drifts to those from the ETABS analysis for both the uniform loading pattern and the triangular loading pattern. If the drifts from each of the nonlinear static procedures are enveloped, the resulting interstory drift at each floor is greater than the nonlinear dynamic interstory drift.

Overall, the linear static and dynamic procedures resulted in the highest drifts, followed by the nonlinear static procedure, then the nonlinear dynamic procedure. The maximum interstory drift ratio of 1.4% resulting from the NSP occurs at the third floor in the Y-direction while an average 0.6% drift resulted from the NDP. Based on Table C1-3 of ASCE 41-06, typical story drifts for a building with steel braced frames are 1.5% transient and 0.5% permanent for a Life Safety performance level. Although ASCE 41-06 does not intend for the story drifts to be used as acceptance criteria, they can be a

better indicator of damage in the nonlinear range of building response than forces. In this range, relatively small increases in force produce large changes in displacement. The story drifts from the BSE-1 Hazard Level analyses represent story drifts that would be typical for a Life Safety performance level. These results are consistent with the results of the brace demand-capacity ratios and are a good indicator of the damage experienced in the nonlinear range.

Figure S: Story Drift in X Direction

Figure T: Story Drift in Y Direction

Figure U: Maximum Roof Displacements in X Direction

Figure V: Maximum Roof Displacements in Y Direction

As seen in Figures U and V above, the maximum roof displacement results show that the code estimated maximum inelastic displacement is less than the displacement from each analytical procedure except the nonlinear dynamic analysis. One explanation for this may be that, for short period structures, ASCE 41 uses the equal energy method of determining the maximum inelastic displacement while the code uses the equal displacement method. An equal energy approach will result in a higher maximum displacement than an equal displacement approach for the same structure, which is consistent with the results. Another reason may be that the C_d factors in the code, which are intended to account for inelastic deformations, are too low and as a result provide non-conservative estimates of inelastic lateral displacements, a conclusion that is supported by previous studies (Fahnestock, *et al* 2003). The maximum roof displacement

results also confirm that the nonlinear procedures produce lower lateral displacements than the linear procedures and give a better approximation of the building performance.

5.2 ASCE 41-06 BSE-2 Results

Buildings meeting the Basic Safety Objective are expected to achieve a Collapse Prevention Building Performance Level for the BSE-2 Earthquake Hazard Level. The following graphs compare the demand from the analysis procedures with the maximum allowable demand associated with the Collapse Prevention Performance Level. The demands are shown as a percentage of the Life Safety capacity to facilitate comparison of the BSE-1 and BSE-2 Hazard Level results. The maximum allowable demand associated with the Collapse Prevention Performance Level is equivalent to 133% of Life Safety capacity.

5.2.1 Brace Results

The figures on the following pages show the averaged results for the compression braces in the BRBFs at each floor for all analyses. The demand in the braces for the linear procedures is equal to the axial brace force and the demand in the braces for the nonlinear procedures is equal to the axial brace deformation.

As illustrated by Figure W below, the axial force and deformation demands in the braces in the X-direction remain within the Life Safety Performance Level criteria. The maximum demand versus Life Safety capacity is approximately 115% for the linear procedures and 112% for the nonlinear procedures. This is equal to an 87% demand to Collapse Prevention capacity ratio for the linear procedures and 75% for the nonlinear procedures.

Figure X below shows that the axial force and deformation demands in the braces in the Y-direction also do not exceed the Life Safety Performance Level criteria with maximum demand to Life Safety capacity ratios of approximately 123% for the linear procedures and 118% for the nonlinear procedures. This is equal to a 92% demand to Collapse Prevention capacity ratio for the linear procedures and 79% for the nonlinear procedures. As stated previously, the demand-capacity ratios from the ASCE 41-06 analyses are smaller in the X-direction than in the Y-direction because the yield strength of the building is greater in the X-direction.

The demands from the linear static and dynamic procedures were similar and resulted in an average demand of 100% of the brace Life Safety capacity, which is significantly higher than the average demand from the nonlinear results in both directions. The nonlinear static procedures had more variation depending on the level than did the results from the linear procedures; however, the average brace demands were approximately 75% of their Life Safety capacity. Again, the tendency for force to concentrate in the lower floors is apparent, however, no pronounced soft story behavior was observed. The nonlinear dynamic procedure resulted in the lowest and most uniform demands at each level with demands equal to 30% of their Life Safety capacity. The basic observation from the analysis summarized here is that the equivalent lateral force design approach produces a reasonable design with better than expected overall BRBF performance that exceeds the expected Collapse Prevention performance level for a BSE-2 Earthquake Hazard Level.

Figure W: Brace DCR in X Direction

Figure X: Brace DCR in Y Direction

5.2.2 Drift Results

The BSE-2 Hazard Level drift results were similar to those for the BSE-1 Hazard Level. As seen in Figures Y and Z below, the nonlinear static procedures produced greater interstory drift ratios at floors 2 through 4, while the linear procedure drift results governed at the higher floors. Like the BSE-1 Hazard Level results, the nonlinear static procedures performed using PERFORM 3D indicated similar interstory drifts to those from the ETABS analysis for both the uniform loading pattern and the triangular loading pattern. The drift results from the nonlinear dynamic procedure were the lowest for each floor and remained below 1%.

The maximum interstory drift ratio of 2.3% resulting from the nonlinear static procedure occurred at the third floor in the Y-direction, while an average 0.75% permanent drift resulted from the NDP. Based on Table C1-3 of ASCE 41-06, typical story drifts for a building with steel braced frames are 2% transient or permanent for a Collapse Prevention Performance Level. As stated previously, drift values are only intended to be qualitative descriptions of the approximate behavior of the structure and are not to be used as acceptance criteria. However, the story drifts from the BSE-2 Hazard Level analyses represent story drifts that would be typical for a Collapse Prevention Performance Level.

Figure Y: Story Drift in X Direction

Figure Z: Story Drift in Y Direction

The maximum roof displacement results shown in Figures AA and BB below indicate that the overall building displacement decreases for each analytical procedure, with the NDP producing the lowest displacement.

Figure AA: Maximum Roof Displacements in X Direction

Figure BB: Maximum Roof Displacements in Y Direction

5.3 Pushover Curves

Static pushover analyses were conducted in two different programs in order to validate the accuracy of the structural model. The pushover curves in Figures CC and DD on the following page depict the force and corresponding displacement in the building when pushed to failure during the nonlinear static procedure using a uniform and a triangular loading profile. The results indicate that the ETABS and PERFORM 3D model have the same elastic stiffness and yield point in both the X and Y-direction. The post yield stiffness of the ETABS model is slightly greater than the post yield stiffness of the PERFORM 3D model in the Y-direction. This could be due to modeling differences in the hinge properties between the two programs, which could affect the post yield stiffness of the building. However, overall the ETABS and PERFORM 3D models result in similar

linear and nonlinear behaviors for the building and confirm the validity of the analysis from both programs.

Figure CC: Pushover Curve in X Direction

Figure DD: Pushover Curve in Y Direction

5.4 Performance Comparison with Different Building Types

The research presented in this paper was also used to compare the performance of a BRBF structure with different lateral force resisting systems. Two similar projects were conducted using Special Concentrically Braced Frames (SCBF) and Special Moment Frames (SMF) designed per code and analyzed using ASCE 41-06. The results from the research on the SCBF structure found that the building met the expected Life Safety Performance criteria when analyzed using the linear analyses. However, the results from the nonlinear analyses showed the building performed worse than expected and only achieved a Collapse Prevention Performance Level for the BSE-1 Earthquake Hazard Level. The results from the research on the SMF structure were more in line with the results from this paper with the structure achieving the expected Life Safety Performance Level using the linear procedures and an Immediate Occupancy Performance Level using

the nonlinear procedures. The difference in performance level of the SCBF in comparison to the BRBF is likely due to the fact that SCBFs have limited ductility with most of the inelastic deformation concentrating in the compression braces. In contrast the equal tension and compression capacities of BRBs allow for a more even distribution of the inelastic deformation, resulting in minimal degradation of the strength and stiffness of the braces. In addition, the moment resisting frame beam-to-column connections in the BRBF structure provide significant frame action, causing the structure to behave more like a SMF.

The research completed shows that analysis of a code based designed SMF and BRBF yield similar results with the buildings achieving an Immediate Occupancy Performance Level for the BSE-1 Earthquake Hazard Level. In contrast, results from the analysis of a code based designed SCBF structure indicated the building performed to a Collapse Prevention Performance Level for the BSE-1 Hazard Level. Future work should be completed to verify the performance results of the SCBF structure, as all structures designed using ASCE 7-05 level forces are required to meet the Life Safety Performance Level criteria.

6.0 CONCLUSIONS

For this thesis, the history of BRBFs including previous analytical and experimental studies along with applicable codes and provisions were summarized. A BRBF structure was designed according to the current building codes and standards, including American Society of Civil Engineers (ASCE) Standard 7-05, *Minimum Design Loads for Buildings and Other Structures*. Analytical models were developed using ETABS and PERFORM 3D. Performance objectives were established and the structure was analyzed using the four analysis procedures prescribed in ASCE 41-06, *Seismic Rehabilitation of Existing Structures*. The BRBF performance was compared with the expected Life Safety Performance Level for the BSE-1 Hazard Level and to a Collapse Prevention Performance Level for the BSE-2 Hazard Level to determine the adequacy of the code based design.

The significant findings and conclusions from this study are as follows:

- For the Linear Static Procedure conducted in this thesis, the brace demands were approximately 65% of their Life Safety capacity for the BSE-1 Hazard Level and 100% for the BSE-2 Hazard Level. Based on these brace demands, the building achieved a Life Safety Performance Level under BSE-1 and a Collapse Prevention Performance Level under BSE-2.
- For the Linear Dynamic Procedure, results showed a decrease in the brace demands by an average of 8% for both Hazard Levels when compared with the demands from the Linear Static Procedure. The building achieved

a Life Safety Performance Level under BSE-1 and a Collapse Prevention Performance Level under BSE-2.

- The performance of the building improved significantly with the Nonlinear Static Procedure in comparison to the linear procedures. The brace demands in the lower 4 levels were in the Life Safety Performance range while the upper floors remained within the Immediate Occupancy range under BSE-1. For BSE-2, all brace demands remained within the Life Safety range. Overall, the building achieved a Life Safety Performance Level under the BSE-1 and BSE-2 Hazard Levels.
- The Nonlinear Dynamic analysis further reduced the brace demands for both the BSE-1 and BSE-2, improving the building performance level to Immediate Occupancy for the BSE-1 Hazard Level and Life Safety for the BSE-2 Hazard Level.
- The use of nonlinear analysis procedures did change the assessment of the building performance results and did improve the building's overall performance level for the BSE-1 and BSE-2 Hazard Levels.
- The brace performance levels from the nonlinear procedures were better than the expected result of Life Safety and are consistent with the resulting story drifts.
- The discrepancy between the code based max inelastic roof displacement and the ASCE 41 displacements is due to the fact that the code

amplification factors are too low and produce non-conservative lateral displacements.

- The linear and nonlinear analysis acceptance criteria established in ASCE 41 are not adequate to accurately capture the significant ductility of BRBs. It is recommended that modifications be made to these acceptance criteria to incorporate the behavior of BRBs.
- Until performance criteria that are more representative of BRB behavior are included in the ASCE 41, test data should be used to develop the nonlinear hinge properties and acceptance criteria for BRBs.
- The nonlinear static analysis results for ETABs and PERFORM were consistent with one another, verifying the accuracy of the structural models. Use of either program to perform a nonlinear analysis of a BRBF would be acceptable.
- A BRBF performs similarly to a SMF and better than an SCBF structure when assessed with the same performance criteria.
- Further research on this topic to expand on the conclusions made should include the addition of soil springs to assess the contribution of soil-structure interaction on the performance of a structure. Additional building configurations, building locations, and lateral force resisting systems could also be included.

7.0 REFERENCES

- (ASCE 7-05) American Society of Civil Engineers. *ASCE 7-05 Minimum Design Loads for Buildings and Other Structures*. Reston, VA. 2006.
- (ASCE 41-06) American Society of Civil Engineers. *ASCE 41-06 Seismic Rehabilitation of Existing Buildings*. Reston, VA. 2007.
- (Asgarian and Amirhesari 2008) Asgarian B. and N. Amirhesari. "A Comparison of Dynamic Nonlinear Behavior of Ordinary and Buckling Restrained Braced Frames Subjected to Strong Ground Motion." *The Structural Design of Tall and Special Buildings* 17 (2008): 367-386.
- (Astaneh-Asl 2005) Astaneh-Asl, Abolhassan. "Design of Shear Tab Connections for Gravity and Seismic Loads." Steel Tips. *Structural Steel Education Council*. 1 June 2005. June 2009. < http://www.steeltips.org/steeltips/tip_details.php?id=90>
- (Fahnestock, *et al* 2003) Fahnestock, Larry, Richard Sause, James M. Ricles, and Le-Wu Lu. "Ductility Demands on Buckling-Restrained Braced Frames Under Earthquake Loading." *Earthquake Engineering and Engineering Vibration*. 2 (2003): 255-268.
- (FEMA 274) United States Department of Homeland Security's Federal Emergency Management Agency. *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings*. Washington, DC: US Dept. of Homeland Security, 1997.
- (FEMA 356) United States Department of Homeland Security's Federal Emergency Management Agency. *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. Washington, DC: US Dept. of Homeland Security, 2000.
- (FEMA 450) United States Department of Homeland Security's Federal Emergency Management Agency. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. Washington, D.C.: US Dept. of Homeland Security, 2003.
- (Ghobarah 2001) Ghobarah, Ahmed. "Performance-Based Design in Earthquake Engineering: State of Development." *Engineering Structures* 23 (2001): 878-884.
- (Hussain et al. 2005) Hussain, Saif, Paul Van Benschoten, Mohamed Al Satari, Silian Lin. *Buckling Restrained Braced Frames (BRBF) Structures: Analysis, Design and Approvals Issues*. Coffman Engineers, Inc., Los Angeles, CA, 2005.

- (IBC 2006) International Code Council. *International Building Code*. USA: International Code Council, 2006.
- (Lopez 2008) Lopez, Walterio A., “On Designing with Buckling-Restrained Braced Frames.” *Structure Magazine* (2008): 40-41.
- (Lopez and Sabelli 2004) Lopez, Walterio A. and Rafael Sabelli. “Seismic Design of Buckling-Restrained Braced Frames.” *Structural Steel Education Council Technical Information and Product Service* (2004): 8-46.
- (Lopez *et al.* 2002) Lopez, Walterio, David S. Gwie, C. Mark Saunders, and Thomas W. Lauck. *Lessons Learned From Large-Scale Tests of Unbonded Braced Frame Subassemblages, 2002*. Sacramento: Proceedings 71st Annual Convention, Structural Engineers Association of California, 2002.
- (Priestley 2000) Priestley, M.J.N. “Performance Based Seismic Design.” *Bulletin of the New Zealand National Society for Earthquake Engineering* 33 (2000): 325-346.
- (Sabelli, Mahin, and Chang 2003) Sabelli, R., S. Mahin, and C. Chang. “Seismic Demands on Steel Braced Frame Buildings with Buckling Restrained Braces.” *Engineering Structures* 25 (2003): 655–666.
- (Vision 2000) Structural Engineers Association of California. *Vision 2000: Performance Based Seismic Engineering of Buildings*. Sacramento, CA: SEAOC, 1995.

LIST OF ACRONYMS

AISC	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
ATC	Advanced Technology Council
BRB	Buckling Restrained Brace
BRBF	Buckling Restrained Braced Frame
BSE-1	Basic Safety Earthquake 1
BSE-2	Basic Safety Earthquake 2
BSO	Basic Safety Objective
CBC	California Building Code
CBF	Conventional Braced Frame
DSA	Division of the State Architect
FEMA	Federal Emergency Management Agency
IBC	International Building Code
LDP	Linear Dynamic Procedure
LSP	Linear Static Procedure
NEHRP	National Earthquake Hazards Reduction Program
NDP	Nonlinear Dynamic Procedure
NSP	Nonlinear Static Procedure
OSHPD	Office of Statewide Health Planning and Development
SCBF	Special Concentrically Braced Frame
SEAOC	Structural Engineers Association of California

APPENDIX

Figure EE: Scaled Ground Motion 1

Figure FF: Scaled Ground Motion 2

Figure GG: Scaled Ground Motion 3

Figure HH: Scaled Ground Motion 4

Figure II: Scaled Ground Motion 5

Figure JJ: Scaled Ground Motion 6

Figure KK: Scaled Ground Motion 7

Figure LL: Nippon Test Data and Design Information (2006)

Nippon Steel Engineering Co., Ltd
Unbonded Brace™

Typical Brace Stiffness Assumptions

$$\frac{1}{K_{total}} = \frac{1}{K_y} + \frac{2}{K_{con}} + \frac{2}{K_g}$$

where

$$K_y = \frac{A_y E}{L_y} \quad \text{Stiffness of yielding portion}$$

$$K_{con} = \frac{A_c E}{L_c} \quad \text{Axial stiffness of connection (10-15 times greater than } K_y)$$

$$K_g \quad \text{Stiffness of gusset/beam/column (large, assume rigid)}$$

$$\frac{1}{K_{total}} \sim \frac{1}{K_y} + \frac{1}{5K_y}$$

$$K_{total} = 0.83 \frac{A_y E}{L_y}$$

Typical Brace Length Assumptions

For usual building story heights and bay widths:

L_{wp} = work point to work point length

L_1 = overall brace length, $L_1 \sim 0.65 \times L_{wp}$

L_y = brace yielding length, $L_y \sim 0.50 \times L_{wp}$

Considering assumptions above, for typical configurations and brace sizes:

$$K_{total} \sim 1.66 \frac{A_y E}{L_{wp}}$$

Nippon Steel Engineering Co., Ltd
Unbonded Brace™

UC San Diego Testing 2006
Maximum Tension / Compression Ratios

Test	Strain (%)	USB-1 (100 kip)			USB-2M (235 kip)			USB-3M (330 kip)			USB-4M (700 kip)			USB-5M (1200 kip)		
		C_n	T_n	C_n/T_n	C_n	T_n	C_n/T_n	C_n	T_n	C_n/T_n	C_n	T_n	C_n/T_n	C_n	T_n	C_n/T_n
2	0.75	1.13	1.13	1.00	1.14	1.14	1.00	1.13	1.08	1.04	1.11	1.06	1.06	1.18	1.12	1.06
3	1.50	1.32	1.32	1.00	1.35	1.31	1.04	1.35	1.30	1.04	1.33	1.26	1.06	1.41	1.34	1.06
4	2.25	1.55	1.48	1.05	1.51	1.42	1.06	1.52	1.43	1.06	1.47	1.38	1.07	1.56	1.46	1.07
5	3.00	1.66	1.53	1.08	1.64	1.50	1.09	1.65	1.53	1.08	1.58	1.46	1.08	1.67	1.54	1.08

Nippon Steel Engineering Co., Ltd.
UC San Diego Testing 2006

Nippon Steel Engineering Co., Ltd. Unbonded Braces™

Nippon Steel Engineering Co., Ltd.
UC San Diego Testing 2006

Force-Displacement Loops for Tests 1-6, Specimen UBB-1

Alipon Steel Engineering Co., Ltd.
U.C. San Diego Testing 2006

Force-Displacement Loops for Tests 1-6, Specimen UBB-2M

Nippon Steel Engineering Co., Ltd.
UC San Diego Testing 2006

Force-Displacement Loops for Tests 1-6, Specimen USB-3M

Japan Steel Engineering Co., Ltd.
U.C. San Diego Testing 2006

Force-Displacement Loop for Test 1-6, Specimen UBB-4M

Nippon Steel Engineering Co., Ltd.
U.S. San Diego Testing 2006

Force-Displacement Loop for Tests 1-6, Specimen UBB-5M

Alipan Steel Engineering Co., Ltd.
U.C. San Diego Testing 2006

Material Specifications

Core Plate	ASTM A36 JIS SS400 (similar to ASTM A36) JIS SM400 (similar to ASTM A572/50) JIS SM520 (similar to ASTM A572/50) JIS SM400 (similar to ASTM A1045/36) JIS SM420 (similar to ASTM A1045/50) * Minimum core plate thickness is 12mm (1/2 in.) [10mm (3/4 in.) or thicker is recommended].
Steel Tube	JIS STKR400, STK400 or ASTM A500, Thickness : 3.2 mm to 16 mm Width or Diameter : 100 to 500 mm (larger sizes are possible upon request)
Mortar	As per Nippon Steel Engineering's Technical Specification
Design of Steel Tube	As per Nippon Steel Engineering's Technical Specification

Contacts

USA	■ Sales	Mitsui Steel, Inc. 601 S Figueroa St., Suite 1900, Los Angeles, CA 90017 Marcus Daniel tel: 213-896-1142 fax: 213-689-7935 e-mail: M.Daniel@mitsui.com
	■ Technical Information	Seismic Isolation Engineering, Inc. 1144 65th St. Unit D, Emeryville, CA 94608 Ian Aiken, Ph.D., P.E. tel: 510-595-7498 fax: 510-595-7499 e-mail: ida@siecorp.com website: www.siecorp.com
Japan	■	Nippon Steel Engineering Co., Ltd. Building Construction & Steel Structures Division 1-5-1 Osaki, Shinagawa-ku, Tokyo Toyoki Kuroiwa tel: +81-3-6665-4366 fax: +81-3-6665-4812 e-mail: kuroiwa.toyoki@nse-eng.co.jp

NIPPON STEEL ENGINEERING