

BUCKLING RESTRAINED BRACED FRAMES AS A SEISMIC FORCE RESISTING
SYSTEM

by

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Abstract

The hazards of seismic activity on building structures require that engineers continually look for new and better methods of resisting seismic forces. Buckling restrained braced frames (BRBF) are a relatively new lateral force resisting system developed to resist highly unpredictable seismic forces in a very predictable way. Generally, structures with a more ductile lateral force resisting system perform better in resisting high seismic forces than systems with more rigid, brittle elements. The BRBF is a more ductile frame choice than special concentrically braced frames (SCBF). The ductility is gained through brace yielding in both compression and tension. The balanced hysteretic curve this produces provides consistent brace behavior under extreme seismic loads. However regular use of the BRB is largely limited to Japan where the brace type was first designed.

The wide acceptance of buckling restrained braced frames requires the system to become easily designable, perform predictably, and common to engineers. This report explains the design process to help increase knowledge of the design and background. This report also details a comparison of a BRBF to a SCBF to give familiarity and promote confidence in the system.

The design process of the BRBF is described in detail with design calculations of an example frame. The design process is from the AISC Seismic Provisions with the seismic loads calculated according to ASCE 7 equivalent lateral force procedure. The final members sizes of the BRBF and SCBF are compared based on forces and members selected. The results of the parametric study are discussed in detail.

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Dedication

To my wife. If it weren't for engineering homework we would have never been. I love you Alia.

Go Cats!!

CHAPTER 1 - Introduction

In a time of great advances in technology, the construction industry is no different. The design of buildings for seismic events should be improving with time and knowledge gained. The introductions of new lateral force resisting systems that more efficiently absorb the energy imposed on a structure during an earthquake are desired. Buckling restrained braced frames (BRBF) are one of these newer systems. BRBF have been used widely in Japan and are gaining acceptance in the United States. BRBF design procedures have just been established in recent years and there is a lack in actual performance information in the United States. The goal of this report then is to explain the design process for a BRBF as part of the lateral force resisting system and compare it to a more common lateral system, the special concentrically braced frame (SCBF).

To fully understand the relationship a lateral force resisting system has with the other members of a building frame, many factors need to be understood. These factors include how an earthquake affects a structure as a whole, how that structure internally transfers forces between members, and how the earthquake forces are ultimately absorbed.

This report focuses on BRBF as a seismic force resisting system. The BRBF is compared to the more widely understood SCBF in a parametric study. The two frames are compared throughout the entire design process.

The earthquake load on the structure is calculated and distributed to the lateral force resisting system. The two lateral force resisting systems are then compared based on required members and forces. The results of this report are for comparison of the two frames designed. The design of a BRBF including the determination of the seismic force, frame load calculations, and frame member selections is presented in this report. The results of this design and comparison are not to develop a new design approach or analytical provision, but to provide an objective side by side comparison of these two lateral force resisting systems.

CHAPTER 2 - Seismic Events and Buildings

In this chapter the process through which seismic forces are applied to a structure is described in detail. The determination of the seismic ground motions that will be applied to a structure are described using the current seismic provisions. The relationship between the classification of an earthquake ground motion as a maximum considered earthquake (MCE) or design ground motion is described. Methods used to absorb the ground motions from an earthquake are presented followed by how the appropriate building code stipulates how a building should perform during an earthquake.

The International Building Code 2006 (IBC) is used in this report as the governing building code (ICC 2006). Building forces are determined from the IBC except where it prescribes the use of another standard. The IBC stipulates the use of American Society of Civil Engineers ASCE 7-05 *Minimum Design Loads for Buildings and Other Structures* (ASCE 7) to determine the earthquake effects on all structures and their components (ASCE 2005). The ASCE 7 seismic provisions for design of a structure using the simplified approach of Equivalent Lateral Force Procedure (ELFP) are described.

The earthquake effects that a structure is required by ASCE 7 to resist are not earthquake magnitudes, but ground motions. The design of a structure for an earthquake magnitude is a very complex process. Determining the earthquake magnitude to which a structure should be required to resist is difficult to determine and even more difficult to provide a consistent margin of safety against collapse for the entire United States. For these reasons, the ASCE 7 seismic provisions determine a ground motion to be resisted. These ground motions are based on the accelerations of the seismic waves as they move through the earth and contact a structure. How these ground motions are turned into forces applied to a structure is explained.

Ground Motion

The force on a building from an earthquake is the result of ground acceleration relative to the structure. The acceleration produced by an earthquake reverses directions multiple times before the earthquake subsides. These reversing or cyclic accelerations result in a dynamic force being applied to the building. Due to the complexity of a dynamic analysis for a structure loaded

by an earthquake ASCE 7 provides a simplified approach in which the seismic force is applied to the building as a static force. This approach is referred to as the Equivalent Lateral Force Procedure (ELFP) in ASCE 7. The static force used for design is calculated from the ground accelerations that are possible at the site where the building is located. These local ground accelerations at the site are calculated from the maximum considered earthquake ground motions (MCE ground motions). The MCE ground motions are defined as the maximum level of earthquake ground acceleration that is considered as reasonable to design normal structures to resist. The ASCE 7 ELFP uses MCE ground motions instead of earthquake magnitude which helps provide a consistent margin of safety against collapse for all locations in the United States. The safety margin is founded in the MCE ground motion being defined by site specific accelerations derived from previous earthquake data, earthquake return probabilities, and soil characteristics of the site among other factors. This data is used to develop spectral response accelerations specific for any location (ASCE 2005; FEMA 2004; Leyendecker et al. 2000).

Spectral response accelerations are based on return rates which are described as a probability of exceedance in a 50 year period. ASCE 7 currently uses a 2% exceedance in 50 year return probability for MCE ground motions. The return period for this probability of acceleration is approximately 2500 years. The 2% exceedance in 50 year accelerations are called mapped MCE spectral response acceleration parameters. Since designing every structure to this extreme level of probability would be too costly, ASCE 7 uses what are called design MCE spectral response acceleration parameters for the calculation of the seismic force in the ELFP. The design level of the mapped acceleration parameters are attained by multiplying the mapped MCE spectral response acceleration parameters by $2/3$. The $2/3$ factor is used to provide every structure with a consistent margin of safety against collapse. Since the 2% exceedance in 50 years acceleration is multiplied by a factor of $2/3$ to attain the design level, there is no return probability associated with the design accelerations used (ASCE 2005; FEMA 2004; Leyendecker et al. 2000).

Earthquakes are formed by a wave moving through the earth from a disturbance in the earths crust. These waves travel through the soil at different speeds and periods. Since the acceleration of the earthquake is dependent on the period of the wave, seismic design must take the period into account. ASCE 7 utilizes mapped MCE spectral response accelerations for short and long periods of 0.2 seconds and 1.0 seconds respectively. These two different periods were

selected and simplified for design purposes from earthquake and test data and are used to calculate the acceleration imposed on the structure according to the ELFP. The short period response acceleration is used to calculate acceleration imposed on the structure while the long term response acceleration is used to calculate minimum and maximum accelerations the structure should be designed to resist (ASCE 2005; FEMA 2004; Leyendecker et al. 2000).

Energy Absorption Methods

Energy absorption systems for a building can be divided into two main types; passive and active. A passive system is one that reacts to the vibration and movement produced by an earthquake. An active system is one that uses mechanical devices whose characteristics change based on real time measurements of earthquake responses. A combination of the two systems can be used to give reliable redundancy. The passive system of supplemental energy dissipation devices is the focus of this report.

Two main types of passive supplemental energy dissipation devices are metallic yielding and frictional devices. These devices are hysteretic devices because they absorb energy through displacements within the device. The frictional device uses sliding contact friction to accomplish this. The surfaces that slide past each other during an earthquake develop friction to resist the movement and absorb the energy (Hanson and Soong 2004).

The metallic yielding devices use deformations in metal during yielding to absorb energy. The deformations in the metal are a result of ductility. A ductile metal is able to yield under the application of load at normal temperatures. The capacity of a ductile metal to undergo large deformations before failure while retaining strength makes it very suitable as an energy dissipation device. The ductile metal, when subjected to sufficiently large forces, will be stressed beyond the elastic range into the inelastic or plastic range.

The goal of ASCE 7 ELFP is to keep the main structural components of the building in the elastic range, which may require the lateral force resisting system (LFRS) to enter into the plastic range to absorb the seismic energy induced into the building. The plastic range of most ductile LFRS has a large capacity for energy dissipation. The energy is dissipated through deformations in the members through yielding which provides for large amounts of energy to be absorbed (Kelly 2008).

A common building material used for energy absorption because of its predictable yielding and ductility is structural steel. A ductile material has visible signs of high loading such as deformation and necking. The stress strain curve graphed in Figure 2-1 for sharp yielding structural steel under tension represents these characteristics.

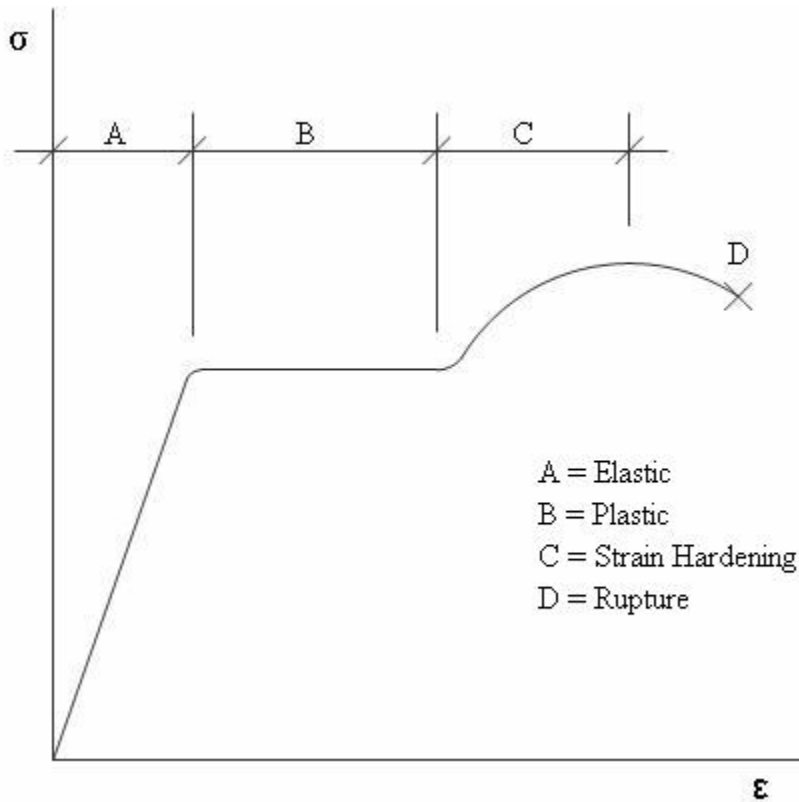


Figure 2-1. Stress strain graph of sharp yielding structural steel.

Segment A of Figure 2-1 represents the elastic range. During this stage of loading, an increase in stress results in a small linear increase in strain, or deformation. The main structural components are kept in this stage because the deformations in the elastic stage are small and not permanent. Some portions of the LRFS are designed to enter segment B of Figure 2-1 which is the plastic range. During this stage of loading, the steel undergoes deformations at a constant stress level. During the plastic stage there are residual deformations, but these permanent deformations are typically not large enough to affect a member's structural capacity once the load has been removed. The next stage is segment C which is strain-hardening. The LRFS is

designed to enter this stage only under extreme loading. Strain-hardening is where the member can undergo even larger deformations, but the load applied is no longer constant. Necking occurs after the maximum tensile load is reached and the member cross section decreases slightly due to local instabilities. During the necking stage, the load carrying capacity decreases with increased deformations until the member ruptures. Rupture is marked by point D in Figure 2-1.

Both frictional and metallic yielding devices are called hysteretic devices because they exhibit the same force dissipation in both tension and compression. In the steel material presented above, this is seen as a balance hysteretic loop for cyclic positive and negative forces. This is desirable and needed for seismic design because of an earthquake's cyclic nature. A hysteretic loop is shown in Figure 2-2 for sharp yielding steel as in Figure 2-1.

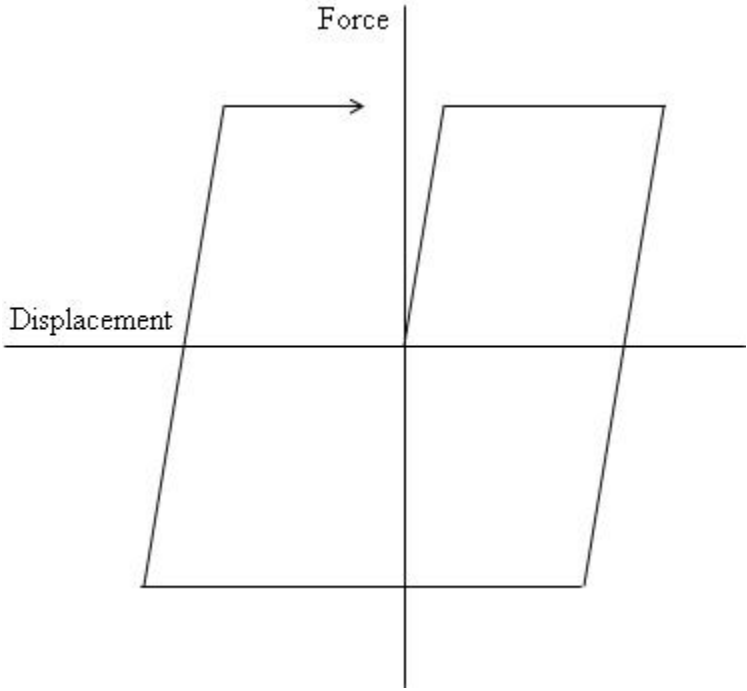


Figure 2-2. Ideal hysteretic loop for structural steel.

The displacement, or deformation, of the steel during cyclic loading acts cyclically as well. As the force applied transitions from a maximum positive to negative force, the displacement transitions between maximum positive and negative displacements. The maximum and minimum displacements occur at the same force level only in opposite directions. The

balanced hysteretic loop of Figure 2-2 represents the elastic and plastic portions of Figure 2-1 for positive and negative stresses.

By allowing some of the elements of the LFRS to undergo cyclic deformation as shown in Figure 2-2, the LFRS absorbs the energy of the earthquake. The intent of seismic design is to limit permanent displacements to the LFRS so the remainder of the building structural system will not have permanent deformations.

Building Performance Levels

Building performance levels are a way of establishing how a building should perform at different levels of seismic ground motion, which is a method of performance based design. Figure 2-3 shows a graph of how buildings with different occupancies could be expected to perform for different levels of ground motions when designed according to the provisions of ASCE 7.

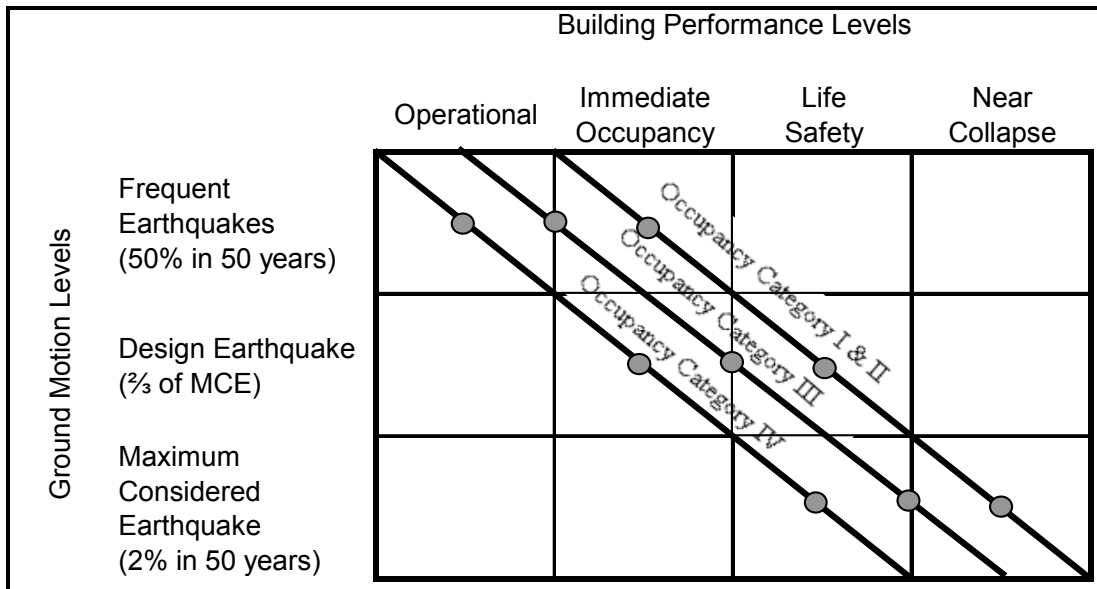


Figure 2-3. Projected building performance based on seismic design provisions of ASCE 7.

In Figure 2-3 buildings are separated into occupancy categories and in turn importance categories by ASCE 7. The occupancy category is a rating system for buildings based on life safety and preservation which includes the importance of the building for post disaster recovery.

Currently there are four occupancy categories designated by ASCE 7 and they are each assigned an importance factor. Occupancy Category IV is for essential facilities that are vital to recovery efforts after an earthquake like police and fire stations or hospitals and are assigned an Importance Factor of 1.5. Occupancy Category III is for buildings that will potentially be a hazard to human life and have many occupants or are capable of being places of refuge during or after the disaster. Also in Occupancy Category III are buildings that may cause substantial economic impact if lost. These buildings are assigned an Importance Factor of 1.25. Buildings assigned an Importance Factor of 1.0 are from both Occupancy Category I and II. Occupancy Category II buildings are the buildings that do not fit into the other categories, and could be small office buildings, stores, or residences. Occupancy Category I buildings are structures that represent little threat to human life and do not meet the requirements of the previous three categories. Some structures that fall into this category are agricultural or storage buildings (ASCE 2005; FEMA 2004).

These importance factors are illustrated in Figure 2-3 and will be explained below. It should be noted that the goal of the lowest level of seismic design performance according to ASCE 7 is to preserve life during and after a design earthquake, not necessarily prevent extensive structural and non-structural damage. The two goals usually coincide, but they are independent functionalities of design.

Ground Motion Levels

The ground motion level is divided into three classifications as shown in Figure 2-3. These ground motion levels represent practical levels and the expected building performance level based on occupancy category when designed according to the ASCE 7 seismic requirements. However the ground motion levels are also categorized by the probability of return cycles. These earthquake ground motions are classified as follows:

- Frequent Earthquake (50% in 50 years)
- Design Earthquake (2/3 of MCE)
- Maximum Considered Earthquake (2% in 50 years)

Building Performance Levels

The following sections will describe in more detail the four building performance levels shown in Figure 2-3. These different performance levels represent the expected condition of the

building after a design earthquake. Following these descriptions for a design earthquake, the building performance at each ground motion level is described for each Occupancy Category.

Operational

At the operational level a building should have minor to no structural damage. The building should perform in a near elastic state and should remain operational during and immediately after the earthquake. This level of performance represents no threat to human life. It should be noted that there are more than structural requirements for this performance level. All of the building mechanical, electrical, and other systems must remain intact and operational during and after the earthquake (FEMA 2004).

Immediate occupancy

The immediate occupancy level is similar to the operational level in the structural performance of the building. The difference in this level is the damage to nonstructural components of the structure. The structure is expected to retain nearly all of its original strength, which means there should be very little inelastic behavior of the structural system during the earthquake. The nonstructural elements within the building are expected to require repair and clean up before the building can resume its normal functionality. The threat to human life in this performance level is very slight (FEMA 2004).

Life Safety

The life safety occupancy level is exactly that, design to life safety or preservation. The risk to human life in this occupancy level is low. Significant structural and nonstructural damage are expected. The damage should not pose a significant threat to human life during the earthquake, but continued occupancy is not safe until required repairs are made. The repair of the structure should be possible but commonly at great cost. After the earthquake the structure will still have structural lateral capacity to prevent collapse but not for life safety. At this level there will be visible damage to the seismic structural system such as cracking, spalling, yielding and buckling. The structure may have permanent lateral offsets. The structure may require demolition in some cases (FEMA 2004).

Near Collapse

The final and most damaging performance level is the near collapse or collapse prevention level. At this level the structural system has sustained damage almost to its capacity. There will be members with extreme cracking, spalling, buckling and rupture. The structure has very little margin of safety against collapse if further ground motions should occur. There will be significant lateral displacements at or exceeding code minimum requirements. Nonstructural damage will be so extensive that objects may become detached and present falling hazards. The structure will have so much damage that repairs required to return to functionality is usually not feasible. The threat to human life at this level is moderate. (FEMA 2004)

Occupancy Category Performance Levels

A building that is in the occupancy categories of I and II, is the most common building type. When this building is designed according to ASCE 7 seismic design, its performance level for a frequent earthquake is expected to be in the immediate occupancy level. For the design earthquake of 2/3 MCE, the building is expected to undergo structural damage to the level of life safety. Finally, if this building were subjected to the MCE of 2% in 50 years exceedance, the structure is expected to preserve human life but at a near collapse level.

A building that falls into the ASCE 7 Occupancy Category III is a building that if structural failure occurs, there would be a substantial hazard to human life. As this structure undergoes frequent earthquake ground motions, it is expected to perform around the operational and immediate occupancy levels. When this structure is subjected to the design earthquake, it is expected to perform slightly better than the life safety level near the immediate occupancy level. If the building is subjected to the MCE the structure should perform between the near collapse and life safety level. At this level, repair of the structure would be more feasible than for the Occupancy Category I and II buildings.

Occupancy Category IV buildings represent the highest performance level of code designed buildings. For these buildings, the frequent earthquake would represent no damage to the operational level. The performance level of this category building at the ASCE 7 design earthquake ground motion would be for immediate occupancy. If this building is subjected to the MCE ground motion it should perform to the life safety performance level. This

performance is needed by this category of building due to the building use and its importance to life safety. All of these relationships are shown in Figure 2-3.

How Seismic Force Is Applied To a Building

The simplified approach in ASCE 7 of Equivalent Lateral Force Procedure (ELFP) is the method used in this report, but is not the only method of seismic force determination provided in the ASCE 7. The seismic provisions also permit a structure to be designed by modal response spectrum analysis of Section 12.9 or seismic response history procedures of Chapter 16. The ASCE 7 seismic provisions only permit the use of the ELFP if the structure meets the requirements of Table 12.6-1. These requirements are loosely described as a structure of regular shape, a period not exceeding a described ratio of short and long period design spectral response parameters, and with limited horizontal and vertical irregularities are present (ASCE 2005).

For structures that meet the requirements of the ELFP in Table 12.6-1, the design procedure starts in Section 12.8 of ASCE 7. The seismic base shear is first determined, then the base shear is distributed vertically to the separate levels of the structure, next the seismic force at each level is distributed horizontally to the elements of the seismic force resisting system at each level, and finally the seismic force is applied to the members of the LFRS using seismic load combinations.

Seismic Base Shear Calculations

The ELFP is the application of dynamic seismic forces to the building as an equivalent static force. This equivalent static force is the seismic base shear, V , which is determined by multiplying the seismic response coefficient, C_s , by the seismic weight of the structure, W , as shown in ASCE 7 Equation 12.8-1 below.

$$V = C_s W \quad \text{(Equation 2-1)}$$

Where

V = seismic base shear, kips

C_s = seismic response coefficient

W = effective seismic weight, kips

The base shear is applied to the structure in two orthogonal directions independently. The base shear force is then distributed vertically to the levels of the building proportionally by the weight of each level. These distributed forces are then used to design the LFRS including diaphragms, chords, collectors, anchorages, and to determine lateral drifts of the building at each story (ASCE 2005). The focus of this report is the design of braced frames. Descriptions of the determination of the seismic response coefficient and the effective seismic weight are provided in the following sections.

C_s is calculated from the short period design spectral response acceleration parameter, S_{DS} , the response modification factor, R , and the importance factor, I . S_{DS} is calculated from the MCE spectral response acceleration adjusted for site class effects, S_{MS} . S_{MS} is calculated from the site coefficient, F_a , and the mapped MCE spectral response acceleration at short periods, S_s .

Design Spectral Response Acceleration Parameters

The design spectral response acceleration parameter at short periods, S_{DS} , and at 1.0 second periods, S_{D1} , are the ground accelerations that are adjusted for the LFRS utilized by the structure and its occupancy. The S_{DS} and S_{D1} parameters are determined by ASCE 7 Equations 11.4-3 and 11.4-4 respectively, which are shown below:

$$S_{DS} = \frac{2}{3} S_{MS} \quad \text{(Equation 2-2)}$$

Where

S_{DS} = design spectral response acceleration parameter at short periods

S_{MS} = MCE spectral response acceleration parameter at short periods

$$S_{D1} = \frac{2}{3} S_{M1} \quad \text{(Equation 2-3)}$$

Where

S_{D1} = design spectral response acceleration parameter at a period of 1 second

S_{M1} = MCE spectral response acceleration parameter at a period of 1 second

The design spectral response acceleration parameters are taken as the MCE spectral response accelerations adjusted for site class effects, S_{MS} and S_{M1} , and multiplied by 2/3. This

multiplication of 2/3 is the ASCE 7 seismic design provisions way of converting the MCE ground motions into design ground motions. Using 2/3 as the multiplier is based on inherent structural overstrength and redundancy which is explained earlier in Chapter 2 of this report (FEMA 2004; Leyendecker et al. 2000).

The MCE spectral response accelerations adjusted for site class effects, S_{MS} and S_{M1} , for short and 1.0 second periods respectively are determined using ASCE 7 Equations 11.4-1 and 11.4-2 below:

$$S_{MS} = F_a S_S \quad \text{(Equation 2-4)}$$

Where

F_a = site coefficient

S_S = mapped MCE spectral response acceleration parameter at short periods

$$S_{M1} = F_v S_1 \quad \text{(Equation 2-5)}$$

Where

F_v = site coefficient

S_1 = mapped MCE spectral response acceleration parameter at a period of 1 second

The S_{MS} and S_{M1} values are determined by multiplying the mapped MCE spectral response accelerations, S_S and S_1 , by site coefficients F_a and F_v . S_S and S_1 are found in Figures 22-1 thru 22-14 of ASCE 7. F_a and F_v are coefficients used to modify the MCE spectral response acceleration to the soil conditions of the site. The S_S and S_1 values, which are for short and 1.0 second periods respectively, are the accelerations that are presented in ASCE 7 as the possible accelerations that a site could experience. These mapped values are normalized to site class B, and therefore must be modified for the actual soil type at the building location. The different site classifications are A, B, C, D, E, or F and are shown in Table 2-1 below.

Table 2-1. Site classification and soil type.

Site Class	Soil Type
A	Hard Rock
B	Rock
C	Very Dense Soil and Soft Rock
D	Stiff Soil
E	Soft Clay Soil
F	Soil Requiring site response analysis

The different soil types transmit the seismic waves in different patterns and speeds. These differences result in an amplification or damping of the seismic waves induced on the structure. Site class A for rock results in the greatest reduction of the MCE ground motion, while site class E soil of soft clay results in the most amplification of the MCE ground motion. The site class F is used when the reaction of a soil type to a earthquake is unknown and further analysis is required to determine the site coefficient.

The effects of the site class on a structure are determined by the site coefficients F_a and F_v . F_a is used to modify the S_S acceleration while F_v is used to modify the S_I accelerations. The coefficients are determined from ASCE 7 Tables 11.4-1 and 11.4-2 respectively. The values of F_a and F_v are based not only on the soil type, but also on the magnitude of S_S and S_I .

Response Modification Coefficient

The response modification coefficient, R , is used to adjust the acceleration, or seismic response coefficient, applied to the structure based on the ductility, damping and overstrength inherent in the structural system. The applied force from ground acceleration is reduced by an amount dependent on the lateral force resisting system (LFRS) in the structure. The greater the ductility of the LFRS, the larger will be the reduction in seismic force that is required for design. This reduced seismic force is then the design level at which the LFRS resists the force in the inelastic range while the remaining structural elements of the building perform in the elastic

range. The response modification coefficient reduces the lateral force to a strength level which is used directly in LRFD load combinations, but the lateral force needs to be adjusted to a service level for ASD load combinations, which is done by the multiplication of the force by the factor of 0.7 (ASCE 2005; FEMA 2004).

R is determined for a structural system through testing and experimental data and building performance history. R is dependent only on the seismic force resisting system, which are the vertical components of the LFRS such as a braced frame, moment frame, or shear wall. The horizontal components of the LFRS such as diaphragms, chords, and drag struts are not used to determine R . The determination of R based only on the vertical components is because the inelastic deformation and damage is assumed to be in them, while the horizontal components are designed to remain in the elastic range.

Importance Factor

The importance factor, I , is used to reduce the response modification coefficient, depending on the occupancy category, and therefore increases the seismic force that must be resisted as the importance of the structure increases. This increase in seismic force is used to insure a higher building performance level for occupancy category III and IV structures. The importance factors for the occupancy categories are given in Table 2-2 below.

Table 2-2. Importance factors based on occupancy category.

Occupancy Category	Importance Factor, I
I, II	1.0
III	1.25
IV	1.5

Seismic Response Coefficient

As previously stated, the seismic response coefficient, C_s , is multiplied by the seismic weight to give the static base shear force. C_s is determined from the seismicity of the site, the

ductility of the lateral system, and the occupancy category of the structure as is shown in ASCE 7 Equation 12.8-2 below.

$$C_s = \frac{S_{DS}}{R/I} \quad (\text{Equation 2-6})$$

Where

R = response modification coefficient

I = importance factor

This is the design acceleration that the ground is assumed to exert on the building. C_s is in the units of gravity, with 1.0 being equal to the vertical acceleration of gravity applied horizontally. C_s is based on the short period design spectral response acceleration parameter, S_{DS} , the response modification coefficient, R , and the importance factor, I . Maximum and minimum values of C_s are based on the 1.0 second design spectral response acceleration parameter and are also a function of the fundamental period, T .

Effective Seismic Weight

Once the seismic response coefficient is determined, it is then multiplied by the seismic weight of the structure to determine the static base shear force. The seismic weight of a structure is a combination of the total dead load of the structure plus some additional loads that may be applicable. The additional loads are loads that can reasonably be expected to be on the structure during an earthquake. Occupant live loads are not required to be included in the seismic weight because they are considered to have negligible contribution to the seismic lateral forces (ASCE 2005; FEMA 2004).

ASCE 7 requires the effective seismic weight of a building to be determined according to Section 12.7.2. This section states that the effective seismic weight of a building is the total dead load of the building with the addition of four other possible loads. The first of these other loads is the inclusion of 25 percent of the floor live load for areas that are used for storage. This load is included because at least a portion of the stored material will likely be present during a seismic event.

The second load that is to be included is the weight of partitions on the floor area. For buildings that are likely to have partitions added or rearranged, the actual weight of the partition or a minimum weight of 10 psf over the floor area is required to be included. This load is applied because the partitions would be considered fixed to the floor and their weight would increase the seismic weight of the structure.

Another load that is required to be included in the effective seismic weight is the total operating weight of permanent equipment. This would include equipment that is not already included in the dead load calculations but will act with the structure during a seismic event. An example of this would be electrical or HVAC equipment and piping.

The last additional load to be included in the effective seismic weight of a structure is a flat roof snow load. Twenty percent of the uniform design snow load is to be included if the flat roof snow load exceeds 30 psf. The requirement of the flat roof snow load to be larger than 30 psf is because if the snow load on the structure is small the contribution of the snow load would be negligibly small.

The effective seismic weight is the additive total of all five of these loads. The weight is typically determined at each level and then combined into a total building weight. The individual weights of the different levels are used in the vertical distribution of the seismic base shear force to the levels of the structure.

Vertical Distribution of Horizontal Force

Once the seismic base shear is calculated it can be distributed vertically to the building as shown in Figure 2-4.

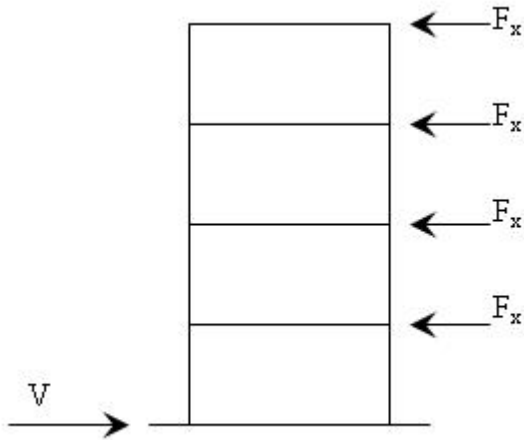


Figure 2-4. Vertical force distribution.

ASCE 7 Section 12.8.3 stipulates how the base shear is to be distributed to each level of the building. The base shear value is divided among the levels of the structure and applied as a concentrated lateral seismic force at each level. These concentrated forces are what the lateral force resisting system is designed to resist.

The vertical distribution of the base shear is determined by effective seismic weight and height of each level. This relationship is given by Equation 12.8-11 and 12.8-12 in ASCE 7 shown respectively below.

$$F_x = C_{vx}V \quad \text{(Equation 2-7)}$$

Where

F_x = lateral seismic force at any level x

C_{vx} = vertical distribution factor

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k} \quad \text{(Equation 2-8)}$$

Where

w_i or w_x = portion of effective seismic weight of the structure located at level i or x

h_i or h_x = height from base to level i or x

k = modification factor of structure period

Equation 2-8 is used to calculate the vertical distribution factor, C_{vx} , for each level by multiplying the seismic weight at the level of interest, w_x , by the height of that level above the base and then adjusted for the structures period, h_x^k . C_{vx} is then multiplied by the seismic base shear (Equation 2-7) to find the portion of the base shear that acts at that level. This process is then repeated for each level of the structure until the corresponding lateral seismic force is determined at each level.

For the design of the lateral force resisting elements at a given level, the lateral seismic force applied at that level is assumed to occur at the center of mass of that level. The center of mass location is typically calculated. The applied seismic force is then transferred throughout the level by the floor diaphragm to the vertical LFRS elements.

Horizontal Force Distribution

Once the seismic force at each level is determined, the distribution of the force horizontally through the level is required. The determination of the diaphragm as flexible or rigid is the first step since it determines how the force is distributed to the vertical LFRS elements. Second, as previously stated, the seismic force at each level is applied through the center of mass of the level. Next the distribution of the seismic force to the vertical elements of the LFRS is calculated. Finally, chords and collecting element forces are calculated.

Seismic Load Combinations

For the design of the seismic force resisting system, ASCE 7 requires the modification of the basic load combinations of Sections 2.3.2 and 2.4.1. The modifications are for insuring adequate redundancy and overstrength in specific portions of the LFRS. These modifications are found in Section 12.4.2.3 and 12.4.3.2 respectively. The load combinations of Section 12.4.2.3 for LRFD are:

$$5. (1.2 + 0.2S_{DS})D + \rho Q_e + L + 0.2S \quad \text{(Equation 2-9)}$$

$$7. (0.9 - 0.2S_{DS})D + \rho Q_e \quad \text{(Equation 2-10)}$$

These load combinations include the redundancy factor, ρ , which is determined from Section 12.3.4, and the modification of the dead load factor, $0.2S_{DS}$ to include the vertical seismic load effects. The redundancy factor is applied to the seismic load effect to increase the seismic load if the structure does not have adequate redundancy according to ASCE 7. If the structure meets the requirements of Section 12.3.4.1 and 12.3.4.2, the redundancy factor can be taken as 1.0 and no increase in the seismic load effect is required.

The vertical seismic load effect is added to the dead load of the building for a vertical force acting downward and is subtracted from the dead load for an upward vertical force. The use of S_{DS} as a multiplier of the dead load to calculate the vertical seismic load effect is similar to C_s multiplied the seismic weight of the structure for calculating the horizontal seismic load. ASCE 7 seismic provisions were developed based on the assumption that it is unlikely that the maximum responses of vertical and horizontal accelerations occur simultaneously, therefore the use of the $0.2S_{DS}$ factor on dead load is deemed sufficient (FEMA 2004).

The modifications of Section 12.4.3.2 to the LRFD basic load combination to include the overstrength factor are given below.

$$5. (1.2 + 0.2S_{DS})D + \Omega Q_e + L + 0.2S \quad (\text{Equation 2-11})$$

$$7. (0.9 - 0.2S_{DS})D + \Omega Q_e \quad (\text{Equation 2-12})$$

These load combinations include the overstrength factor, Ω_o , and the same vertical seismic load effect as the load combination is Section 12.4.2.3. The overstrength factor is included to account for instances where an isolated, individual, brittle element could fail and result in the loss of an entire seismic force resisting system or in an instability leading to collapse. The overstrength factors for the different seismic force resisting systems are found in ASCE 7 Table 12.2.-1. The horizontal seismic load effect is multiplied by Ω_o to include any inherent overstrength that a system includes from the design, material, and system. The design overstrength portion is defined as the difference between the lateral base shear force at which the first significant yield of the structure will occur and the minimum specified force given by design strength. The value of the design overstrength portion will be small for systems that are strength controlled like most braced frames because the system will be designed close to the minimum

requirements of the seismic provisions. The material overstrength portion results from the difference between the minimum required strength used in design and the actual required strength of the members used in construction. This difference is due to the use of conservative lower bound member material strengths rather than the probable actual strength of members within a specific grade. The system overstrength is the ratio of the ultimate lateral force the structure is capable of resisting to the actual force applied that results in the first significant yielding.

The seismic load combinations are to be used for the design of all structural members as they apply, even members not included in the seismic lateral force resisting system. This requirement is to help insure adequate strength in the structure for life safety during a MCE.

CHAPTER 3 - Comparison of BRBF with SCBF

BRBF's are an alternative to special concentrically braced frames (SCBF) within the classification of concentrically braced frame (CBF) systems. The requirements that must be met to be classified as a CBF are as follows (AISC, 2006a):

- A vertical truss system resists lateral loads and is formed by centerlines of members meeting at a joint
- Members are primarily subjected to axial loads
- Bracing members and their connections are expected to undergo significant inelastic deformation into the post-buckling range when exposed to severe earthquake loads

In this report, the lesser known BRBF is compared to a more common SCBF system as the lateral force resisting system (LFRS) for seismic loading. First the differences in mechanics and configuration of the two bracing types are explored and then the force dissipation methods of the two braces are compared and contrasted.

How a BRBF is different than a SCBF in mechanics

The SCBF is a LFRS that develops energy dissipation capacity through the ductility of yielding and plastic hinge formation in the braces. The yielding occurs in the tension brace while the plastic hinge is formed in the compression brace. Since the buckling stress of a brace is typically much lower than the yielding stress, most of the ductility of the system is through the plastic hinge which is a result of brace buckling.

The Components of a Buckling Restrained Brace (BRB)

The mechanics of how a BRBF works is much different than a SCBF even though the frames are very similar when looked at from statics; both systems use a diagonal brace to transfer force through primarily axial loaded members. The differences between the two bracing systems lie in how the SCBF functions under loading beyond the critical buckling stress of the braces. In the following sections the components of a Buckling Restrained Brace (BRB) are explained in

detail with a comparison to the corresponding components in a SCBF. The energy dissipative properties of the two frames are also compared.

The Steel Core

The steel core of the typical BRB is divided into three segments. These segments are the yielding zone, transition zone and the connection zone. The yielding zone is the segment of the brace where all the force will be dissipated through tensile and compressive yielding. This zone has a reduced cross section to insure yielding occurs here and occurs uniformly. This zone is fully braced by the restraining components to allow compression yielding rather than local or overall buckling. The transition zones are the segments of the brace directly on either side of the yielding zone. These segments have larger cross sectional area than the yielding zone but are similarly restrained. The connection zone is the portion of the brace that extends beyond the restraining components and is used to connect the brace to other structural elements of the frame. (Higgins and Newell 2004; Sabelli and Lopez 2004) The configuration of the connection zone changes depending on the connection type used and will be discussed in more depth in the Brace Connection Options section. These segments are shown in Figure 3.1 with the connection zone being “C”, the transition zone “B” and the yielding zone “A”.

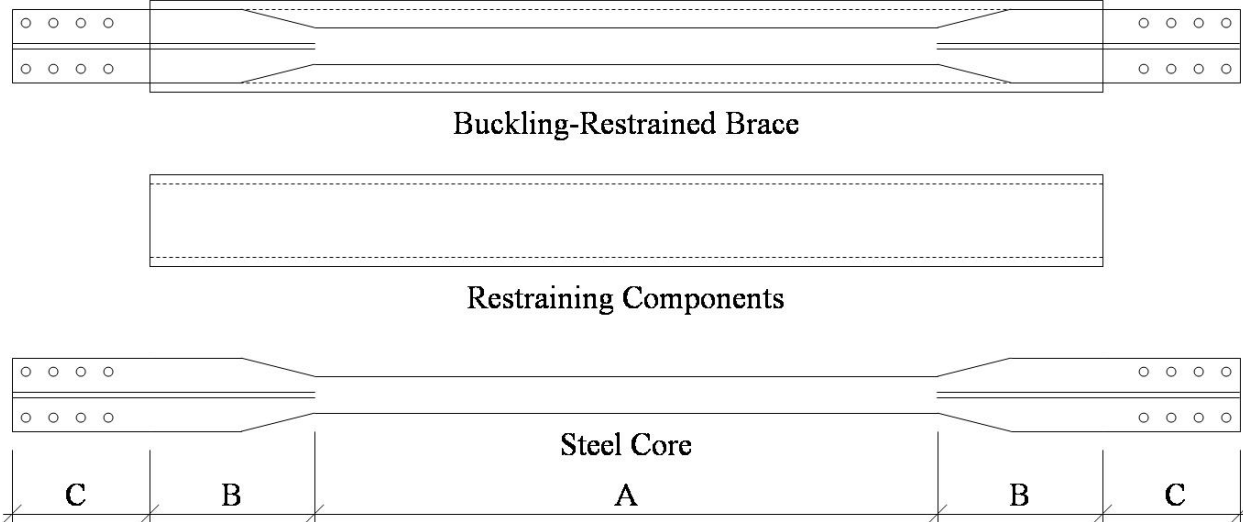


Figure 3-1. Diagram of common buckling-restrained brace components.

Figure 3-1 represents a typical configuration of a BRB, however there are many other configurations that are currently available and others that are being researched. The brace illustrated is the configuration that will be used in Chapter 4 for the illustration of the BRBF design process, but Figure 3-2 shows other configurations that are available. The blackened portions shown are the steel core while the shaded areas represent the concrete or mortar restraining element. The outlined elements in Figure 3-2 (e, g, h, i, j, and k) represent steel shapes used to restrain buckling in place of concrete mortar. Even with so many different configurations, the functionality and components of each brace is essentially the same.

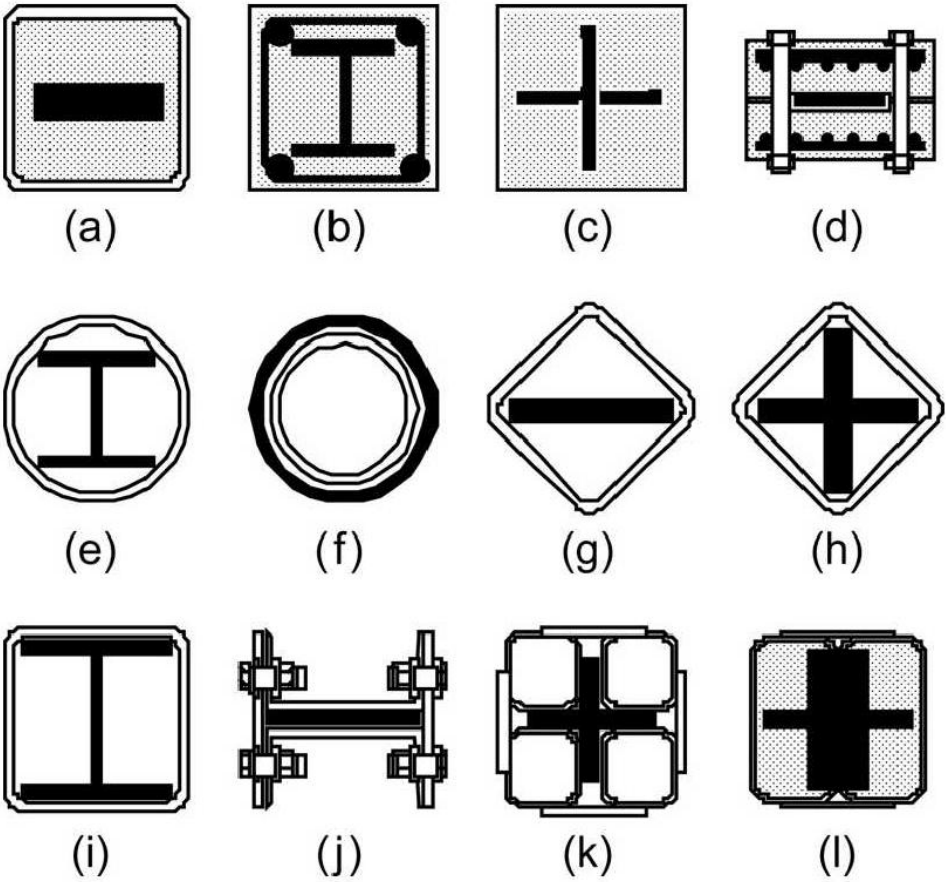


Figure 3-2. Different configurations of BRB cross sections available (Xie 2005).

The steel core of the BRB is designed to resist the entire axial load in the brace. This means that the restraining components of the bracing system are not used to carry any part of the axial load in the brace. This is achieved through the use bond breaker which will be explained in

more detail in the following sections. Also adequate gap is required at the ends of the yielding and transition zones to prevent the core from bearing on the restraining components during yielding (AISC 2006a; AISC 2006b).

The steel core resists the design earthquake axial load by both tensile and compressive yielding. This is possible because of the restraining system that prevents lateral buckling during compressive loading. The cyclic loading of the seismic force, as previously describe, is why the ability to have uniform yielding in both tension and compression over the length of the yielding zone is so appealing. The longer the yielding zone is, the less fatigue effects the brace capacity. This allows for very efficient force dissipation (Sabelli 2003).

This balanced performance in tension and compression is the main difference in BRBF and SCBF. The performance of the SCBF in tension is similar to the BRBF. However, the SCBF performance in compression is very different than a BRBF. Local or overall buckling which can occur in the SCBF under compression occurs at a stress level much lower than yielding. Overall buckling can cause unbalanced tension and compression stress during cyclic loading leading to a loss of stiffness and inelastic drift in the frame. Buckling degrades the strength and stiffness of the brace and leads to the formation of a plastic hinge in the brace. Ultimately overall compression buckling in the SCBF and the formation of the plastic hinge can lead to fatigue and eventual fracture of the brace in a non-ductile way (AISC 2006a; AISC 2006b; Sabelli and Lopez 2004).

The Restraining System

The restraining system of a BRBF consists of the components which resist the local and overall buckling of the steel core during compression loading. By restraining the buckling in compression, the brace gives balanced, stable and predictable hysteretic behavior by attaining both tensile and compressive yielding. However, this restraint needs to be provided to the steel core without resisting any part of the axial load in the brace. This is achieved typically through bond breakers and in some configurations through a gap between the steel core and the restraining system.

The lateral restraint against compression buckling of the steel core is most commonly provided by concrete mortar cast in a square or round steel HSS. This is the configuration that is the focus of this report. This sleeve around the steel core is where the BRB attains its name and function. The concrete mortar braces the entire length of the yielding and transition zones of the

brace while resisting no axial load. There is no shear transfer of axial load from the steel core to restraining system because there is no bond between the two components. The concrete mortar is prevented from bonding to the steel core by the use of bond breakers or debonding materials. These debonding materials can be epoxy resin, silicon resin, vinyl tapes, polyethylene film sheets, butyl rubber sheets, silicon rubber sheets and styrol foam or combinations of them. The exact details of some brace bond breakers are proprietary (Higgins and Newell 2004; Sabelli and Lopez 2004; Xie 2005).

The debonding material used must also provide for the transverse expansion of the steel core that occurs during compressive yielding due to Poisson's effect. If the debonding material is not suitable to accommodate the expansion, a separation between the steel core and the mortar to allow for the transverse expansion is required. The typical gap sizes vary from 0.025 to 0.15 inches depending on the configuration of the steel core used for the brace and the debonding mechanism (Xie 2005).

The restraining components are also required to have an adequate gap around the transition zone for the longitudinal shortening of the steel core during compressive yielding. The compressive yielding results in a shortened length of the yielding zone and therefore the movement of the transition zones towards each other. To prevent bearing on the restraining components like the encasing concrete, a gap is provided as can be seen in Figure 3-3 (Xie 2005).

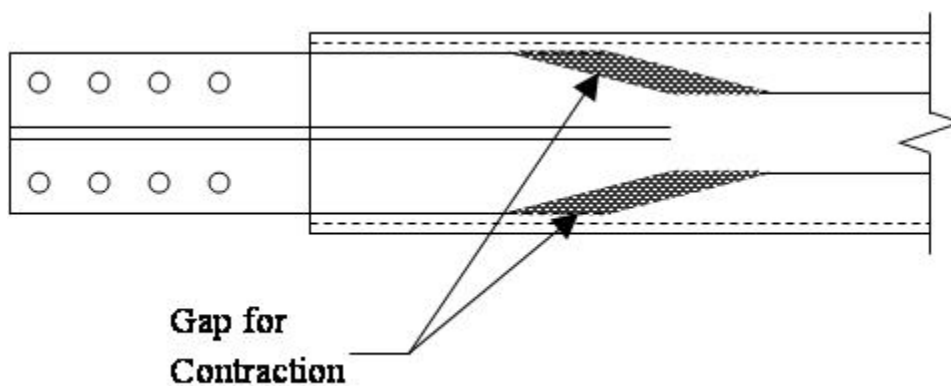


Figure 3-3. Gap to prevent bearing on buckling-restraining sleeve.

The amount of lateral displacement the frame is required to undergo without failure is 2.0 times the design story drift. This displacement along with the capacity of the brace is used by the BRB manufacturer to determine the gap size required for each brace in a frame. This also means the restraining component is required to provide lateral support to the steel core and to prevent local and overall buckling for the required lateral displacements (AISC 2006b; Sabelli et al. 2005).

The lack of buckling-restraining components in a SCBF and therefore the buckling of the brace element prior to yielding is the difference between SCBFs and BRBFs. Buckling of the brace increases the surrounding member sizes in inverted-V or chevron brace configuration which are shown in this report. Also the effective length of a SCBF brace is considered its total length depending on the end conditions while the BRB effective length can be considered zero (AISC 2006a; AISC 2006b).

Brace Connection Options

The connection between the brace and the other frame members is typically a bolted connection. True pinned or welded connections are also used but less widely. The ease of construction of a bolted connection is very beneficial. To help with construction tolerances the bolt holes may be oversized. However due to the cyclic load being applied, slip critical connections are required. After a design level or higher earthquake, brace replacement may be necessary and the bolted connection allows for replacement without interfering with other members. The gusset plates may also need to be replaced, but beams and columns in the frame should not (AISC 2006a; Uang and Nakashima 2004).

True pinned connections do not transfer shear or moment from the frame to the brace. This allows the brace to act as an idealized two force member. The shear and moment that the frame would transfer to the brace in bolted or welded connections is due to drift of the frame. Figure 3-4 shows a true pinned connection.



Figure 3-4. True pinned connection [courtesy of Star Seismic, LLC. <<http://www.starseismic.net/powercat.php>>].

The collar at the end of the brace shown in Figure 3-4 covers and stabilizes the connection zone. The collar can also eliminate the need to add stiffeners to gusset plates for local buckling. Since the connection is only through the single pin, the connection and gusset length is greatly reduced. The shorter connection also increases the available length of the yielding zone which reduces the axial strain in the core. The braces using pinned connections can be easily removed and replaced after a design level or higher earthquake. The ease of replacement decreases the time it takes a structure to become ready for occupancy after a design level or higher earthquake. The construction tolerances of a true pinned connection, however, can be very stringent and may create difficulty placing the bracing members during erection (Uang and Nakashima 2004).

Welded connections for BRB are not widely used. The ease of construction of the bolted or pinned connection makes using a welded connection uneconomical. Using a welded connection may actually increase the cost of brace replacement.

Beam-Column Connections

The beam to column connections within the BRBF are allowed to be designed as moment resisting or non-moment resisting. The non-moment resisting connection is assumed to be

pinned as in an idealized braced frame. The beams and columns of the pinned connected frame would therefore be exposed to less moment. The frame would have larger story drifts and less ductility compared to a frame with moment resisting connections.

The BRBF with moment resisting connections has a larger response modification coefficient due to greater ductility. Also the deflection amplification factor is smaller due to the decreased story drift but they require a larger system overstrength factor. The moment resisting connection would act as a re-centering force to reduce residual drift of the frame.

Qualification of a BRB

Buckling-restrained braces are required to be tested under cyclic loading representative of the loading applied when exposed to an earthquake. The braces are required to meet two test setups. The two tests are of the individual BRB and the BRB subassemblage. The BRB is the brace as it will be used in the structure. The BRB subassemblage is the combination of the brace and its connections that will be used in the structure. The testing is typically performed by the brace manufacturer. The qualifying tests of the individual BRBs are performed for the purpose of proving that the BRB satisfies the requirements for strength and plastic deformation required by the AISC Seismic Provisions. The reason for the intensive testing of the BRB is the lack of data on the performance of BRBF during actual earthquakes (AISC 2006a).

The subassemblage test requirements in the Seismic Design Manual (AISC 2006a) allow manufacturers to build a history of tested subassemblages based on the different parameters of the testing requirements. This history can eventually build to the point where further testing of the subassemblage would only be required for special cases. The testing procedures can be found in Appendix T of AISC 341-05 (AISC 2006a).

Story Drift of BRBF

The method used by the braces to absorb the seismic energy is through displacement. These displacements are a result of the low post-yield stiffness of the steel core. Once the braces reach their yielding stress, the brace continues to carry load, but at large strain in the steel core. Therefore the story drift of BRBF are prone to be larger than the SCBF and story drift may be the controlling factor in design rather than strength (Kiggins and Uang 2006).

Also the BRBF is susceptible to large residual story drifts after a design or larger earthquake because of the yielding of the steel core. The frames displace by yielding during an

earthquake, and then when the earthquake subsides, part of the displacement will be permanent since components will be stressed beyond their elastic limit. According to Sabelli et al. (2003) the residual story drift could be on average 40% to 60% of the maximum drift of the frame. The study also reported that these residual drifts were similar to those of moment resisting frames.

The large residual story drifts for a BRBF are due to a lack of a re-centering force to counteract these permanent displacements. If a BRBF is coupled with another system that can provide a re-centering force, the residual drifts of the frame can be greatly reduced. Kiggins and Uang (2006) suggested using BRBF as part of a dual system to reduce the residual drift in the frame. In their report moment frames were added in the building to provide the re-centering force. Their study found that using BRBF in a dual system reduced the residual drift by approximately 50%.

How a BRBF Dissipates or Absorbs the Seismic Energy vs. SCBF

The BRBF absorbs the seismic energy imposed on the structure from an earthquake through yielding in the braces. The yielding is in both tension and compression which results in a balanced system for the cyclic load of the earthquake. The energy is absorbed by the brace displacement during yielding. The displacement occurs while the brace continues to be able to withstand a constant level of force. By absorbing the energy through yielding, a high level of ductility is achieved (AISC 2006a).

The SCBF dissipates the seismic energy through buckling of the brace and the formation of a plastic hinge. The energy dissipated through the plastic hinge requires the brace to be designed and detailed for the high concentration of flexural strains that result from the hinge. The ductility gained through the brace buckling is significantly lower than the ductility of the BRBF brace yielding (AISC 2006a).

Figure 3-5 shows a typical hysteretic curve for both a BRBF and SCBF.

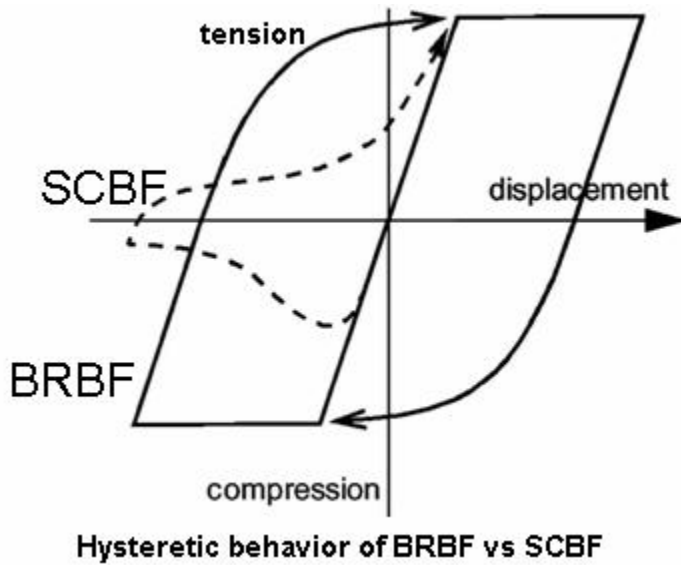


Figure 3-5. Typical ideal hysteretic graph of a BRBF and SCBF (AISC 2006a).

For both systems the tension portion of the graph is the same once yielding has been reached. For the compression side of the graph, the BRBF displays the same yielding plateau as in tension. For the SCBF in compression the brace never reaches the yielding stress. Instead the SCB follows the dotted line under compression which represents the buckling of the brace. The area contained within a hysteretic curve represents the energy dissipated by the system for one cycle of loading. For the BRBF the area under the curve is much larger than the SCBF due to the buckling of the compression brace. Therefore this larger area directly represents a large ductility for the BRBF compared to the SCBF (AISC 2006a; Hanson and Soong 2004).

CHAPTER 4 - BRBF Design Process

The design requirements according to AISC 341 (AISC 2006a) for a BRBF are presented in this chapter. ASCE 7-05 is used as the governing code for the calculation of the loads applied to the building and frame. The AISC Seismic Design Manual (AISC, 2006b) is used as a reference for the design of the BRBF with the AISC 341 Seismic Provisions (AISC, 2006a) being the governing document of the design process. Load and Resistance Factored Design (LRFD) is the design method used in this report.

After the building loads are calculated and distributed the member forces of the braced frame can then be determined. The load combinations required to be used in the design of the braced frame members are from ASCE 7 Sections 2.3, 2.4, 12.4.2.3, and 12.4.3.2. Of these load combinations, usually only a select few are required for the design of the braced frame. These load combinations are divided into three sets, each specific to a step in the design process. In the first step, LRFD load combinations 5 and 7 in Section 12.4.2.3 are used to size the brace cross section area. In the second step, LRFD load combinations 5 and 7 in Section 12.4.3.2 are used to size the columns and beams. Finally, load combinations 5, 6, and 8 in Section 12.4.2.3 are used to check drift of the frame. Once the members of the braced frame and the loading applied to the frame are modeled, the frame design process can begin.

The testing procedures of AISC Seismic Provisions Section 16.2c need to be met by the brace selected in the final design. These requirements are typically preformed by the manufacturer of the brace. For this reason it is necessary to contact the manufacturer once the brace is sized to determine if it is within their prequalified brace sizes.

Brace Design

In the first step of the BRBF design process, the cross sectional area of the steel core yielding segment is determined in accordance with the AISC 341 (AISC 2006a). A BRB manufacturer is contacted to determine the remaining dimensions and details of the steel core and the buckling restraining system. These components determined by the BRB manufacturer are required to meet the testing requirements of the AISC 341. The manufacturer should have prequalified sizes of the brace yielding core that meet the testing requirements. It is often

economical to choose one of these prequalified brace configurations because further testing is not required.

The steel core is sized according to Equation 16-1 in Section 16.2a of the AISC 341. This equation for LRFD is based on the limit states of tensile and compressive yielding:

$$\phi P_{ysc} = \phi F_{ysc} A_{sc} \quad (\text{Equation 4-1})$$

Where

- ΦP_{ysc} = brace design axial strength, kips
- F_{ysc} = yield stress of the steel core, ksi
- A_{sc} = net area of steel core, in²

The load combination used for calculating the brace area would be the load combination from ASCE 7 that results in the largest axial force in the brace without the amplification of the seismic load effects by an overstrength factor. These load combinations are normally ASCE 7 Equations 5 and 7 in Section 12.4.2.3 (ASCE 2005).

Once the brace steel cores are sized and selected, the adjusted brace strength can be calculated according to AISC Seismic Provisions Section 16.2d. The adjusted brace strength is the maximum expected force that could be developed by the brace. This force is then used to determine the size of the beams, columns, and all of the connections of the braced frame system. The adjusted brace force for compression is:

$$\beta \omega R_y P_{ysc} \quad (\text{Equation 4-2})$$

Where

- β = compression strength adjustment factor
- ω = strain hardening adjustment factor
- R_y = expected yield stress factor
- P_{ysc} = brace axial strength, kips

And in tension it is:

$$\omega R_y P_{ysc} \quad \text{(Equation 4-3)}$$

The factors included in the adjusted brace force equations are to account for any possible overstrength of the brace. The compression strength adjustment factor, β , is calculated from the required testing of the brace material used in the frame. β is the ratio of maximum compressive force to maximum tensile force as measured in qualification testing. The value of β shall be the largest ratio determined from the two tests required for qualification, and shall not be taken as less than 1.0. The strain hardening adjustment factor, ω , is also calculated from qualification testing. ω is the ratio of maximum tension force to the specified minimum yield stress of the test specimen and considers the effects of the cyclic loading of the steel. R_y is the ratio of expected yield stress to the specified minimum yield stress of the steel core material. This ratio can be found in Table I-6-1 of AISC 341. The value of R_y can be taken as 1.0 if the yield stress used in sizing the brace cross section area is taken from a coupon test of the core material actually used (AISC 2006b; Sabelli et al. 2005).

The adjusted brace strength should be multiplied by fabrication tolerances with the tolerance range to be provided by the manufacturer. In most cases the fabrication tolerances for the steel are negligible.

Column Design

The next step in the design of the BRBF is the design of the column. The design of columns according to AISC 341 is required to meet all the qualifications of Section 8 including column strength checks and seismic compactness. The strength check of the column is found in Section 8.3. First the column size is determined based on strength and the ratio of required strength of the column to the nominal axial strength of the column is evaluated. If the ratio is less than or equal to 0.4, the column size selected is the final size. The ratio is presented in Equation 4-4.

$$\frac{P_u}{\phi_c P_n} \quad \text{(Equation 4-4)}$$

Where

P_u = required axial compressive strength, kips

$\Phi_c P_n$ = design axial compressive strength, kips

If the ratio in Equation 4-4 is greater than 0.4, according to Section 8.3(1) the column design must include the amplified seismic load effect. The required axial strength of the column when including the amplified seismic load effect is considered without any applied moment. The amplified seismic load effect is determined from the appropriate load combinations. These are the ASCE 7 basic load combinations Equations 5 and 7 from Section 12.4.3.2 which include seismic effects and are given in Equations 2-11 & 2-12. According to Section 16.5b, the seismic effects must be modified by the adjusted brace strengths. This modification requires the calculation of a resultant overstrength factor from the adjusted brace strength, which is referred to as the brace overstrength factor.

The load combination used in the analysis to find the member forces should include this brace overstrength factor. The Equations 4-2 & 4-3 for the adjusted brace strength in tension and compression are combined with Equation 4-1 to determine the brace overstrength factor:

$$\Omega = \frac{\beta \omega R_y F_y A_{sc}}{P_u} \quad \text{(Equation 4-5)}$$

Where

Ω = brace overstrength factor

This can then be altered into the following equation for the brace overstrength factor:

$$\Omega = \frac{\beta \omega R_y}{\phi} \quad \text{(Equation 4-6)}$$

The brace overstrength factor calculated from Equation 4-6 is used in the ASCE 7 seismic load combinations of AISC 341 Section 12.4.3.2 as the overstrength factor applied to the horizontal seismic load effect. The column is designed according to these load combinations.

The requirements of Section 8.2b for a column to be seismically compact also apply and need to be checked before a final selection of a column size can be made. The requirements of the members of the braced frame to be seismically compact are because of the inelastic rotations that may occur (AISC 2006a; ASCE 2005).

Beam Design

The remaining members of the braced frame to be determined are the beams. The beams of a BRBF are required to meet the AISC 341 Section 8.2b, 16.4, and 16.5. Section 8.2b focuses on seismic compactness requirements, while Sections 16.4 and 16.5 are strength requirements for the beam. The requirements for the beam to be seismically compact are the same as for the columns. The strength requirements for the beams are the same as the columns with additional requirements based on the brace configuration (AISC 2006a).

According to Section 16.4, if V-type or inverted-V-type braced frames are used, such as is used in Chapter 5, the required strength is determined from all gravity loading in the appropriate load combination on the frame without considering any support from the braces intersection them. This is because the beam should have sufficient strength to support the entire load on the beam for the full span to remain intact if the braces were to fail. The seismic load effect in load combinations should be from the adjusted brace strength in tension and compression resolved into vertical and horizontal components. Figures 4-1 shows schematics of how the vertical and horizontal components are determined.

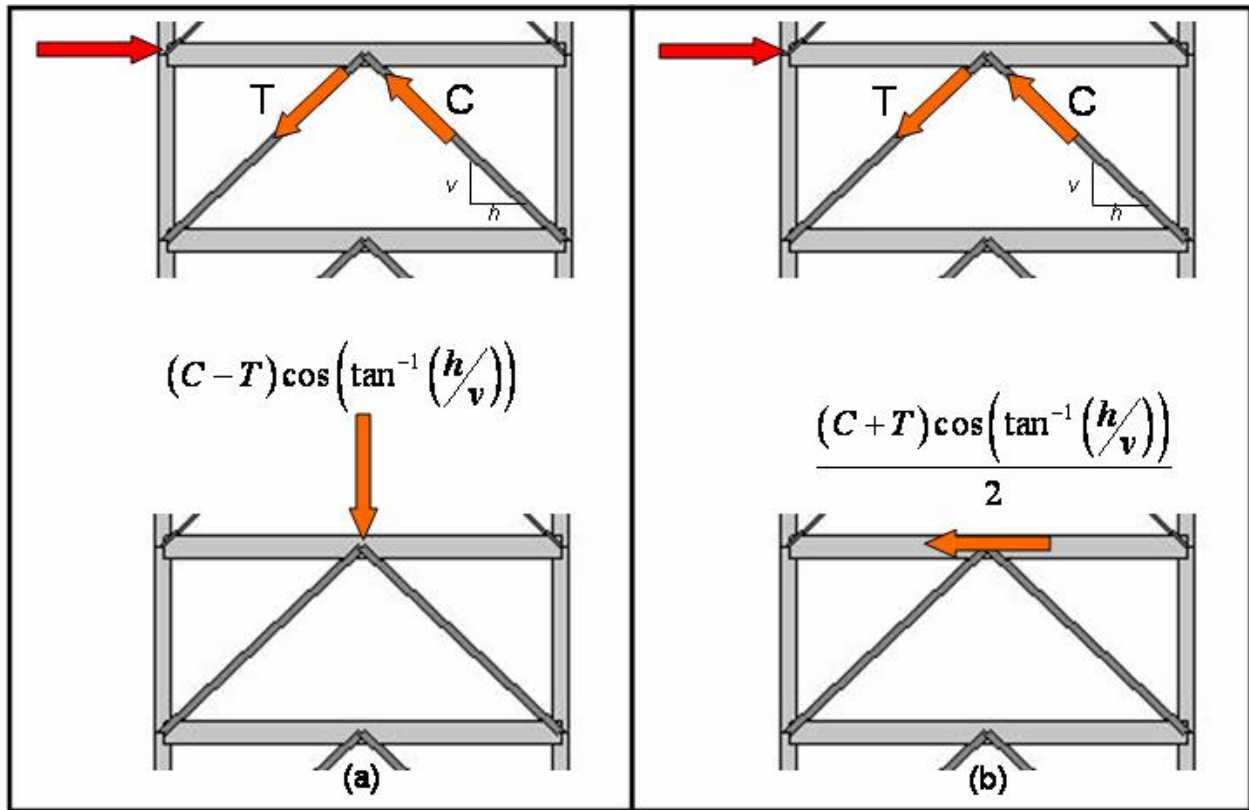


Figure 4-1. Seismic effects on beam from adjusted brace strengths.

The vertical unbalanced force depicted in Figure 4-1(a) is in the downward direction assuming the result is negative. However the calculation could be positive, which would then act upward. This will happen when the compression adjusted brace force is larger than the tension force. Since the unbalanced load acts upward, the moment created by it counteract the gravity load moments. Because it is conservative to do so, the upward vertical effect of the seismic force on the beam can be neglected. The horizontal component determined in Figure 4-1(b) is divided by 2 because the axial force is assumed to be shared equally by the level above and below the brace.

Drift Checks

Once the framing members are designed, the story drift of the frame should be checked. The drift check in Chapter 5 uses ASCE 7 load combinations 5, 6, and 8 from Section 12.4.2.3. The frame is typically modeled using computer software for structural analysis with the

appropriate load combinations from which the joint displacements at each level can be determined. These joint displacements are then used in ASCE 7 Equation 12.8-15 (Equation 4-7) to determine the story drifts.

$$\delta_x = \frac{C_d \delta_{xe}}{I} \quad (\text{Equation 4-7})$$

Where

- δ_x = deflection at level x, in
- C_d = deflection amplification factor
- δ_{xe} = deflection determined by elastic analysis
- I = importance factor

The joint displacements from the computer analysis model are deflections determined using elastic analysis, δ_{xe} . The deflection amplification factor, C_d , is found in ASCE 7 Table 12.2-1 and is based on the lateral system being used. The importance factor, I , is found in ASCE 7 Table 11.5-1. Once the story drifts are calculated, they are compared to the allowable story drifts to determine if the frame is adequate. The allowable story drifts, Δ_a , based on occupancy category are given in ASCE 7 Table 12.12-1. For a BRBF of occupancy category IV the allowable story drift is given in Equation 4-8.

$$\Delta = 0.010h_{sx} \quad (\text{Equation 4-8})$$

Where

- Δ = allowable story drift, in
- h_{sx} = story height below level x, in

Connection Forces

After the member selection is finalized, the connections can then be determined. This report focuses on member selection for a BRBF compared to a SCBF, therefore connections specific to each type are not determined; however the connection forces are calculated and used in the comparison.

Brace connection forces are determined from the adjusted brace strength increased by a factor of 1.1 according to AISC 341 Section 16.3. The 1.1 factor is used to account for the possibility that the braces may exceed the range of deformations that the β and ω factors were determined from.

The connection force that is thus determined will prevent the connection from yielding due to forces and deformation from yielding of the steel core. These connection forces are used to design brace gusset plates and determine the length of the connection zone of the brace. These forces are also used in the design of the connections of beams and columns of the BRBF.

The required strength specified by the AISC Seismic Provisions is for tension and compression loads only; there is no required strength for flexure in the connection. This is permitted because the connection used in the structure is required to be shown by tests that it will accommodate the rotations and deformations that correspond to brace deformations at a MCE level.

CHAPTER 5 - Parametric Study

The parametric study of this report was performed to compare the design of a BRBF with a SCBF. The two frames were placed in identical buildings and modeled using RISA-3D 7.0. The seismic forces on the buildings were calculated from ASCE 7 using the ELFP. The purpose of the study was to compare the two frames based on member sizes, member forces, and design requirements. The goal of this study was to determine if for high seismic areas the BRBF is a more efficient frame than the SCBF. Another goal of the study was to show that the design requirements of the BRBF, even though widely unused, are simple and result in a more economically designed frame.

The following sections describe the process of selecting the model building that was designed, the calculations and computer modeling of the buildings, determination of member sizes for each frame, and the results and comparisons of the two systems. Detailed calculations for load determination and frame design in the study are provided in Appendix A through D.

Parameters of the Model Building

The model building used for this parametric study was chosen such that the effects of seismic activity would develop stresses beyond the elastic range for parts of the lateral force resisting system during a design earthquake. To ensure a high seismic force, the building location was selected near Memphis, Tennessee. MCE ground motions of 2.0 and 0.6 for spectral response accelerations of 0.2 and 1.0 seconds were determined for this location. This higher area of seismic activity makes the use of a ductile lateral system, such as BRBF feasible. The building type selected was a hospital which required more amplification of the seismic force due to the importance factor.

The building was selected to be 4 stories tall, 3 floors above grade plus the roof. The plan dimensions were 78 feet square with 3 bays of 26 feet in each direction. The floor to floor height was 12 feet for all 4 levels. The building was chosen to be a steel framed structure with the floors composed of composite steel beams with 2 inch composite deck and 3.25 inch lightweight concrete for a total slab thickness of 5.25 inches. The roof was chosen to have non-composite deck with no concrete fill on open web joists. The exterior walls were selected as

brick veneer with steel stud walls. For the interior partition walls, 10 pounds per square foot was applied as a live load to each floor level.

The lateral force resisting system of the building was chosen to be braced frame of the inverted-V, or chevron style. Two bays of braced frames were placed on each exterior wall to provide a uniform seismic resisting system of 4 braced frames in each orthogonal direction. This balance in resisting system allows for the building to be classified as a regular structure which minimizes the effects of torsion. Figure 5-1 shows one of the buildings braced frames with each level labeled.

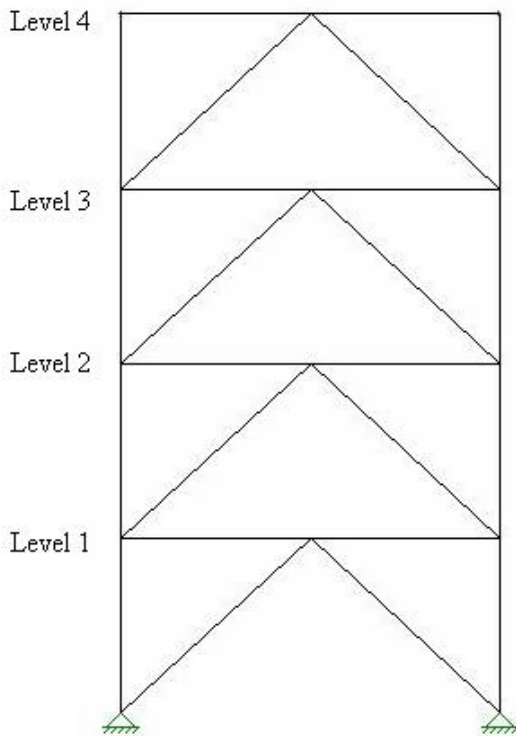


Figure 5-1. Braced frame elevation.

Building Load Calculations

The different loads applied to the model building were determined according to the requirements of the IBC 2006 (ICC 2006) and ASCE 7. The live loads were calculated from the

IBC 2006 with a floor live load of 60 psf and a roof live load of 20 psf. The dead loads were calculated from the ASCE 7 and are 52 psf on the floor and 15 psf on the roof. The snow load was also calculated from the ASCE 7 and is 25 psf, which governs over the roof live load. The building wind load was not calculated because the focus of this report is the seismic design. The seismic base shear and resulting story forces were calculated from the ASCE 7 ELFP. The response modification coefficient, R , for the BRBF was 7 while for the SCBF it was 6. For both frames the mapped MCE spectral response acceleration parameters of S_S and S_I were 2.0 and 0.6, respectively. The site class was chosen as D because ASCE 7 Section 11.4.2 requires site class D to be used if soil properties are not known. The building base shear was calculated from the seismic building weight of 1670 kips (kilo-pounds) and a seismic response coefficient of 0.285g for the BRBF and 0.3325g for the SCBF buildings. This is where the calculations are no longer the same for the two LFRS. For the remainder of this chapter, “building” will refer to the conceptualized buildings being designed, and “model” will refer to the RISA-3D 7.0 computer model, with both the building and model being distinguished by BRBF and SCBF.

The base shear was calculated to be 475 kips for the BRBF building and 555 kips for the SCBF building. The vertical distribution of the seismic base shear is shown in Table 5-1.

Table 5-1. Vertical distribution of seismic base shear to each level and frame.

Level	BRBF		SCBF	
	Horizontal Force per Level (kips)	Force per Braced Frame (kips)	Horizontal Force per Level (kips)	Force per Braced Frame (kips)
4	92	23	107	26.8
3	192	52.8	224	62
2	127	35	149	41
1	64	17.6	75	20.6

As can be seen, horizontal seismic force increases as building height increases for both buildings up to the roof level. The significant decrease in the force at the roof is due to the decrease in building weight at the level.

The force distribution to the individual braced frames was calculated by accounting for the rigidities of the frames, which were identical for each type, and the center of mass of the floor. Since a detailed floor plan was not used to find the actual center of mass, but was assumed to be at the center of the floor plan due to uniformity of the mass. ASCE 7 Section 12.8.4.2 requires that accidental torsion be considered by the center of mass being offset by 5 percent of the building length in each orthogonal direction. The moment that this offset creates must be included in the design. By including this torsional effect, the horizontal force was distributed from the diaphragm at each level to the braced frame at each level. The maximum brace forces are presented in Table 5-1.

Once the seismic forces were distributed to the braced frames at each level, models were created of the braced frames. All of the building loads that are applied to the members of the braced frame in the buildings were included in the models. These models were used to analyze the force distribution to the separate members of the braced frames. From these member forces, the braced frame members were sized and selected according to the AISC 341.

Results

For both frames the members were selected based on the economics of the section with consideration for practical construction. Therefore members were chosen from similar families of W-sections for the beams and columns, and square HSS for the SCBF braces. The results of the frame models are presented with the forces on a member and then the final member selected. For the member design calculations see Appendix C & D. The members selected for the two frame types are briefly discussed with further detail presented in the comparison section of this report.

Design of Braces

The braces for the BRBF were sized from the axial forces determined from the structural analysis. Table 5-2 shows the axial force determined in the brace member at each level along with the brace cross sectional area required.

Table 5-2. BRBF axial forces and member cross sectional area.

BRBF				
Level	Seismic Force (kips)	Compression in Brace (kips)	Tension in Brace (kips)	Area of Brace (in ²)
4	23	21	14	0.63
3	52.8	76	44	2.38
2	35	99	67	3.08
1	17.6	113	79	3.49

As was described in Chapter 4, the BRBF brace cross sectional area was calculated assuming the brace steel core carries the entire axial load for the limit state of yielding. The higher axial force in compression controlled the design axial strength of the braces at each level. The force was higher in compression because of the gravity loads transferred to the brace.

The SCBF axial force applied at each level and the force generated in each brace along with the final selection of the square HSS used for the brace is shown in Table 5-3.

Table 5-3. SCBF axial forces and members used.

SCBF				
Level	Seismic Force (kips)	Compression in Brace (kips)	Tension in Brace (kips)	Square HSS used
4	26.8	24.5	18	5½ x 5½ x 5/16
3	62	84	52	5½ x 5½ x 5/16
2	42	112	81	5½ x 5½ x 5/16
1	20.6	123.5	93.5	6 x 6 x 3/8

Table 5-3 shows a similar force distribution by level to the braces of the SCBF model as was shown for the BRBF model in Table 5-2. This similarity in proportional force distribution is expected since at this point in the design process there is no difference between the BRBF and SCBF except the applied seismic force. However, in the selection of the brace to be used, which

in the BRBF building is a steel core area and in the SCBF building is a square HSS, there are differing requirements for member selection per the AISC 341.

The design of a SCBF brace as stated in Chapter 4, requires the brace to resist the entire axial load to meet the requirements of a minimum slenderness ratio and to be seismically compact. The latter two of these three requirements typically govern the design of a SCBF brace. For the design of the braces in level 1 of the SCBF, the design member selection was driven by the seismically compact requirements. At level 2, the member selection was controlled by seismic compactness criteria, and the minimum required slenderness ratio. Level 3 and 4 member selection was governed by the minimum slenderness ratio.

The uniformity of bracing members required by the SCBF will significantly impact the beam selection and connection requirements for the same frame. Similarly the decrease of brace steel core area as the levels increase of the BRBF will likewise impact the beam selections and connection requirements of the BRBF.

Brace Connection Forces

The required strength of the connections of a BRBF is determined from the adjusted brace strength at each level. The required strength is 1.1 times the adjusted brace strength. For a BRBF, this required strength is applied to both the bracing connections in tension and compression, and the beam to column connection that are part of the braced frame. Table 5-4 shows the required strengths of the brace connections in tension and compression of the BRBF.

Table 5-4. BRBF summary of required connection strength.

BRBF			
Level	Area of Brace (in ²)	Required Strength in Compression (kips)	Required Strength in Tension (kips)
4	0.63	48	44
3	2.38	181.5	165
2	3.08	235	214.5
1	3.49	267	242

The required strengths in Table 5-4 are used to calculate the connections for the braces and the associated beam to column connections of the frame. The required connection size, number of bolts or welding requirements, of the frame will decrease dramatically for the upper levels. In contrast to the SCBF required connection forces, the BRBF has no requirements for minimum strength in flexure. However the BRBF is required to have connections that are able to accommodate the rotations and deformations that coincide with the design drifts, which is proven through testing.

The required strengths of the SCBF bracing connections and beam to column connections of the frame are shown in Table 5-5.

Table 5-5. SCBF summary of required connection strengths.

SCBF				
Level	Square HSS used	Required Strength in Compression (kips)	Required Strength in Tension (kips)	Required Strength in Flexure (kip-ft)
4	$5\frac{1}{2} \times 5\frac{1}{2} \times \frac{5}{16}$	287	377	49
3	$5\frac{1}{2} \times 5\frac{1}{2} \times \frac{5}{16}$	287	377	49
2	$5\frac{1}{2} \times 5\frac{1}{2} \times \frac{5}{16}$	287	377	49
1	$6 \times 6 \times \frac{3}{8}$	392	488	84

The connections of the braces and the associated beams to columns are to be designed for the required strengths provided in Table 5-5. The compressive connection force is calculated from AISC 341 Section 13.3c and is based on the critical buckling stress of the brace. The tensile force is calculated from the AISC 341 Section 13.3a which depends on the expected yield strength of the member. The flexural connection force is calculated from AISC 341 Section 13.3b and is based on the expected flexural strength of the brace member. Section 13.3b also states that the brace connections need not be designed for the flexural strength if the connection is designed for the required tensile strength and can accommodate any inelastic rotations associated with brace post buckling deformations (AISC 2006b; AISC 2006a).

Since the member used as the brace at levels 2, 3, and 4 are the same, the required connection forces for these levels are the same. This does not provide a balance between the seismic force applied to the level from Table 5-3 to the connection requirements. This is due to

the brace members being sized for stability requirements and not for strength. Since the forces in the top three levels are the same, the number of bolts or welding requirements in the SCBF connections will not decrease for the upper levels like the BRBF.

Design of Columns

The column design processes for the BRBF and SCBF have essentially the same requirements. The columns of both frame types are to comply with the requirements of Section 8 of the AISC 341. Section 8 specifies member compactness, required strength, splice strength, and strength at the base of the column. For this report, only the compactness and required strength criteria were used. This is because the columns are designed as one continuous member without any splices, and the foundation of the frames was not investigated so the strength at the base of the column was not determined.

Section 8.2 stipulates the requirements that members be seismically compact for local buckling of the flanges and webs. These requirements are given in AISC 341 Table I-8-1. In the design of the frames for this report, local buckling requirements governed the member selection over the required strength.

The required strength stipulated in Section 8.3 requires the column to be sized for the axial loads applied including amplified seismic load effects without including any applied moment. The amplified seismic load for the BRBF is calculated from the adjusted brace strength while for the SCBF the overstrength factor of ASCE 7 Table 12.2-1 is used. Once the column is selected, the member size is evaluated using Equation 5-1 to determine if it can be reduced. If the ratio in Equation 5-1 is less than 0.4, the member size can be reduced.

$$\frac{P_u}{\phi_c P_n} > 0.4 \quad \text{(Equation 5-1)}$$

P_u is calculated without applying the amplified seismic load. Also the required axial compressive and tensile strength does not need to exceed the maximum force that can be transferred to the system.

The column size selected for both the BRBF and SCBF was the same. A summary of the axial forces and the member used is provided in Table 5-6.

Table 5-6. BRBF and SCBF column member selection and axial forces.

BRBF			SCBF		
Column Used	Compression (kips)	Tension (kips)	Column Used	Compression (kips)	Tension (kips)
W12x45	284.4	181.3	W12x45	307.1	201.9

The similarity of the forces used to size the columns is because the columns are continuous from the ground level to the fourth level. So the columns were sized for the most extreme case, which was the first level.

Design of Beams

The beams of the BRBF are required to meet the same seismically compact requirements of Section 8.2 of the AISC 341 as the columns. Since chevron style braced frames were used, the beams also have to meet the requirements of Section 16.4. Section 16.4 requires beams that are intersected by braces must include the effects of the applicable load combination assuming the braces provide no support for gravity loads on the beam to. Also the vertical and horizontal effects of the earthquake load in load combinations need to be resolved from the adjusted brace strength in tension and compression as discussed in Chapter 4.

For the BRBF, the applicable load combinations used were ASCE 7 Section 2.3.2 combination 2 and 5. In combination 5, the earthquake vertical component was taken as the unbalanced vertical load from the adjusted brace strength and the horizontal component was half of the combined horizontal effects of the adjusted brace strength of the compression and tension brace. This style of brace configuration results is a vertical component that acts in the upward direction as was discussed in Chapter 4. Therefore, in the design of the BRBF beam, the vertical seismic effects were neglected. The horizontal component was taken as half because it is assumed that the load is shared equally by the beam above the brace and the column below. The resulting flexure and axial compressive loads on the beams and the members selected are presented in Table 5-7.

Table 5-7. BRBF beam selection and forces.

BRBF						
Level	Beam Used	Flexure (kip-ft)				LRFD Load Combo #5 Compression (kips)
		Service Dead Load	Service Live Load	Service Seismic Load	LRFD Load Combo #5	
4	W10x26	17	22	-18	31	31
3	W18x46	106	66	-66	160	116
2	W18x50	106	66	-84	160	150
1	W18x50	106	66	-101	160	170

The beam selections in Table 5-7 were governed by required strength not stability requirements. The ratio of axial compression to compressive strength was only about 50% for the combined forces unity check for each level.

The SCBF beam design procedure is the same as the BRBF excluding the requirement of the beam to meet the seismically compact requirements of Section 8.2. The design of the beam for gravity loads and the unbalanced brace capacities without the support of the brace is the same. The brace force used for calculating the unbalanced seismic force and axial compressive force is calculated differently than in a BRBF. According to AISC 341 Section 13.4(a), chevron braces in tension are assumed to have a maximum force of the yield strength of the member multiplied by the ratio of actual to assumed strength, R_y . For braces in compression, the assumed braced force is 0.3 times the nominal compressive strength which indicated the controlling limit state is overall buckling. The application of the brace forces to the beams is the same as described for the BRBF in Chapter 4. Table 5-8 shows the beams selected for each level along with the required flexural and compressive forces.

Table 5-8. SCBF beam selection and forces.

SCBF						
Level	Beam Used	Flexure (kip-ft)				LRFD Load Combo #5 Compression (kips)
		Service Dead Load	Service Live Load	Service Seismic Load	LRFD Load Combo #5	
4	W27x146	17	22	1490	1528	153
3	W27x161	106	66	1490	1665	153
2	W27x161	106	66	1490	1665	153
1	W27x217	106	66	1898	2264	200

Comparison

The results of the two models were compared based on building level, force levels, and individual members selected. The two frame systems were not compared based on economics or total structure weight. Economic comparisons were not made because the cost of a BRB varies widely and changes rapidly, while the total structure weight was deemed an unnecessary comparison in light of the member by member comparisons. The comparisons presented are the member forces and weights of the BRBF as a percentage of the corresponding element in the SCBF. The SCBF is assumed to be equal to 100%. The comparison is explained from these percentages with insight into where the differences are founded in design.

Brace forces

The required brace forces used to size the braces of the BRBF and SCBF are very similar. The difference is the forces are from the difference in the seismic force applied to the structure and the overstrength factor used. The difference in the seismic force is due to the difference in the response modification coefficients of the two systems. The overstrength factor for the BRBF is calculated from the adjusted brace force while the SCBF overstrength factor is system dependent. However, even though the forces applied to the braces are similar, the brace selection for the two frames is very different. This is because of the SCBF is required to have compact and non-slender braces. The selection of the braces for the SCBF is driven by local

buckling and slenderness requirements. The difference in brace forces and the brace cross section areas used are provided in Table 5-9.

Table 5-9. Comparison of brace forces and cross sectional area.

Level	BRBF% of SCBF		
	Compression	Tension	Brace Area
4	83%	78%	11%
3	90%	85%	41%
2	89%	82%	53%
1	91%	85%	46%

Table 5-9 compares the differences of Tables 5-2 and 5-3. The required compressive strength of the BRB is approximately 90% of the SCB. The required tensile strength of the BRB is between 78-85% that of the SCB. However the area of steel required for the brace of the BRBF is only 11-53% of the SCBF. This drastic difference in area of steel required when the forces are much more comparable is from the slenderness and seismically compactness requirements for braces in the SCBF. The braces for the SCBF were not selected by required strength, but rather by section properties. The SCBF level 1 brace was selected based on local buckling requirements. The brace at level 2 was selected by a combination of local buckling and slenderness ratio requirements. While the braces at levels 3 and 4 were selected by their slenderness ratio.

As will be seen in the following sections, the compact and slenderness requirements of the SCBF braces will affect the remainder of the member selections.

Brace Connection Forces

The brace connection forces of the BRBF are much lower than the SCBF. However the process used to determine the brace connection forces for the two frames is very similar. For the BRBF the connection forces are the adjusted brace strengths amplified by 10%. This represents the maximum expected strength of the brace in either tension or compression. For the SCBF the connection force in tension is the expected yield strength while in compression it is the required

buckling compressive stress amplified by R_y and 10%. These forces, like in the BRBF, represent the maximum expected strengths of the braces (AISC 2006a; AISC 2006b).

Table 5-10 shows a comparison of the connection forces of the BRBF and SCBF from Tables 5-4 and 5-5 respectively. Flexural forces of the SCBF are not included in the comparison because if the connection is designed to accommodate the rotation and deformation of the brace, the flexural forces need not be applied. This requirement is similar to the BRBF requirement for flexural forces.

Table 5-10. Comparison of brace connection forces.

Level	BRBF% of SCBF	
	Compression	Tension
4	17%	12%
3	63%	44%
2	82%	57%
1	68%	50%

The decrease in required compressive strength of the BRBF from the SCBF on the lower 3 levels is moderate. However, on level 4 the BRBF compressive strength is 17% that of the SCBF. This drastic difference, as previously stated, is from the comparatively large brace size used on the 4th level due to compact and slenderness requirements of the brace.

The decrease of required tensile strength from the SCBF to the BRBF is similar to the compressive strength. On the lower 3 levels, the BRBF is approximately 50% of the SCBF, while at the 4th level it is 12%. The large difference is due to the same reason as the compressive strength.

The significantly lower required connection strengths in the upper level of the BRBF will greatly reduce the size and therefore cost of the connections. The number of bolts or weld size and gusset plates for the connections will be smaller.

Column Sizes and Forces

There is very little difference in the column forces between the two frame types. This is because the design procedure for the columns of the two frames is the same. The only differences in forces applied to the columns come from the difference in the response modification coefficient and the overstrength factor. The member section chosen was the same for both frames. Table 5-11 gives the comparison the axial forces applied to the columns of the two frames.

Table 5-11. Comparison of column axial forces.

BRBF% of SCBF	
Compression	Tension
93%	90%

The forces compared in Table 5-11 are from Table 5-6 level 1 only. The first level was only compared because the columns in the frame will be continuous from the ground level to the 4th level. If the forces for the remaining 3 levels were to be compared, the BRBF would continue to require less and less strength compared to the SCBF.

Beam Sizes and Forces

The beams used for the SCBF are largely driven by the unbalanced vertical seismic load of the braces. The selection of the same brace for 3 of the levels results in a more uniform selection of beams sizes and weights than the BRBF. It should also be noted that the SCBF has a strong beam in relation to its column, where as the BRBF has a beam of comparable strength to its column.

The beams selected for the BRBF are significantly lighter than the SCBF, however not all the forces applied to the BRBF are less. Table 5-12 shows a comparison of the forces applied to the beams and the weight of the beam section selected.

Table 5-12. Comparison of beams.

Level	BRBF% of SCBF		
	Flexure	Compression	Beam Weight
4	2%	20%	18%
3	10%	76%	29%
2	10%	98%	31%
1	7%	85%	23%

The overwhelming difference in required flexural strengths of 2-10% is from the large unbalanced vertical seismic load applied from the chevron braces in the SCBF. In the BRBF, the unbalanced vertical seismic loads effect was in the upward direction and therefore neglected in design. The SCBF vertical seismic load effect is much larger than the BRBF and in the downward direction. This resulted in a significantly higher flexural requirement the beam.

The comparison of required compressive strength in the beams between the two frames presents a much different result. The BRBF beam has similar compressive force to the SCBF for levels 1, 2, and 3. The effect of the similarity of this force is not seen in the selection of the beams because the flexural force governed the member selection over consideration of the compressive force.

The SCBF beam selection does not present as stark of contrast as the flexural forces might indicate. The BRBF has a beam weight of close to 30% that of the SCBF for the first 3 levels with the 4th level being 18% of the SCBF. The larger difference for the 4th level supports the conclusion that the large brace cross section area in the SCBF has a significant impact on frame members selected.

CHAPTER 6 - Conclusions

The procedure of applying a seismic force to a structure was explained in detail. The determination of the maximum considered earthquake ground motion was also described. The simplified approach of equivalent lateral force procedure outlined in ASCE 7-05 was described in detail with a design example included in Appendix A through D. The mechanics and behavior of a buckling restrained braced frame (BRBF) was described in depth. This description was then compared to a special concentrically braced frame (SCBF). The design procedure differences of a BRBF compared to a SCBF were explained. The final member selections for both frame types were then compared on the basis of required strength, selection, and individual weight. The two frames were not compared based on economics or total structure weight. These comparisons were not made because the economics of a BRB varies widely and changes rapidly, while the total structure weight was deemed an unnecessary comparison in light of the member by member comparisons.

For the parametric study the determination of the seismic load on the frame for the BRBF and SCBF was through the ASCE 7-05 equivalent lateral force procedure with frame response modification coefficients being the only alteration. The building location was chosen as Memphis Tennessee to ensure a high seismic force. MCE ground motions of 2.0 and 0.6 for spectral response accelerations of 0.2 and 1.0 second periods were required for this location. The braced frames were modeled in RISA-3D 7.0 for analysis.

A BRBF was determined to have these advantages over a SCBF:

1. The BRBF has a greater ductility than the SCBF because of the BRB mechanics. The BRB achieves yielding in tension and compression because of the restraining components mitigating buckling of the brace. The SCB yielding in tension but buckles in compression which reduces the ductility of the frame
2. The required brace cross section area for the BRBF was directly determined from the required strength. In the SCBF the brace selection was governed by local and

overall buckling requirements. This resulted in the BRB being proportional to the seismic forces at each level while the SCB were the same for the top 3 levels.

3. The required connection strengths for the BRBF decreased vertically up the frame while the SCBF required connection strengths were more uniform. This would require fewer bolts or smaller weld sizes and smaller gusset plates for the BRBF than for the SCBF in the upper levels. Also the beam to column connection in the upper levels would be reduced for the BRBF which would lessen any requirements for stiffeners at the lower levels.
4. The beam sections for the BRBF were significantly smaller than the SCBF. This was due to the large vertical unbalanced seismic forces in the braces in the SCBF.

A BRBF was determined to be at a disadvantage compared to a SCBF in these ways:

1. The implementation of the BRBF as a LFRS in the United States is still growing and is not yet used extensively. Widespread use and acceptance of the BRB has not yet been attained. The lack of knowledge of the system and how it is constructed can hinder the decision to use BRBF in projects. The manufacturing of BRB is still quite proprietary and expensive.
2. The BRBF has not been subjected to a real seismic event of large enough magnitude in the United States to truly test the cyclic loading capability of the system.
3. The brace in a BRBF is required to be purchased as a unit from a manufacturer while the SCBF brace can be produced by any steel fabricator at a significantly lower cost.

The overall conclusion of this report is that the BRBF is the superior system for high seismic areas in structures of important occupancy categories for the following reasons:

1. The design process of the BRBF may be new, but it has less stringent requirements and provides a more efficient frame design. The more efficient design is attained by the members being designed close to their required strength

while in the SCBF the member strengths are often significantly higher than required strengths.

2. The required cross section areas for the braces of the BRBF were determined from strength calculations instead of stability requirements which governed the SCBF design. This allows the BRBF braces to be more proportionally sized at each level.
3. The required brace connection strength of the BRBF was approximately 30% of the SCBF brace in tension and 50% in compression.
4. The beam selection for the BRBF was determined from vertical gravity loads and the horizontal seismic effects of the braces which resulted in proportional beams at each level of the frame.
5. A project that utilizes BRBF will benefit in steel erection time and costs. Both connections and member sizes will generally be less for the BRBF helping to offset the additional cost of the BRB.

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Appendix A - Seismic Load Calculations for BRBF

All equation references are from ASCE 7-05 (ASCE 2005).

Seismic Base Shear

$$V = C_s W \quad (\text{Eqn 12.8-1})$$

$$C_s = \frac{S_{DS}}{R/I} \quad (\text{Eqn 12.8-2})$$

$$R = 7 \quad (\text{Tbl 12.2-1})$$

$$I = 1.5 \quad (\text{Tbl 11.5-1})$$

$$S_{DS} = \frac{2}{3} S_{ms} \quad (\text{Eqn 11.4-3})$$

$$S_{ms} = F_a S_s \Rightarrow (1.0)(2.0) = 2.0 \quad (\text{Eqn 11.4-1})$$

$$F_a = 1.0 \quad (\text{Tbl 11.4-1})$$

$$S_s = 2.0 \quad (\text{Tbl 22-7})$$

$$S_{DS} = \frac{2}{3} (2.0) = 1.33$$

$$C_s = \frac{1.33}{\frac{7}{1.5}} = 0.285$$

Check C_s Maximum and Minimum

$$T = C_t h_n^x = (0.02)(48)^{0.75} = 0.365 \quad (\text{Eqn 12.8-7})$$

$$C_t = 0.02 \quad (\text{Tbl 12.8-2})$$

$$x = 0.75 \quad (\text{Tbl 12.8-2})$$

$$h_n = 48 \text{ ft}$$

$$T_L = 12 \quad (\text{Fig 22-15})$$

$$T < T_L$$

$$C_{s \text{ max}} = \frac{S_{D1}}{T R/I} \quad (\text{Eqn 12.8-3})$$

$$S_{D1} = \frac{2}{3} S_{M1} \quad (\text{Eqn 11.4-4})$$

$$S_{M1} = F_v S_1 \Rightarrow (1.5)(0.6) = 0.9 \quad (\text{Eqn 11.4-2})$$

$$F_v = 1.5 \quad (\text{Tbl 11.4-2})$$

$$S_1 = 0.6 \quad (\text{Tbl 22-8})$$

$$S_{D1} = \frac{2}{3}(0.9) = 0.6$$

$$C_{s \max} = \frac{0.6}{0.365 \left(\frac{7}{1.5} \right)} = 0.352 > 0.285 \therefore \text{OK}$$

$$C_{s \min} = 0.01 < 0.285 \therefore \text{OK} \quad (\text{Eqn 12.8-5})$$

$$C_{s \min} = \frac{0.5 S_1}{R/I} = \frac{(0.5)(0.6)}{7/1.5} = 0.064 < 0.285 \therefore \text{OK} \quad (\text{Eqn 12.8-6})$$

$$W = 1670 \text{ kips} \quad (\text{Sec 12.7.2})$$

$$V = (0.285)(1670) = 475 \text{ kips}$$

Vertical Distribution of Base Shear

$$F_x = C_{vx} V \quad (\text{Eqn 12.8-11})$$

$$C_{vx} = \frac{w_x h_x k}{\sum w_i h_i k} \quad (\text{Eqn 12.8-12})$$

The vertical distribution is shown in Table 13.

Table 13. Vertical distribution of base shear in BRBF.

Level	w_x (kips)	h_x (ft)	$w_x h_x$ (k-ft)	C_{vx}	F_x (kips)
4	180	48	8640	0.194	92
3	500	36	18000	0.403	192
2	500	24	12000	0.269	127
1	500	12	6000	0.134	64
$w_i h_i$ (k-ft) =			<u>44640</u>	$V =$	<u>475</u>

Horizontal Distribution of Base Shear Force

Level 4 – Flexible diaphragm

$$F_x = 92 \text{ kips}$$

$$E = F_x/4 = 92/4 = 23 \text{ kips per frame}$$

Level 3 – Rigid diaphragm

$$F_x = 192 \text{ kips}$$

Center of Rigidity (CR) is at geometric center of building due to symmetry

Center of Mass (CM) is assumed at geometric center of building

CM must include accidental torsion of 5% building length each direction (Sec 12.8.4.2)

$$0.05L = 0.05(78) = 3.9 \text{ ft}$$

$$CM = 78/2 \pm 3.9 = 47.4 \text{ or } 39.6$$

$E = V_D + V_T =$ Direct shear + Torsional shear

$$V_D = F_x/4 = 192/4 = 48 \text{ kips}$$

$$2 * V_T = M_T/78 \text{ ft} = F_x e / 78 \text{ ft} = (192)(3.9)/78 = 9.6 \text{ kips}$$

$$E = 48 + 9.6/2 = 52.8 \text{ kips per frame}$$

Level 2 – Rigid diaphragm

$$F_x = 127 \text{ kips}$$

Center of Rigidity (CR) is at geometric center of building due to symmetry

Center of Mass (CM) is assumed at geometric center of building

CM must include accidental torsion of 5% building length each direction (Sec 12.8.4.2)

$$0.05L = 0.05(78) = 3.9 \text{ ft}$$

$$CM = 78/2 \pm 3.9 = 47.4 \text{ or } 39.6$$

$E = V_D + V_T =$ Direct shear + Torsional shear

$$V_D = F_x/4 = 127/4 = 32 \text{ kips}$$

$$2 * V_T = M_T / 78 \text{ ft} = F_x e / 78 \text{ ft} = (127)(3.9) / 78 = 6.3 \text{ kips}$$

$$E = 32 + 6.3 / 2 = 35 \text{ kips per frame}$$

Level 1 – Rigid diaphragm

$$F_x = 64 \text{ kips}$$

Center of Rigidity (CR) is at geometric center of building due to symmetry

Center of Mass (CM) is assumed at geometric center of building

CM must include accidental torsion of 5% building length each direction (Sec 12.8.4.2)

$$0.05L = 0.05(78) = 3.9 \text{ ft}$$

$$CM = 78 / 2 \pm 3.9 = 47.4 \text{ or } 39.6$$

$E = V_D + V_T =$ Direct shear + Torsional shear

$$V_D = F_x / 4 = 64 / 4 = 16 \text{ kips}$$

$$2 * V_T = M_T / 78 \text{ ft} = F_x e / 78 \text{ ft} = (64)(3.9) / 78 = 3.2 \text{ kips}$$

$$E = 16 + 3.2 / 2 = 17.6 \text{ kips per frame}$$

Appendix B - Seismic Load Calculations for SCBF

All equation references are from ASCE 7-05 (ASCE 2005).

Seismic Base Shear

$$V = C_s W \quad (\text{Eqn 12.8-1})$$

$$C_s = \frac{S_{DS}}{R/I} \quad (\text{Eqn 12.8-2})$$

$$R = 6 \quad (\text{Tbl 12.2-1})$$

$$I = 1.5 \quad (\text{Tbl 11.5-1})$$

$$S_{DS} = \frac{2}{3} S_{ms} \quad (\text{Eqn 11.4-3})$$

$$S_{ms} = F_a S_s \Rightarrow (1.0)(2.0) = 2.0 \quad (\text{Eqn 11.4-1})$$

$$F_a = 1.0 \quad (\text{Tbl 11.4-1})$$

$$S_s = 2.0 \quad (\text{Tbl 22-7})$$

$$S_{DS} = \frac{2}{3} (2.0) = 1.33$$

$$C_s = \frac{1.33}{\frac{6}{1.5}} = 0.3325$$

Check C_s Maximum and Minimum

$$T = C_t h_n^x = (0.02)(48)^{0.75} = 0.365 \quad (\text{Eqn 12.8-7})$$

$$C_t = 0.02 \quad (\text{Tbl 12.8-2})$$

$$x = 0.75 \quad (\text{Tbl 12.8-2})$$

$$h_n = 48 \text{ ft}$$

$$T_L = 12 \quad (\text{Fig 22-15})$$

$$T < T_L$$

$$C_{s \text{ max}} = \frac{S_{D1}}{T R/I} \quad (\text{Eqn 12.8-3})$$

$$S_{D1} = \frac{2}{3} S_{M1} \quad (\text{Eqn 11.4-4})$$

$$S_{M1} = F_v S_1 \Rightarrow (1.5)(0.6) = 0.9 \quad (\text{Eqn 11.4-2})$$

$$F_v = 1.5 \quad (\text{Tbl 11.4-2})$$

$$S_1 = 0.6 \quad (\text{Tbl 22-8})$$

$$S_{D1} = \frac{2}{3}(0.9) = 0.6$$

$$C_s \max = \frac{0.6}{0.365 \left(\frac{6}{1.5} \right)} = 0.411 > 0.3325 \therefore \text{OK}$$

$$C_s \min = 0.01 < 0.3325 \therefore \text{OK} \quad (\text{Eqn 12.8-5})$$

$$C_s \min = \frac{0.5 S_1}{R/I} = \frac{(0.5)(0.6)}{6/1.5} = 0.075 < 0.3325 \therefore \text{OK} \quad (\text{Eqn 12.8-6})$$

$$W = 1670 \text{ kips} \quad (\text{Sec 12.7.2})$$

$$V = (0.3325)(1670) = 555 \text{ kips}$$

Vertical Distribution of Base Shear

$$F_x = C_{vx} V \quad (\text{Eqn 12.8-11})$$

$$C_{vx} = \frac{w_x h_{xk}}{\sum w_i h_{ik}} \quad (\text{Eqn 12.8-12})$$

The vertical distribution is shown in Table 14.

Table 14. Vertical distribution of base shear in SCBF

Level	w_x (kips)	h_x (ft)	$w_x h_x$ (k-ft)	C_{vx}	F_x (kips)
4	180	48	8640	0.194	107
3	500	36	18000	0.403	224
2	500	24	12000	0.269	149
1	500	12	6000	0.134	75
$w_i h_i$ (k-ft) =			44640	V =	555

Horizontal Distribution of Base Shear Force

Level 4 – Flexible diaphragm

$$F_x = 107 \text{ kips}$$

$$E = F_x/4 = 107/4 = 26.8 \text{ kips per frame}$$

Level 3 – Rigid diaphragm

$$F_x = 224 \text{ kips}$$

Center of Rigidity (CR) is at geometric center of building due to symmetry

Center of Mass (CM) is assumed at geometric center of building

CM must include accidental torsion of 5% building length each direction (Sec 12.8.4.2)

$$0.05L = 0.05(78) = 3.9 \text{ ft}$$

$$CM = 78/2 \pm 3.9 = 47.4 \text{ or } 39.6$$

$E = V_D + V_T =$ Direct shear + Torsional shear

$$V_D = F_x/4 = 224/4 = 56 \text{ kips}$$

$$2 * V_T = M_T/78 \text{ ft} = F_x e / 78 \text{ ft} = (224)(3.9)/78 = 11.6 \text{ kips}$$

$$E = 56 + 11.6/2 = 62 \text{ kips per frame}$$

Level 2 – Rigid diaphragm

$$F_x = 149 \text{ kips}$$

Center of Rigidity (CR) is at geometric center of building due to symmetry

Center of Mass (CM) is assumed at geometric center of building

CM must include accidental torsion of 5% building length each direction (Sec 12.8.4.2)

$$0.05L = 0.05(78) = 3.9 \text{ ft}$$

$$CM = 78/2 \pm 3.9 = 47.4 \text{ or } 39.6$$

$E = V_D + V_T =$ Direct shear + Torsional shear

$$V_D = F_x/4 = 149/4 = 37.3 \text{ kips}$$

$$2 * V_T = M_T / 78 \text{ ft} = F_x e / 78 \text{ ft} = (149)(3.9) / 78 = 7.5 \text{ kips}$$

$$E = 37.3 + 7.5 / 2 = 41 \text{ kips per frame}$$

Level 1 – Rigid diaphragm

$$F_x = 75 \text{ kips}$$

Center of Rigidity (CR) is at geometric center of building due to symmetry

Center of Mass (CM) is assumed at geometric center of building

CM must include accidental torsion of 5% building length each direction (Sec 12.8.4.2)

$$0.05L = 0.05(78) = 3.9 \text{ ft}$$

$$CM = 78 / 2 \pm 3.9 = 47.4 \text{ or } 39.6$$

$E = V_D + V_T =$ Direct shear + Torsional shear

$$V_D = F_x / 4 = 75 / 4 = 18.8 \text{ kips}$$

$$2 * V_T = M_T / 78 \text{ ft} = F_x e / 78 \text{ ft} = (75)(3.9) / 78 = 3.7 \text{ kips}$$

$$E = 18.8 + 3.7 / 2 = 20.6 \text{ kips per frame}$$

Appendix C - BRBF Design Process Calculations

All equation references are from AISC Seismic Provision 341 unless noted otherwise (AISC 2006a).

Brace Design

$$P_u \leq \phi P_{ysc} = \phi F_{ysc} A_{sc} \Rightarrow A_{sc} = \frac{P_u}{\phi F_{ysc}} \quad (\text{Eqn 16-1})$$

$$\phi F_{ysc} = (0.9)(36) = 32.4 \text{ ksi}$$

Table 15 shows the brace core areas calculated. Required strength, P_u taken from RISA-3D 7.0.

Governing load combination for brace design

$$(1.2 + 0.2S_{DS})D + \rho Q_e + 0.5L + 0.2S \quad (\text{ASCE Sec 12.4.2.3})$$

$$\rho = 1.0 \text{ for BRBF member design} \quad (\text{ASCE Sec 12.3.4})$$

Table 15. BRBF brace required strength and sizes.

Level	Axial Force P_u (kips)	Area of Brace A_{sc} (in ²)
4	20.5	0.63
3	77	2.38
2	100	3.08
1	113	3.49

Adjusted Brace Force

Compression:

$$\beta \omega R_y P_{ysc} = \beta \omega R_y F_y A_{sc} = (1.1)(1.35)(1.3)(36)(A_{sc}) \quad (\text{Sec 16.2d})$$

Tension:

$$\omega R_y P_{y_{sc}} = \omega R_y F_y A_{sc} = (1.35)(1.3)(36)(A_{sc}) \quad (\text{Sec 16.2d})$$

Table 16 shows the adjusted brace forces in tension and compression for each level.

Table 16. Adjusted brace strength

Level	Area of Brace A_{sc} (in ²)	Compression (kips)	Tension (kips)
4	0.63	44	40
3	2.38	165	150
2	3.08	214	195
1	3.49	243	220

Column Design

The sizing of the column uses ASCE 7 load combinations in Section 12.4.3.2 with the overstrength factor calculated as below.

$$\Omega = \frac{\beta \omega R_y F_y A_{sc}}{P_u} \Rightarrow \frac{\beta \omega R_y}{\phi} = \frac{(1.1)(1.35)(1.3)}{0.9} = 2.145 \quad (\text{Sec 16.5b})$$

The forces on the column are then taken from a computer model (RISA-3D 7.0). The column is sized for the axial force corresponding to the amplified seismic load without any applied moment.

Try W12x45

$$\Omega P_u = 283 \leq \phi P_n = 397 \therefore OK \quad (\text{Sec 8.3(1)})$$

Check if column size can be reduced

$$P_u / \phi_c P_n > 0.4 \Rightarrow 173 / 397 = 0.44 > 0.4 \therefore \text{Cannot reduce size} \quad (\text{Sec 8.3})$$

Check W12x45 flange if seismically compact

$$b_f/t < 0.30\sqrt{E/F_y} \Rightarrow 7.0 < 0.30\sqrt{29000/50} = 7.225 \therefore OK \quad (\text{Tbl I-8-1})$$

Check W12x45 web if seismically compact

$$C_a = P_u / \phi_b P_y = 283 / (0.9)(655) = 0.48 \quad (\text{Tbl I-8-1 Footnote [k]})$$

$$C_a > 0.125 \therefore \downarrow \quad (\text{Tbl I-8-1})$$

$$h/t_w < 1.12\sqrt{E/F_y}(2.33 - C_a) \geq 1.49\sqrt{E/F_y} \quad (\text{Tbl I-8-1})$$

$$(1.12)\sqrt{29000/50}(2.33 - 0.48) = 49.90 \geq (1.49)\sqrt{29000/50} = 35.88$$

$$h/t_w = 29.6 < 49.90 \therefore OK$$

W12x45 Column selected for frame.

Beam Design

The beams of the BRBF are sized with the same overstrength factor as the columns.

$$\Omega = 2.145 \quad (\text{Sec 16.5b})$$

However, for the beams the seismic load effect is taken as the adjusted brace forces in Table 16, which include the overstrength factor, resolved into vertical and horizontal components. The process to determine these components for Level 1 is shown with a summary of the components for every level in Table 17.

Vertical

$$(C - T) \cos\left(\tan^{-1}\left(\frac{l}{h}\right)\right) = (243 - 220) \cos\left(\tan^{-1}\left(\frac{13}{12}\right)\right) = 15.6 \text{ kips}$$

The vertical component is positive which means it acts in the upwards direction. Because of this it is neglected for design.

Horizontal

$$\frac{(C + T) \sin\left(\tan^{-1}\left(\frac{l}{h}\right)\right)}{2} = \frac{(243 + 220) \sin\left(\tan^{-1}\left(\frac{13}{12}\right)\right)}{2} = 170 \text{ kips}$$

The horizontal component is divided by two because it is assumed that the force is equally shared by the level above and below the braces.

Table 17. Earthquake effects on beam in BRBF.

Level	Vertical Component (kips)	Horizontal Component (kips)
4	2.7	30.9
3	10.2	116
2	12.9	150
1	15.6	170

These seismic load effects are then combined with the gravity loads and the beam is sized. The calculations for the beam at Level 1 are shown below.

Moments

$$M_D = \frac{(1.252klf)(26^2)}{8} = 105.8kft$$

$$M_L = \frac{(0.78klf)(26^2)}{8} = 65.9kft$$

$$M_E = \text{neglect}$$

Load Combinations

$$1.2D + 1.6L$$

$$1.2(105.8) + 1.6(65.9) = 232.4kft$$

$$1.2D + 0.5L + 1.0E$$

$$1.2(105.8) + 0.5(65.9) + 1.0(0) = 160kft \Leftarrow \text{Governs due to axial load}$$

$$1.2(0) + 0.5(0) + 1.0(170) = 170kips$$

Applied loads beam designed for

$$M_u = 160kft$$

$$P_u = 170kips$$

Try W18x50

$$P_r/P_c = P_u/\phi P_n = 170\text{kips}/344\text{kips} = 0.49 \geq 0.2 \therefore \downarrow$$

$$\frac{P_r}{P_c} + \frac{8}{9} \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \quad (\text{AISC Specification Eqn H1-1a})$$

$$\frac{170\text{kips}}{344\text{kips}} + \frac{8}{9} \left(\frac{160\text{kft}}{379\text{kft}} \right) = 0.869 \leq 1.0 \therefore \text{OK}$$

Check W18x50 flange if seismically compact

$$b_f/t < 0.30 \sqrt{E/F_y} \Rightarrow 6.57 < 0.30 \sqrt{29000/50} = 7.225 \therefore \text{OK} \quad (\text{Tbl I-8-1})$$

Check W18x50 web if seismically compact

$$C_a = P_u/\phi_b P_y = 170/(0.9)(735) = 0.257 \quad (\text{Tbl I-8-1 Footnote [k]})$$

$$C_a > 0.125 \therefore \downarrow \quad (\text{Tbl I-8-1})$$

$$h/t_w < 1.12 \sqrt{E/F_y} (2.33 - C_a) \geq 1.49 \sqrt{E/F_y} \quad (\text{Tbl I-8-1})$$

$$(1.12) \sqrt{29000/50} (2.33 - 0.257) = 55.92 \geq (1.49) \sqrt{29000/50} = 35.88$$

$$h/t_w = 45.2 < 55.56 \therefore \text{OK}$$

W18x50 Beam selected for Level 1 of the frame.

Table 18 shows the beams selected for each level of the BRBF.

Table 18. Beams used in BRBF

Level	Beam Used
4	W10x26
3	W18x46
2	W18x50
1	W18x50

Connection Forces

The connection forces, which are the required strengths of the connections, are used to design the connections of the braces and the beam-to-column. The connection force is 1.1 times the adjusted brace strength for tension and compression. Table 5-4 is repeated here as Table 19 to show the connection forces for each level.

Compression

$$1.1\beta\omega R_y F_y A_{sc} = (1.1)(1.1)(1.35)(1.3)(36)(A_{sc}) \quad (\text{Sec 16.3a})$$

Tension

$$1.1\omega R_y F_y A_{sc} = (1.1)(1.35)(1.3)(36)(A_{sc}) \quad (\text{Sec 16.3a})$$

Table 19. Connection forces in BRBF

Level	Area of Brace A_{sc} (in ²)	Compression (kips)	Tension (kips)
4	0.63	48	44
3	2.38	181.5	165
2	3.08	235	214.5
1	3.49	267	242

Story Drift Check

Using the joint displacement outputs from RISA-3D 7.0 as the elastic displacements of the frame, the story drift for each level is calculated and then compared to the allowable drift limits. The story drift for Level 1 is shown below followed by Table 20 showing the story drifts at each level.

Level 1

$$\delta_x = C_d \delta_{xe} / I$$

$$C_d = 5 \frac{1}{2} \quad (\text{Tbl 12.2-1})$$

$$I = 1.5$$

$$\delta_{xe} = 0.236 \text{ in}$$

$$\delta_x = \left(5\frac{1}{2}\right)(0.236) / 1.5 = 0.87 \text{ in}$$

(Tbl 11.5-1)

(RISA-3D 7.0 output)

Check allowable drift limit

$$\Delta \leq 0.010h_{sx}$$

(Tbl 12.12-1)

$$\Delta = (0.010)(12' * 12'') = 1.44$$

$$\delta_x = 0.87 < 1.44 \therefore \text{OK at Level 1}$$

Table 20. Story drifts of BRBF

Level	$\bar{\delta}_{xe}$ (in)	$\bar{\delta}_x$ (in)	Δ_x (in)
4	1.01	3.70	0.97
3	0.744	2.73	0.91
2	0.495	1.82	0.95
1	0.236	0.87	0.87

Appendix D - SCBF Design Process Calculations

All equation references are from AISC Seismic Provision 341 unless noted otherwise (AISC 2006a).

Brace Design

For the brace sizing the axial loads on the brace were determined through RISA-3D 7.0. The load combination resulting maximum compression was ASCE 7 Section 12.4.2.3 LRFD Equation 5 and the maximum tension was from the same section Equation 7. The brace calculations for Level 1 are below followed by Table 21 with the brace selections for each level.

Governing compression load combination for brace design

$$(1.2 + 0.2S_{Ds})D + \rho Q_e + 0.5L + 0.2S \quad (\text{ASCE Sec 12.4.2.3})$$

$$\rho = 1.0 \text{ for SCBF member design} \quad (\text{ASCE Sec 12.3.4})$$

$$P_u = 123.5 \text{ kips}$$

Governing tension load combination for brace design

$$(0.9 - 0.2S_{Ds})D + \rho Q_e \quad (\text{ASCE Sec 12.4.2.3})$$

$$\rho = 1.0 \text{ for SCBF member design} \quad (\text{ASCE Sec 12.3.4})$$

$$T_u = 93.5 \text{ kips}$$

Try HSS 6x6x3/8 – $F_y = 46 \text{ ksi}$

$$\phi P_n = 172 \text{ kips} \quad (\text{Steel Construction Manual (AISC 2005b) (SCM) Tbl 4-4})$$

$$\phi T_n = 248 \text{ kips} \quad (\text{SCM Tbl 5-5})$$

$$\phi P_n > P_u \therefore \text{OK}$$

$$\phi T_n > T_u \therefore \text{OK}$$

Check Local Buckling

Seismic Design Manual (SDM) Table 1-4b

HSS 6x6x3/8 ∴ OK

(SDM Tbl 1-4b)

Check Slenderness Requirements

$$KL/r \leq 4.0\sqrt{E/F_y}$$

(Sec 13.2a)

$$K = 1.0$$

(SCM Tbl C-C2.2)

$$L = 17.7 \text{ ft} = 212.4 \text{ in}$$

$$r = 2.28 \text{ in}$$

(SCM Tbl 1-12)

$$(1.0)(212.4) / 2.28 = 93.16 \leq 4.0\sqrt{29000/46} = 100.4 \therefore \text{OK}$$

HSS 6x6x3/8 selected for brace at Level 1.

Table 21. SCBF brace sizes

Level	Square HSS used
4	5½ x 5½ x 5/16
3	5½ x 5½ x 5/16
2	5½ x 5½ x 5/16
1	6 x 6 x 3/8

Column Design

The sizing of the column uses ASCE 7 load combinations in Section 12.4.3.2 with the overstrength factor from Table 12.2-1.

$$\Omega = 2.0$$

(ASCE 7-05 Tbl 12.2-1)

The forces on the column are then taken from a computer model (RISA-3D 7.0). The column is sized for the axial force corresponding to the amplified seismic load without any applied moment.

Try W12x45

$$\Omega P_u = 307 \leq \phi P_n = 397 \therefore OK \quad (\text{Sec 8.3(1)})$$

Check if column size can be reduced

$$P_u / \phi_c P_n > 0.4 \Rightarrow 193 / 397 = 0.49 > 0.4 \therefore \text{Cannot reduce size} \quad (\text{Sec 8.3})$$

Check W12x45 flange if seismically compact

$$b_f / t < 0.30 \sqrt{E / F_y} \Rightarrow 7.0 < 0.30 \sqrt{29000 / 50} = 7.225 \therefore OK \quad (\text{Tbl I-8-1})$$

Check W12x45 web if seismically compact

$$C_a = P_u / \phi_b P_y = 283 / (0.9)(655) = 0.48 \quad (\text{Tbl I-8-1 Footnote [k]})$$

$$C_a > 0.125 \therefore \downarrow \quad (\text{Tbl I-8-1})$$

$$h / t_w < 1.12 \sqrt{E / F_y} (2.33 - C_a) \geq 1.49 \sqrt{E / F_y} \quad (\text{Tbl I-8-1})$$

$$(1.12) \sqrt{29000 / 50} (2.33 - 0.48) = 49.90 \geq (1.49) \sqrt{29000 / 50} = 35.88$$

$$h / t_w = 29.6 < 49.90 \therefore OK$$

W12x45 Column selected for frame.

Beam Design

The seismic load effects on the beams of the SCBF are taken from an assumed brace force. These forces are then resolved into vertical and horizontal components. The process to determine these components for Level 1 is shown with a summary of the components for every level in Table 22.

Assumed brace tension force

$$P_t = R_y F_y A_g = (1.4)(46)(7.58) = 488 \text{ kips} \quad (\text{Sec 13.4a})$$

$$A_g = 7.58 \text{ in}^2 \quad (\text{SCM Tbl 1-12})$$

Assumed brace compression force

$$P_c = 0.3 P_n = 0.3 \left(\frac{\phi P_n}{\phi} \right) = (0.3) \left(\frac{172}{0.9} \right) = 57 \text{ kips} \quad (\text{Sec 13.4a})$$

$$\phi P_n = 172 \text{ kips} \quad (\text{SCM Tbl 4-4})$$

Vertical

$$(C - T) \cos \left(\tan^{-1} \left(\frac{l}{h} \right) \right) = (57 - 488) \cos \left(\tan^{-1} \left(\frac{13}{12} \right) \right) = -292 \text{ kips}$$

The vertical component is negative which means it acts in the downwards direction.

Horizontal

$$\frac{(C + T) \sin \left(\tan^{-1} \left(\frac{l}{h} \right) \right)}{2} = \frac{(57 + 488) \sin \left(\tan^{-1} \left(\frac{13}{12} \right) \right)}{2} = 200 \text{ kips}$$

The horizontal component is divided by two because it is assumed that the force is equally shared by the level above and below the braces.

Table 22. Earthquake effects on beam in SCBF.

Level	Assumed brace force Tension (kips)	Assumed brace force Compression (kips)	Vertical Component (kips)	Horizontal Component (kips)
4	377	40	-229	153
3	377	40	-229	153
2	377	40	-229	153
1	488	57	-292	200

These seismic load effects are then combined with the gravity loads and the beam is sized. The calculations for the beam at Level 1 are shown below.

Moments

$$M_D = \frac{(1.252klf)(26^2)}{8} = 105.8kft$$

$$M_L = \frac{(0.78klf)(26^2)}{8} = 65.9kft$$

$$M_E = \frac{(292)(26)}{4} = 1898kft$$

Load Combinations

$$1.2D + 1.6L$$

$$1.2(105.8) + 1.6(65.9) = 232.4kft$$

$$1.2D + 0.5L + 1.0E$$

$$1.2(105.8) + 0.5(65.9) + 1.0(1898) = 2058kft \leftarrow \text{Governs}$$

$$1.2(0) + 0.5(0) + 1.0(170) = 200kips$$

Applied loads beam designed for

$$M_u = 2058kft$$

$$P_u = 200kips$$

Try W27x217

$$P_r/P_c = P_u/\phi P_n = 200kips/1995kips = 0.10 \leq 0.2 \therefore \downarrow$$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \quad (\text{AISC Specification Eqn H1-1b})$$

$$\frac{200kips}{2(344kips)} + \left(\frac{2058kft}{2670kft} \right) = 0.82 \leq 1.0 \therefore OK$$

Check W27x217 flange if compact

$$b_f/t < 0.38\sqrt{E/F_y} \Rightarrow 4.71 < 0.38\sqrt{29000/50} = 9.15 \therefore OK \quad (\text{SCM Tbl B4.1})$$

Check W27x217 web if compact

$$b_f/t < 3.76\sqrt{E/F_y} \Rightarrow 28.7 < 3.76\sqrt{29000/50} = 90.6 \therefore OK \quad (\text{SCM Tbl B4.1})$$

W27x217 Beam selected for Level 1 of the frame.

Table 23 shows the beams selected for each level of the SCBF.

Table 23. Beams used in SCBF.

Level	Beam Used
4	W27x146
3	W27x161
2	W27x161
1	W27x217

Connection Forces

The connection forces are used to design the connections of the braces and the beam-to-column. The connection force in tension, flexure, and compression are described in Sections 13.3a, 13.3b, and 13.3c respectively. The calculation for the connection forces at Level 1 are presented below with Table 24 showing the connection forces for each level.

Tension

$$R_y F_y A_g = (1.4)(46)(7.58) = 488 \text{ kips} \quad (\text{Sec 13.3a})$$

Flexure

$$1.1 R_y M_p = 1.1(1.4)(54.6) = 84 \text{ kft} \quad (\text{Sec 13.3b})$$

$$M_p = 54.6 \text{ kft} \quad (\text{SCM Tbl 3-13})$$

Compression

$$1.1R_y P_n = 1.1R_y F_{cr} A_g = 1.1(1.4)(33.6)(7.58) = 392 \text{ kips} \quad (\text{Sec 13.3c})$$

$$F_{cr} = 33.6 \text{ ksi} \quad (\text{SCM Tbl 4-22})$$

Table 24. Connection forces in SCBF

Level	Square HSS used	Compression (kips)	Tension (kips)	Flexure (kip-ft)
4	5½ x 5½ x 5/16	287	377	49
3	5½ x 5½ x 5/16	287	377	49
2	5½ x 5½ x 5/16	287	377	49
1	6 x 6 x 3/8	392	488	84

Story Drift Check

Using the joint displacement outputs from RISA-3D 7.0 as the elastic displacements of the frame, the story drift for each level is calculated and then compared to the allowable drift limits. The story drift calculations for Level 1 are shown below followed by Table 25 showing the story drifts at each level.

Level 1

$$\delta_x = C_d \delta_{xe} / I$$

$$C_d = 5 \quad (\text{Tbl 12.2-1})$$

$$I = 1.5 \quad (\text{Tbl 11.5-1})$$

$$\delta_{xe} = 0.123 \text{ in} \quad (\text{RISA-3D 7.0 output})$$

$$\delta_x = \left(5\frac{1}{2}\right)(0.123) / 1.5 = 0.41 \text{ in}$$

Check allowable drift limit

$$\Delta \leq 0.010h_{sx}$$

(Tbl 12.12-1)

$$\Delta = (0.010)(12' * 12'') = 1.44$$

$$\delta_x = 0.41 < 1.44 \therefore \text{OK at Level 1}$$

Table 25. Story drifts of SCBF

Level	$\bar{\delta}_{xe}$ (in)	$\bar{\delta}_x$ (in)	Δ_x (in)
4	0.52	1.74	0.27
3	0.44	1.47	0.49
2	0.294	0.98	0.57
1	0.123	0.41	0.41