NIST GCR 13-917-24

NEHRP Seismic Design Technical Brief No. 8

Seismic Design of Steel Special Concentrically Braced Frame Systems

A Guide for Practicing Engineers

Rafael Sabelli
Charles W. Roeder
Jerome F. Hajjar

National Institute of
Standards and Technology
U.S. Department of Commerce

NEHRP Seismic Design Technical Briefs

National Earthquake Hazards Reduction Program (NEHRP) Technical Briefs are published by the National Institute of Standards and Technology (NIST) as aids to the efficient transfer of NEHRP and other research into practice, thereby helping to reduce the nation's losses from earthquakes.

NIST National Institute of Standards and Technology

NIST is a federal technology agency within the U.S. Department of Commerce that promotes U.S. innovation and industrial competitiveness by advancing measurement science, standards, and technology in ways that enhance economic security and improve our quality of life. It is the lead agency of the NEHRP. Dr. John (Jack) R. Hayes, Jr., is the Director of NEHRP within NIST's Engineering Laboratory (EL). Dr. John (Jay) L. Harris, III, managed the project to produce this Technical Brief for EL.

NEHRP Consultants Joint Venture

This NIST-funded publication is one of the products of the work of the NEHRP Consultants Joint Venture carried out under Contract SB134107CQ0019, Task Order 12-335. The partners in the NEHRP Consultants Joint Venture are the Applied Technology Council (ATC) and the Consortium of Universities for Research in Earthquake Engineering (CUREE). The members of the Joint Venture Management Committee are James R. Harris, Robert Reitherman, Christopher Rojahn, and Andrew Whittaker, and the Program Manager is Jon A. Heintz.

About The Authors

Rafael Sabelli, P.E., S.E., is Director of Seismic Design at Walter P Moore, a structural and civil engineering firm with offices nationwide. He is a member of the American Institute of Steel Construction (AISC) Task Committee 9 – Seismic Provisions, AISC Seismic Design Manual Committee, the Building Seismic Safety Council's 2014 Provisions Update Committee (PUC), and the American Society of Civil Engineers (ASCE) Seismic Subcommittee for ASCE 7.

Charles W. Roeder, Ph.D., P.E., is a Professor of Civil Engineering at the University of Washington. He has performed extensive research on the seismic performance of steel and composite structures, and his research has focused on the seismic performance and rehabilitation of braced frames for the past several years. He is a member of AISC, Structural Engineering Institute (SEI), and Earthquake Engineering Research Institute (EERI), and serves on the ASCE 41 and AISC 358 standards committees.

Jerome F. Hajjar, Ph.D., P.E., is a Professor and the Department Chair in the Department of Civil and Environmental Engineering at Northeastern University. He serves on the AISC Committee on Specifications and several of its task committees and is a member of the SEI Technical Activities Division Executive Committee. He is a fellow of ASCE and SEI and the winner of the 2010 Popular Mechanics Breakthrough Award, the 2005 AISC T. R. Higgins Lectureship Award, the 2004 AISC Special Achievement Award, and the 2000 ASCE Norman Medal. He is a registered professional engineer in Illinois and Minnesota.

About the Review Panel

The contributions of the three review panelists for this publication are gratefully acknowledged.

Michel Bruneau, Ph.D., P.E., is Professor in the Department of Civil, Structural, and Environmental Engineering at the University at Buffalo, the State University of New York, and an ASCE Fellow. He is a member of several AISC and Canadian Standards Association committees developing design specifications for bridges and buildings, and he served as Director of the Multidisciplinary Center for Earthquake Engineering Research. He has conducted extensive research on the design and behavior of steel structures subjected to earthquake and blast and has received many awards for his work. He has authored over 400 technical publications, including co-authoring the textbook *Ductile Design of Steel Structures* and three fiction books.

John A. Rolfes, P.E., S.E., is a Vice-President with Computerized Structural Design in Milwaukee, Wisconsin, a consulting engineering firm that provides structural engineering services throughout the United States. He is a member of the American Institute of Steel Construction Task Committee 9 – Seismic Provisions, AISC Seismic Design Manual Committee, AISC Industrial Buildings and Non-Building Structures Committee, and the Association of Iron and Steel Technology (AIST) Mill Buildings Committee.

C. Mark Saunders, P.E., S.E. is Senior Consultant and past President of Rutherford + Chekene Consulting Engineers in San Francisco and has been involved in seismic design and code development for more than 40 years. He has been a member of the AISC Task Committee 9 - Seismic Provisions, for more than 20 years and is currently its Vice Chair. He has served on the PUC for the NEHRP Provisions and is a past president of both the Structural Engineers Association of Northern California and ATC.

ATC

Applied Technology Council (ATC)
201 Redwood Shores Parkway - Suite 240
Redwood City, California 94065
(650) 595-1542
www.atcouncil.org email: atc@atcouncil.org

CUREE

Consortium of Universities for Research in Earthquake Engineering (CUREE)
1301 South 46th Street - Building 420
Richmond, CA 94804
(510) 665-3529
www.curee.org email: curee@curee.org

NIST GCR 13-917-24

Seismic Design of Steel Special Concentrically Braced Frame Systems

A Guide for Practicing Engineers

Prepared for
U.S. Department of Commerce
National Institute of Standards and Technology
Engineering Laboratory
Gaithersburg, MD 20899-8600

By

Rafael Sabelli
Walter P Moore

Charles W. Roeder
University of Washington

Jerome F. Hajjar
Northeastern University

July 2013

U.S. Department of Commerce
Penny Pritzker, Secretary

National Institute of Standards and Technology
Patrick D. Gallagher, Under Secretary of Commerce for Standards and Technology and Director

Contents

Disclaimers

This Technical Brief was prepared for the Engineering Laboratory of the National Institute of Standards and Technology (NIST) under the National Earthquake Hazards Reduction Program (NEHRP) Earthquake Structural and Engineering Research Contract SB134107CQ0019, Task Order 12-335. The statements and conclusions contained herein are those of the authors and do not necessarily reflect the views and policies of NIST or the U.S. Government.

This report was produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council (ATC) and the Consortium of Universities for Research in Earthquake Engineering (CUREE). While endeavoring to provide practical and accurate information, the NEHRP Consultants Joint Venture, the authors, and the reviewers assume no liability for, nor express or imply any warranty with regard to, the information contained herein. Users of the information contained in this report assume all liability arising from such use.

The policy of NIST is to use the International System of Units (metric units) in all of its publications. However, in North America in the construction and building materials industry, certain non-SI units are so widely used instead of SI units that it is more practical and less confusing to include measurement values for customary units only in this publication.

Cover photo – Entrance lobby of the Kirsch Center for Environmental Studies, De Anza College, Cupertino, CA.

How to Cite This Publication

Sabelli, Rafael, Roeder, Charles W., and Hajjar, Jerome F. (2013). "Seismic design of steel special concentrically braced frame systems: A guide for practicing engineers," *NEHRP Seismic Design Technical Brief No. 8*, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 13-917-24.

1. Introduction

Concentrically Braced Frames (CBFs) are a class of structures resisting lateral loads through a vertical concentric truss system, the axes of the members aligning concentrically at the joints. CBFs tend to be efficient in resisting lateral forces because they can provide high strength and stiffness. These characteristics can also result in less favorable seismic response, such as low drift capacity and higher accelerations. CBFs are a common structural steel or composite system in areas of any seismicity.

Special Concentrically Braced Frames (SCBFs) are a special class of CBF that are proportioned and detailed to maximize inelastic drift capacity. This type of CBF system is defined for structural steel and composite structures only.

The primary source of drift capacity in SCBFs is through buckling and yielding of diagonal brace members. Proportioning and detailing rules for braces ensure adequate axial ductility, which translates into lateral drift capacity for the system. Special design and detailing rules for connections, beams, and columns attempt to preclude less ductile modes of response that might result in reduced lateral drift capacity.

This Guide addresses the seismic design of steel SCBFs in typical building applications. While the emphasis here is placed on steel SCBFs, some aspects are also applicable to composite SCBFs. Where appropriate, experimental and analytical studies are described, and observations from past earthquakes are included to illustrate the purpose of certain design requirements. Additionally, some of the strategies for achieving ductility are applicable to other structural systems. This Guide is not a complete treatment of the steel SCBF system nor of the general principles of ductile design of steel structures.

Items not covered in this document

A number of important issues related to the topic of steel concentrically braced frames are not addressed in this document; these include:

- Classes of braced frames other than SCBFs, such as Buckling-Restrained Braced Frames, Eccentrically Braced Frames, and Ordinary Concentrically Braced Frames; (separate Technical Briefs are planned for these systems);
- Self-centering systems with braced frames;
- Diagrid systems and other braced frames in which braces are additionally required to support substantial gravity loads;
- Multi-tier concentrically braced frames requiring special stability considerations;
- The use of Special Concentrically Braced Frame (SCBF) design methods in other components, such as in a horizontal diaphragm; and
- Special configurations of concentrically braced frames designed to engage multiple stories in a single yielding mechanism (e.g., the “Zipper” configuration).

This Technical Brief refers to the following building codes and standards:

- AISC 341-10, *Seismic Provisions for Structural Steel Buildings and Commentary*, 2010 edition (AISC 2010a)
- AISC 360-10, *Specification for Structural Steel Buildings and Commentary*, 2010 edition (AISC 2010b)
- ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, 2010 edition (ASCE 2010)
- IBC *International Building Code*, 2012 edition (IBC 2012).

Designers are responsible for verifying the current legally applicable requirements in the jurisdiction of their project. The Technical Briefs in this NEHRP series typically are based on the latest available codes and standards, which may not yet have been adopted locally. Discussion with and approval by the building official should occur to verify that a later version of a code or standard not yet adopted locally may be used.

In addition to the code and standards listed above, designers should be aware of other available resources:

- AISC *Seismic Design Manual* (AISC 2012)
- *Ductile Design of Steel Structures* (Bruneau et al. 2011)
- *SEAOC Structural/Seismic Design Manual* (SEAOC 2013).

This Guide is intended to aid the reader in identifying significant aspects of seismic design and behavior and to identify resources that are useful for design and for understanding braced frame behavior and performance. It is not intended to repeat detailed design guidance found elsewhere. It was written for practicing structural engineers and is intended to provide guidance in the application of code requirements for the design of SCBFs. This Guide is also useful to others wishing to apply building code provisions correctly, such as building officials, and to those interested in understanding the basis of such code provisions and of common design methods, such as educators and students.

Section 2 discusses where and how SCBFs are typically employed. Section 3 summarizes the design principles for the system. Section 4 provides guidance for analysis of this system. Section 5 provides guidance on design procedures and decisions. Section 6 discusses additional considerations and requirements. Section 7 discusses detailing and constructability issues.

2. The Use of Steel Special Concentrically Braced Frames

The SCBF system is generally an economical system to use for low-rise buildings in areas of high seismicity. It is sometimes preferred over Special Moment Frames because of the material efficiency of CBFs and the smaller required beam and column depths. SCBFs are only possible for buildings that can accommodate the braces in their architecture. Buildings for which this a problem may be well suited for Special Moment Frames.

Up to the present, SCBFs have been used more extensively than Buckling-Restrained Braced Frames (BRBFs). BRBFs generally offer cost and performance advantages for buildings three stories and higher, but SCBFs continue to be popular because of the level of experience designers and fabricators have with the system.

The desired performance of the SCBF system is based on providing high levels of brace ductility to achieve large inelastic drifts. It is not particularly well suited for applications in which the seismic demands are low. The capacity design rules for connections can be uneconomical in cases where brace sizes are governed by wind loads or by slenderness limits.

SCBFs are designed using capacity design procedures, with the braces serving as the fuses of the system. Optimal design of SCBFs entails careful selection and proportioning of braces so as to provide limited overstrength and avoid a concentration of inelastic demands. Designers should strive for a small range of brace demand-to-capacity ratios so that the resulting system is proportioned to spread yielding over multiple stories rather than concentrating it at a single location. Overstrength can be beneficial, but care should be taken to maintain a well-proportioned design in order to avoid concentration of ductility demands.

Braced frames are most effective at the building perimeter, where they can control the building's torsional response. ASCE 7 allows buildings to be considered sufficiently redundant (and thus avoid a penalty factor) with two braced bays on each of the presumed four outer lines (assuming a rectangular layout). Such a layout is good for torsion control as well.

In mid-rise or high-rise buildings, SCBFs are often used in the core of the structure, with a perimeter moment frame used to provide additional torsional resistance.

Stacked braced frames (frames in which the braces occupy the same plan location at each level) can have high overturning forces. In many cases it is advantageous to spread the overturning forces out over several bays to reduce foundation and anchorage forces. The design of elements interconnecting these frames is critical to ensure that brace ductility remains the primary source of inelastic drift.

Seismic Retrofit

Braced frames can be an effective system for seismic retrofit due to their high stiffness and because they can be assembled from pieces of relatively small size and weight. SCBFs may be considered for seismic retrofit in cases in which the building deformations corresponding to brace axial ductility are not detrimental to the building performance. In many retrofit projects this is not the case due to the presence of brittle, archaic materials and sensitive finishes not detailed to accommodate significant drift. In such cases, the added drift capacity provided by the careful proportioning and detailing required for the SCBF system is of little benefit, and a conventional braced frame system or other stiff system should be considered instead.

3. Principles for Design of Steel Special Concentrically Braced Frames

SCBFs economically develop the lateral strength and stiffness needed to assure serviceable structural performance during smaller, frequent earthquakes, but the inelastic deformation needed to ensure life safety through collapse prevention during extreme earthquakes is dominated by tensile yielding of the brace, brace buckling, and post-buckling deformation of the brace. The ductility and inelastic deformations required by this second design goal vary in magnitude depending upon the seismic hazard level and the seismic design procedure. For areas of low seismicity, ASCE 7 allows steel framing systems to be designed with a Response Modification Factor, *R*, of 3.0 with no special detailing requirements to improve ductility. ASCE 7 also allows the use of Ordinary Concentrically Braced Frames (OCBFs). However, SCBFs are designed with relatively large *R* factors, and as a consequence are expected to experience relatively large inelastic deformation demands during extreme ground shaking. A story drift of approximately 2.5 % is commonly assumed as a target inelastic deformation to be achieved by SCBFs prior to brace fracture. As a result, ductile detailing and proportioning requirements are needed to ensure that SCBFs can achieve the required inelastic deformations. Corresponding inelastic flexural deformation in beams, columns, and connections will occur during these large inelastic excursions. The inelastic deformations in the beams and columns are not primary effects because they are not specific goals of the design process. Nevertheless, they influence the seismic performance of SCBFs and contribute to the cost of repair. Local slenderness limits for beams and columns are required by AISC 341 in recognition of these local inelastic deformations.

To achieve the desired performance, a number of ductile detailing requirements are applicable to SCBF design. The current procedure is generally rational, but recent research demonstrates increased inelastic deformation capacity may be developed with some modifications to the connection design; these potential modifications are discussed later in this document.

3.1 Success versus Failure

The design method described in general terms above is a multilevel design approach. It is expected to achieve serviceability during the more frequent design basis earthquake through the elastic behavior provided by the initial factored load design. Capacity-based design limits essentially address life safety through collapse prevention limit states for the infrequent, maximum considered design event. As a result, a successful design is expected to have significant inelastic deformation and associated structural damage during larger earthquakes, but structural collapse and associated loss of life are not expected. The primary inelastic deformation occurs within the brace, and therefore, the initial failure within the system should normally be brace fracture. The evaluation of building collapse is an inexact science, but it is clear that brace fracture does not immediately trigger structural collapse. The gusset plate connections are designed conservatively relative to the brace resistance, and experimental and analytical research has shown that the capacity-based design of the gusset plate results in significant lateral resistance after brace fracture because of moment frame behavior. Experiments and analysis suggest that this lateral resistance after initial brace fracture may be the range of 20 % to 40 % of the original braced frame resistance. Limited guidance is available on the behavior of this resulting moment connection for seismic assessment of SCBFs.

General System Performance

Current trends in practice places increasing emphasis upon performance-based design, and more in-depth predictions of damage, structural performance, and collapse are required. Fragility curves are often used as an aid in this process (Roeder et al. 2011). Recent research (Hsiao et al. 2012, 2013a) has developed nonlinear analytical models on the OpenSees computer platform that accurately predict buckling, tensile yielding, and post-buckling behavior of rectangular hollow structural section (HSS) braces and provide verified prediction of brace fracture and frame behavior beyond brace fracture. This model also provides an approximate prediction of local damage to beams and columns, but this prediction of local behavior is inherently more limited with this analytical platform. Hsiao et al. (2013b) performed nonlinear dynamic analyses on braced frames designed to the minimum SCBF design standards and with increased and decreased *R* factors. All designs used the equivalent lateral force method, and the analyses were performed with seismic excitations scaled to the 2 % in 50-year and the 10 % in 50-year seismic levels. Potential collapse was estimated at the point where the analysis became mathematically unstable or when the maximum story drift reached 5 %. The 5 % drift limit was arbitrarily chosen because this is a deformation where the gusset plate connections are expected to start to lose their integrity as moment frame connections. These analyses clearly showed that brace buckling will occur and may be quite common even for 10 % in 50-year earthquake hazard, but brace fracture in SCBFs designed to current standards should not occur during this event. Once brace fracture occurs, the building retains significant structural integrity, but inelastic deformations concentrate in the stories with fractured bracing. The potential for brace fracture during 2 % in 50-year events is significantly larger for shorter (short period) buildings than it is for 20-story (long period) buildings. Taller buildings more commonly experience reduced brace buckling deformation and significantly fewer brace fractures, but the buckling damage more commonly occurs in the upper stories of taller systems because of the contribution of higher modes to the dynamic response. Collapse potential was small for well-designed SCBFs, but it was larger for shorter (three-story) SCBFs than for taller structures. This work suggests that improved or more consistent SCBF design may be possible with changes in the current *R* factors because reduced *R* values reduce the potential for brace buckling, brace fracture, and structural collapse, particularly for short period systems. This suggests that shorter period buildings require a smaller *R* value than longer period buildings to achieve comparable structural safety. Further, the research results suggest that the OCBF and *R*=3 concepts that are used in braced frame design are unlikely to ensure elastic performance during the maximum considered earthquake.

3.2 Intended Behavior

The prior discussion has shown that brace buckling, tensile yielding, and post-buckling performance are the predictably intended behaviors for the SCBF system. Brace fracture is the "preferred" initial failure mode, but it does not in itself trigger immediate collapse. However, it is important to get the maximum possible inelastic deformation capacity from the brace because, once brace fracture occurs, severe concentration of inelastic deformation in the damaged story also occurs.

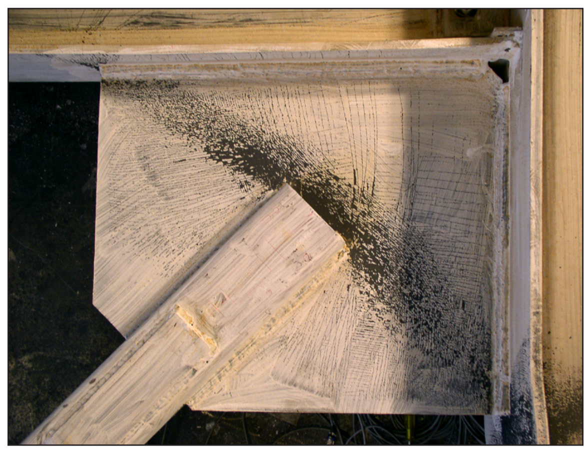
System Behavior

System performance is strongly influenced by aspects of brace behavior (Lehman et al. 2008). Brace buckling places large inelastic demands on the brace at the middle of the brace, typically resulting in a plastic hinge at midspan (**Figure 3-1a**). Brace buckling also places significant demands on gusset plate connections (**Figure 3-1b**) and adjacent framing members (**Figure 3-1c**). Limited cracking of the welds joining the gusset plate to the beams and columns generally is expected because of gusset plate deformation. These cracks normally initiate at story drifts in the range of 1.5 % to 2.0 %, but the cracks remain stable if the welds meet size and demand-critical weld requirements in AISC 341. Current design criteria encourage conservative gusset plate design, but overly conservative gusset plate design can increase the inelastic deformation in the beams and columns adjacent to the gusset plate and does not significantly reduce the deformation of the gusset plate or the demands on the weld. Gusset plate damage and the weld cracking are largely driven by the brace end rotations and the opening and closing of the right angle of the connection.

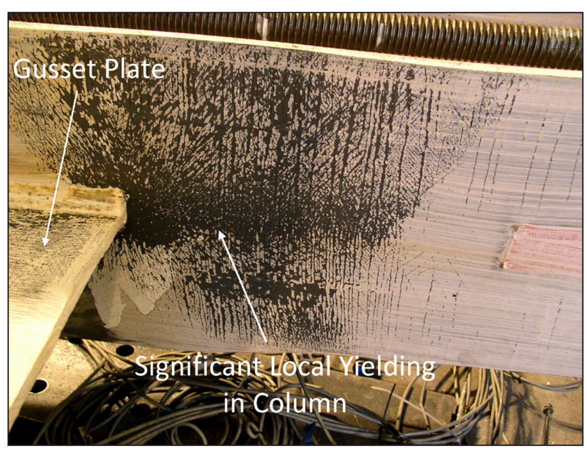
Configuration Issues

The configuration of braces also affects system performance. Multiple configurations of bracing are used, and these configurations are identified in **Figure 3-2**. Braces buckle in compression and yield in tension. The initial compressive buckling capacity is smaller than the tensile yield force, and for subsequent buckling cycles, the buckling capacity is further reduced by the prior inelastic excursion. Therefore, bracing systems must be balanced so that the lateral resistance in tension and compression is similar in both directions. This means that diagonal bracing (**Figure 3-2**) must be used in matched tensile and compressive pairs. As a result, diagonal

(a) Brace buckling deformation



(b) Deformation of gusset plate



(c) Local yielding in beam and column

Figure 3-1 – Various aspects of braced frame behavior.

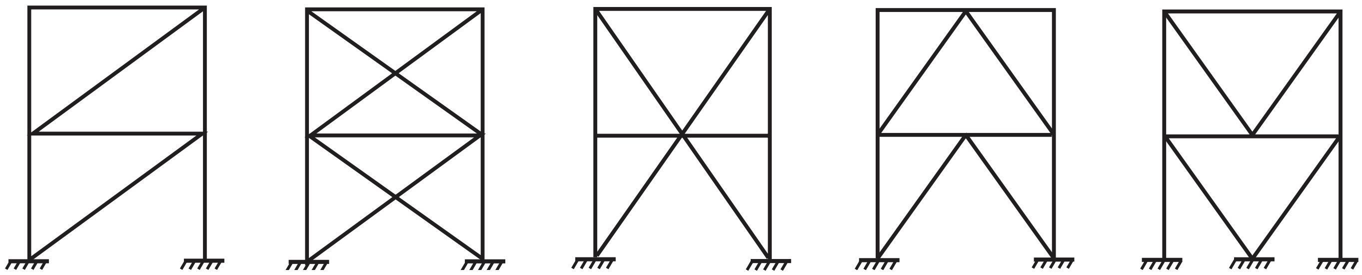


Figure 3-2 – Various braced frame system configurations.

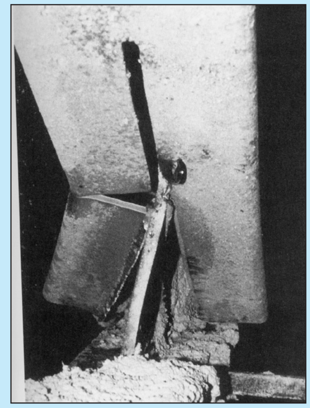
bracing (**Figure 3-2**) must be used in opposing pairs to achieve this required balance. Other bracing configurations, such as the X-brace, multistory X-brace and chevron brace directly achieve this balance. X-bracing is most commonly used with light bracing on shorter structures. Research shows that the buckling capacity of X-bracing is best estimated by using one half the brace length when the braces intersect and connect at mid section (Palmer 2012). However, the inelastic deformation capacity of the X-braced system is somewhat reduced from that achievable with many other braced frame systems because the inelastic deformation is concentrated in one-half the brace length because the other half of the brace cannot fully develop its capacity as the more damaged half deteriorates. The compressive buckling resistance of most other brace configurations is best estimated by considering true end-to-end length of the brace with an effective length factor, K, of 1.0 (i.e., neglecting rotation stiffness of the brace-to-gusset connection.)

Concentration of inelastic deformation in a limited number of stories occurs with braced frames. Experiments suggest that multistory X-bracing offers a slight advantage in that it provides a somewhat more robust path for transferring story shear to adjacent stories even after brace buckling and fracture because the remaining tension brace may directly transfer its force to the next story. Chevron or inverted-chevron bracing (inverted V- or V-bracing) has intersecting brace connections at midspan of the beam (**Figure 3-2**). Large unbalanced forces and bending moments on the beam occur because the buckling load is smaller than the tensile yield resistance and decreases with increasing damage. The bending moment increases as the compressive resistance deteriorates, and AISC 341 requires that the beam be designed for these bending moments. Research shows that the beam deformation associated with the unbalanced forces in chevron bracing increases the axial compressive deformation of the brace and reduces the inelastic deformation capacity prior to brace fracture (Okazaki et al. 2012). However, flexural yielding of the beam increases the damping of dynamic response.

Other bracing configurations are possible, and some are expressly prohibited in AISC 341. K-braces intersect at mid-height of the column. They have the same unbalanced force problem as noted with chevron bracing, but bending moments and inelastic deformation will occur in the column and may fail, triggering collapse. As a result, K-bracing is not permitted for the SCBF system. In addition, tension-only bracing has had relatively poor performance during past earthquakes because the lack of compressive brace resistance leads to inelastic behavior with slack braces that have no stiffness until the slack is taken up. The slack braces may lead to progressively increasing drift and impact loading on the brace, and early brace fracture may occur. Consequently, tension-only bracing is also prohibited for the SCBF system.

(a) Buckled brace

(b) HSS brace fracture at mid-length



(c) HSS brace fracture at net section

(d) Base plate fracture

(e) CBF connection failure

(f) Fracture of CBF box column

Figure 3-3 – Earthquake damage to CBFs.

Observed Earthquake Damage

Braced frames have sustained damage in prior earthquakes (**Figure 3-3**). However, the SCBF design concept was first presented in the first edition of the AISC *Seismic Provisions for Structural Steel Buildings* (AISC 1997), and the SCBF requirements have evolved steadily since then. As a result, past braced frame earthquake damage is relevant to concentrically braced frames, but it does not specifically reflect current SCBF behavior. This brief discussion focuses on the more recent 1989 Loma Prieta, the 1994 Northridge, and the 1995 Hyogo-ken Nanbu (Kobe, Japan) Earthquakes. SCBFs are more commonly used in today's building construction than were CBFs prior to these three earthquakes, but nevertheless, there was a substantial number of CBFs in service during those three earthquakes, and brace buckling (**Figure 3-3a**) was observed.

Limited CBF damage was reported from the 1989 Loma Prieta Earthquake. Buildings generally suffered little damage except for locations on soft soil (EERI 1990a), and no discussion of braced frame damage in buildings was noted. Several braced frames in power generating plants were noted to have significant brace buckling (EERI 1990b). These braces were typically light T-sections in chevron or inverted-V braced frames. Yielded and deformed gusset plate connections and fractured bolted connections were also noted in a few cases.

Somewhat more definitive braced frame damage was noted after the 1994 Northridge Earthquake, but no braced frames collapsed or appeared near incipient collapse from this damage (EERI 1996). Brace buckling was noted for both rectangular HSS tube bracing as well as light steel strap bracing. Rectangular HSS tubes sometimes had substantial plastic hinges at mid-length and near their end connections, and severe local buckling was observed. Fracture occurred at the net section of the connection to the gusset plate (**Figure 3-3c**) and at the mid-length plastic hinge (**Figure 3-3b**). Column base plate fracture because of the applied loads and deformations was also observed (**Figure 3-3d**).

Extensive damage to braced frames was also noted during the 1995 Hyogo-ken Nanbu Earthquake, and a few older low-rise braced frames collapsed during this seismic event (AIJ 1995). A large portion of the collapsed structures had light tension-only strap bracing. The vast majority of the CBFs damaged in this earthquake have design details quite different from the SCBF details commonly used in the U.S. Brace buckling, brace fracture, and connection fracture occurred in a number of buildings (**Figure 3-3e**). Significant problems in the columns of some new high-rise CBFs occurred, and complete fracture of heavy built-up columns was noted (**Figure 3-3f**). **Figure 3-3e** and **Figure 3-3f** show connection and box column failures respectively.

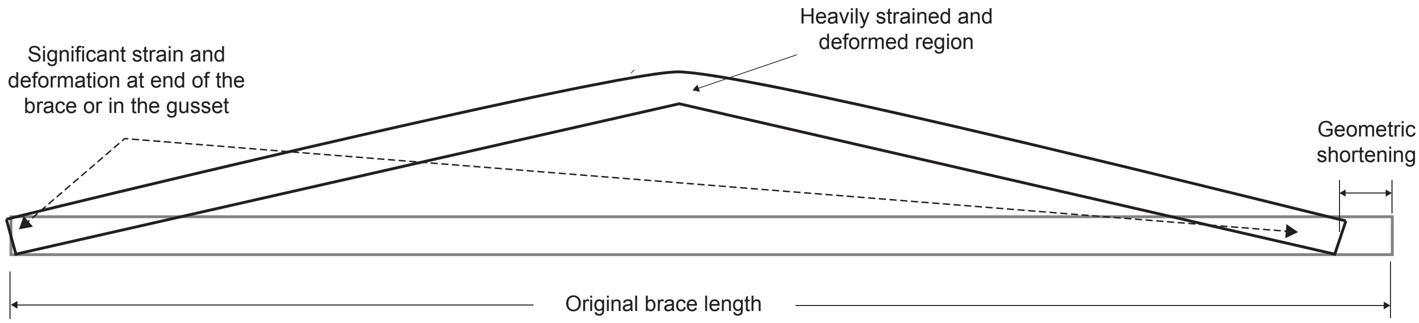


Figure 3-4 – Geometric shortening of the brace and locations of local yield deformation.

Element Behavior

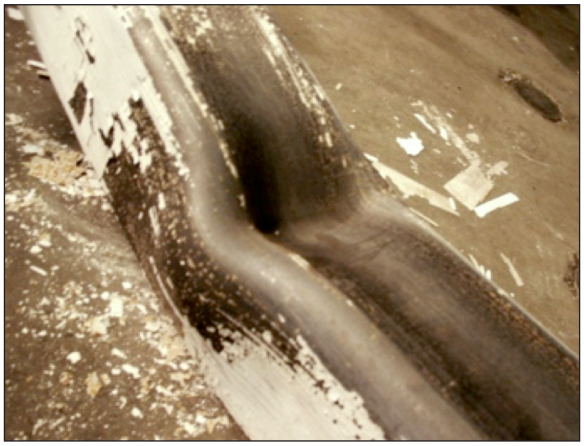
Inelastic deformation of the brace dominates the inelastic performance of SCBFs during moderate and large earthquakes, and fracture of the brace at mid-length is clearly the anticipated initial failure mode of the braced frame system. A number of brace design issues affect the inelastic deformation and ultimate fracture of the brace as illustrated in the sketch of **Figure 3-4**. The inelastic story drift of SCBFs is mostly due to axial shortening and elongation of the brace, as shown in the figure, but this axial shortening and elongation are caused primarily by the geometric effects of the brace buckling deformation. Plastic hinging due to buckling deformation occurs at the center of the brace and at each end. The plastic hinges at the brace ends preferably occur in the gusset plate, although plastic hinging may occur in the brace itself adjacent to the connection if the gusset plate is stiffer and stronger than required, if the gusset plate does not have proper allowance for rotation, or if the brace is rigidly connected to the framing members. Prior to brace buckling, tensile yielding along the length of the brace is possible, but after initial buckling, most of the tensile elongation and plastic strain occurs within the plastic hinge region because of the residual stress, imperfections, and P-δ effects. As a consequence, the large strains caused by cyclic load reversal in this region cause the brace to fracture. The sequence of localization of inelastic deformation amplifies the local strains in the fracture region as illustrated for a rectangular HSS brace in the sequence of photos for **Figure 3-5**. **Figure 3-5a** shows the localized strain and deformation that occurs at the plastic hinge. After multiple inelastic cycles of strain, tearing initiates at the corners of the tube (**Figure 3-5b**), tearing progresses across the flange (**Figure 3-5c**), and fracture ultimately occurs (**Figure 3-5d**). The local strain concentration initiates at smaller deformations and is more severe in rectangular tubes than for many cross sections because the rectangular shape concentrates the local strains (**Figure 3-5**). Therefore, brace fracture occurs at smaller story drift and inelastic deformation for rectangular HSS tubes than for comparable wide flange sections. Wide flanges and other open sections do not localize the strain as quickly and as severely as rectangular tubes. Hence, wide flange braces typically provide approximately 25 % larger inelastic story drift than rectangular HSS braces prior to brace fracture if all other factors are equal.

Local slenderness (b/t) of flanges and webs of various structural shapes is also important because smaller local slenderness values delay initiation of local buckling and permit larger local strains prior to initiation of tearing. These smaller values facilitates development of larger story drifts prior to fracture or failure.

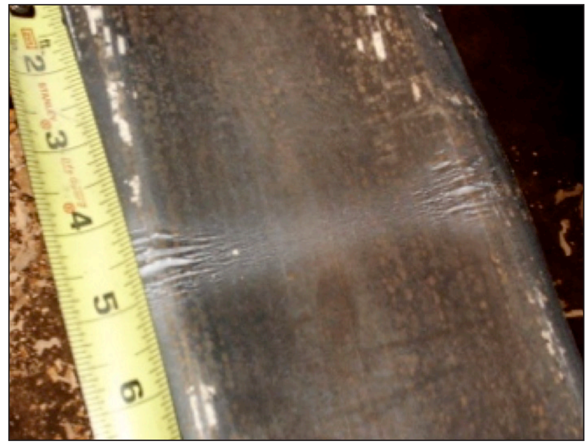
Global slenderness of the brace (Kl/r) also affects the inelastic performance. In general, braces with smaller Kl/r ratios dissipate significantly more energy through inelastic deformation prior to brace fracture than do slender braces, but stockier braces tend to fracture at a smaller story drift because the story drift is largely as result of the geometry, which causes the brace to fracture (**Figure 3-4**). Stocky braces are relatively short and require larger plastic rotation and local strain at the plastic hinge to achieve a given story drift than a longer (more slender) brace. At the same time, engineers may prefer braces with smaller Kl/r ratios because they have smaller differences between the magnitude of the tensile and compressive resistance. These goals are somewhat divergent, and so intermediate Kl/r ratios, which are not extremely small (less than about 40) nor overly large (more than about 100), are commonly used.

Secondary Strength and Stiffness

SCBFs develop most of their lateral stiffness and resistance from the axial stiffness and resistance of the brace. Recent experiments on a three-story braced frame showed that approximately 85 % to 90 % of the original elastic stiffness and resistance was provided by the bracing (Lumpkin 2009). After the initial cycle of brace buckling, the stiffness and compressive resistance of the brace are reduced, and frame action through bending of the beams and columns plays an increasing role. For a three-story frame at approximately 1 % story drift, the braces resisted approximately 75 % to 85 % of the lateral load, and at 2 % story drift, the bracing resisted approximately 60 % to 70 % of the lateral load. The brace's role decreased rapidly as brace tearing initiated. Experiments also show that the braced frame may retain 20 % to 40 % of its maximum resistance after all braces in a given story have fractured. This secondary resistance is again contributed by moment frame action developed by the beam-column and



(a) Local strain concentration



(b) Initiation of tearing

(c) Progression of tearing

(d) Fracture

Figure 3-5 – Progression of local strain to fracture for HSS brace.

gusset plate connections (Roeder et al. 2011). The specific distribution clearly depends on the specifics of the design, but this comparison illustrates the importance of the gusset plate and beam-to-column connection in developing the resistance and deformation capacity of the SCBF and the subsequent moment frame behavior developed after brace buckling and fracture.

3.3 Modes of Behavior to be Avoided

Ductile system behavior is needed to ensure good SCBF performance. Brace fracture is relatively sudden and brittle, but SCBFs are designed with the goal that it occurs only after significant inelastic deformation and ductile tearing at the plastic hinge location. Avoiding other brittle failure modes prior to developing the inelastic deformation of the system is essential.

Elements with Insufficient Ductility

Capacity-based design principles are used to design the connections of braced frames because connections are one major potential source of abrupt, nonductile failure. Although essential, this requirement does not mean that connections should be designed to avoid all yielding in the connection. As noted in the prior discussion and illustrated in **Figures 3-1b** and **3-4**, yielding in the connection is necessary and highly desirable, because it permits end rotation of the buckled brace and allows the brace to develop larger inelastic deformations prior to brace fracture and maintain consistency with brace capacity asuumed in connection design. Hence, the capacity-based design of these connections cannot be viewed as an absolute capacity measure but as a relative, balanced design criteria (Roeder et al. 2011). The connections should be designed to be stiff enough and strong enough to fully develop the brace capacity in compression and tension, but excessive strength and stiffness are undesirable in that they reduce the inelastic deformation capacity of the brace and the SCBF system.

Furthermore, the current connection design procedures do not always assure ductile behavior. As noted earlier, the welds and bolts joining the gusset plate to beams and columns are normally sized by a Uniform Force Method (UFM) equilibrium evaluation to expected tensile resistance, P_{ut}, of the brace. Recent research shows that this approach is inadequate to consistently assure ductile performance of the system (Lehman et al. 2008). The gusset plate yields because of local bending and deformation because of end rotation of the buckled brace, and this bending is in addition to the applied brace load. As a result, sudden, brittle weld fractures have been noted in a few tests where welds were designed by the UFM approach. To ensure ductile behavior of the connection, it is necessary to design the bolts or welds joining the gusset plate to the beams and columns to develop the full plastic capacity of the gusset plate rather than only the strength of the brace.

Finally, earthquake loads are inertial loads, applied to the mass of the structure. These earthquake loads are then transmitted to the SCBF frames, and therefore, the connections between the diaphragms (and other framing attached to the mass) and the SCBF must be adequate to fully transmit this force.

Story Mechanisms

Concentration of inelastic deformation in a limited number of stories may occur in braced frames, and the potential increases with increasing inelastic deformation and damage. Most structural systems concentrate damage to some extent, but braced frames are one of several systems that concentrate their deformation more readily than others. Distribution of story shear is partially dependent upon the excitation and dynamic response of the structure, but the relative stiffness of adjacent floor levels plays an important role. Large changes in story stiffness occur as braces buckle, experience post-buckling deformation, yield in tension, and ultimately fracture. These changes are major contributors to the concentration of deformation effects, and the concentration of damage tends to be more common and severe when the braced frame experiences severe inelastic deformation. However, an initial structural design that does not appropriately balance the relative stiffness of adjacent floors may compound this effect.

Capacity Design

Capacity-based design criteria are applied to the columns, column splices, and column-foundation connections because the columns support the gravity load and must have sufficient axial load capacity to fully develop the brace. Specific details of these capacity design provisions are provided in a later section. It is unacceptable to have the column fail in compression, such that it is unable to support the gravity load of the system because this would lead to potential structural collapse. However, limited yielding of the column in tension or compression will occur (**Figure 3-1c**) and may be beneficial to the overall system performance. This yielding must be limited and controlled. Tensile fracture of column splices or foundation (baseplate or attachment) failures (**Figure 3-3d**) may be quite brittle, will clearly limit the lateral resistances, and should be avoided in SCBF design.

Foundation uplift can significantly attenuate dynamic response from earthquake excitation, but the design requirements for controlling this uplift and the consequences of the uplift are far different from the design considerations commonly applied to SCBF systems. Column uplift may be tolerable in seismic evaluations of existing buildings, but it is not the goal of a new SCBF design. Column splice failure also has attributes of column uplift, but the ability to control this behavior is more difficult, and column splice failure is also an unacceptable behavior for the system.

4. Analysis Guidance

4.1 Code Analysis

Analysis of Special Concentrically Braced Frames is governed by provisions of both AISC 341 and the applicable building code, typically ASCE 7. The required minimum strengths of the braces, beams, columns, and connections are established initially through a combination of computer structural analysis for the applicable load combinations to determine the required strengths of the braces, and then through analysis that takes the form of a capacity design to determine the required strengths of the columns, beams, and connections. The following sections outline these various methods of analysis.

Analysis Requirements of ASCE 7

ASCE 7 permits three different types of analysis procedures to be used to analyze special concentrically braced frames. These procedures, outlined in Table 12.6-1 of ASCE 7, include the Equivalent Force Analysis (§12.8); Modal Response Spectrum Analysis (§12.9); and Seismic Response History Procedures (Chapter 16). The Equivalent Force Analysis procedure is the most straightforward to execute, but Table 12.6-1 lists restrictions on the use of this approach based on the structure configuration: structures having a long period or having specific types of horizontal or vertical irregularities are not permitted to use this approach. (For SCBFs, the story shear strength is taken at the sum of the horizontal components of the expected brace strengths in tension and compression; the lateral resistance provided by shear and bending in the columns is typically neglected.) Table 12.2-1 identifies the values of the seismic performance factors, R, Ω_0, and C_d, required for analysis of SCBFs.

The Equivalent Force Analysis procedure enables the use of static analysis procedures to estimate the effects of an earthquake. ASCE 7 §12.8 outlines the parameters of the analysis. An approximate procedure is provided in ASCE 7 to conservatively compute the fundamental period of vibration of the structure that is needed for this approach. This approximate period is often below the period calculated by more accurate methods, with the shorter approximate period leading to larger base shears, although the base shear has a cap as specified in ASCE 7 §12.8.1.1. As braced frames are relatively stiff structures, they are included in the category of “All other structural systems” in Table 12.8-2 to determine the approximate period. Although ASCE 7 requires that global second-order elastic P-Δ effects be included in the analysis if the stability coefficient, θ, exceeds 0.1, AISC 360 requires that second-order elastic effects be considered for all frames.

The Modal Response Spectrum Analysis accounts more directly for the dynamic performance of the structure by requiring calculation of the modes of vibration of the structure sufficient to obtain a combined modal mass participation factor of 90 % of the actual mass in each of the orthogonal horizontal directions of response.

Either Modal Response Spectrum Analysis or Seismic Response History procedures are required for structures over 160 feet in height with specific types of structural irregularities or with long periods. Both linear elastic and nonlinear Seismic Response History procedures are outlined in Chapter 16 of ASCE 7. Seismic Response History involves using numerical integration to analyze the structure for specific ground motions. A minimum of three ground motions are required. Chapter 16 outlines specific requirements of the characteristics of the ground motions and the procedures used to assess the results.

Typically, linear elastic Seismic Response History Analysis provides few benefits as compared to Modal Response Spectrum Analysis because both procedures account for the linear dynamic response of the structure, which is dominated by the lower-period vibrational modes. ASCE 7 §16.2 establishes the procedure for nonlinear analysis, including the hysteretic material nonlinear behavior of the components of the seismic force-resisting system. The hysteretic constitutive behavior of the members or connections should be consistent with laboratory testing of comparable components, including all significant yielding, strength degradation, stiffness degradation, and pinching. Strength of the elements should be based on expected mean values, including material overstrength, strain hardening, and strength degradation. The use of linear versus nonlinear analysis is discussed further below.

4.2 SCBF Modeling Issues

AISC 341 allows two beam-to-column connection types for SCBFs: simple connections and a moment-resisting connection comparable to those used for Ordinary Moment Frames. For the former, SCBFs are usually modeled as trusses with pin connections assumed in both planes, particularly for analyses related to initial design. As such, the stiffness offered by the gusset plates to the girders or columns at the brace connections are largely ignored under a presumption that they will yield relatively early during the seismic excitation. In addition, any inherent flexural resistance may not result in significantly reduced required strength of the members, and thus pin-connected and fixed-connection models generally lead to the selection of identical member sizes, including compactness requirements. Detailing for the effects of rotational restraint is addressed in Section 5.

In such models, if V-bracing or inverted V-bracing is used, the analysis should also enable modeling of flexure in the girders. While columns that are continuous across several stories typically have flexural forces induced in them because of interstory drift, flexural forces in the columns because of design story drifts may be neglected according to AISC 341 to facilitate modeling the SCBF as a truss system.

When moment connections are used, the connections should be modeled accordingly in the analysis because moment will be transferred between the girders and columns. A moment connection at the end of the brace may be used if the connection is deemed to be adequately stiff and strong.

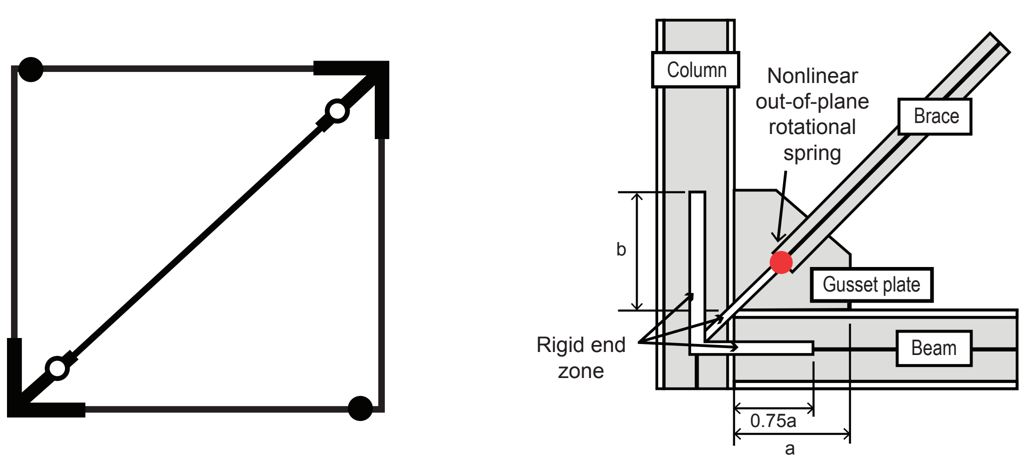
In models in which floor diagrams are assumed to be rigid, it is also important to consider the importance of isolating the seismic force-resisting system from the diaphragm so as to adequately model the axial force distribution in the girders. This isolation can be done, for example, by using gap-contact elements at each floor level to attach the braced frame to adjacent nodes that are part of the rigid floor system.

Within AISC 360, two approaches are available to account for structural stability per Chapter C: the Direct Analysis Method and the Effective Length Method. In the former, if Equivalent Force Analysis is conducted, the effective length factor of all members may be taken as 1.0, and the analysis will be based on reduced member properties coupled with the use of a notional load for load combinations that are dominated by gravity load. Using an effective length factor of 1.0 would also be common in applying the Effective Length Method to braced frames, although smaller values of effective length may be used in SCBFs that include moment-resisting connections. Although both strategies use an effective length factor equal to or less than 1.0, the Direct Analysis Method typically provides force distributions that are more commensurate with those expected at incipient instability of the frame. More information can be found in the AISC *Seismic Design Manual*.

For calculating design story drifts, typically the nominal properties of each member should be used, rather than the reduced properties outlined for the Direct Analysis Method. It is assumed that the deflection amplification factor, C_d, accounts for such variation.

For common steel buildings under four to five stories, these models typically have sufficient accuracy to capture elastic forces and deformations. However, braced frame models that assume truss behavior and pin connections may often underestimate stiffness and thus overestimate deflections and period, particularly for taller structures. Where such response is important to model more accurately, it is common to use more refined elastic models that account for the stiffness inherent in typical braced frame connections, including the stiffness of the shear tabs, gusset plates, and other key components. Hsiao et al. (2012, 2013a) provide recommendations for appropriate modeling assumptions that include rigid links in the connection region to model the enhanced stiffness in the region. Such modeling recommendations (**Figure 4-1a** and **Figure 4-1b**), identify a combination of rigid links, pin connections, and nonlinear springs to provide an assemblage model that compares well to experimental tests of SCBF subassemblages.

When brace buckling occurs at larger load levels, deflections start to increase more significantly, leading in turn to significant yielding in the connections. Ductility in the connection design is thus essential, and nonlinear analysis and associated models may be used to develop more accurate predictions of behavior when needed (Hsiao et al. 2012).



(a) SCBF panel configuration with rigid links, pin connections, and nonlinear spring (Hsiao et al. 2012)

(b) Geometric details identifying typical link lengths and nonlinear spring location (Hsiao et al. 2013a)

Figure 4-1 – Schematic structural model of SCBF panel.

4.3 Limitations of Elastic Analysis

As discussed earlier, the post-elastic response of concentrically braced frames typically entails not only magnitudes of force larger than the elastic limits of elements but also modes of behavior markedly different from those of the elastic structure. Thus, magnifying elastic forces by a constant factor can be insufficient to capture demands on many structural elements.

Typical elastic analysis of the various configurations of SCBFs sometimes yield members with little or no force, such as the center column in a two-bay braced frame or in the beams at the mid-story of a two-story X-configuration for bracing. In part for these reasons and also to ensure that the progression of damage in the SCBF is appropriate for large loadings, AISC 341 requires a plastic mechanism analysis leading to a capacity design approach. Thus, although it is appropriate to use elastic analysis for determination of the brace forces, which are the ductile elements in SCBF, it is important to use capacity design procedures to investigate the possible plastic mechanisms to determine the required strengths of the columns, beams, or connections.

4.4 Plastic Mechanism Analysis

Frame

AISC 341 §F2.3 permits each of the analysis procedures outlined in ASCE 7 to be used for analysis of SCBFs to obtain the required strengths in the braces. The required strengths from these analyses may then be used directly for design of the braces. For computing the required strengths in the columns, beams, and connections, load combinations appropriate for use in the static analysis procedures must be taken as the larger determined from the following two analyses:

- an analysis in which all braces are assumed to resist forces corresponding to their expected brace strength in compression or tension, representing the elastic limit of the frame

- an analysis in which all braces in tension are assumed to resist forces corresponding to their expected tensile strength, and all braces in compression are assumed to resist their expected post-buckling strength, representing potential conditions after some braces have buckled and lost significant compression strength and stiffness

The expected tensile strength of the brace may be taken as $R_yF_yA_g$. The expected compression strength of the brace may be taken as the smaller of $R_yF_yA_g$ and $1.14F_{cre}A_g$, where F_{cre} is the critical buckling strength determined by Section E of AISC 360 using an expected yield stress of R_yF_y and 1.14 is computed by removing the out-of-straightness parameter (1/0.877 = 1.14). The post-buckling strength of compression members may be taken as 30 % of the expected brace strength in compression.

Expected brace strengths are determined while maintaining consistency with the assumed brace capacity used to design the gusset plates and the mechanics of the actual connection (i.e., effective length factor). These analyses ensure that any force imbalances that are created by a tension brace and a compression brace intersecting at a work point in a connection region or the center of a beam span are accounted for. Together, this analysis procedure explores the different possible plastic mechanisms that are likely to form in an SCBF during large earthquakes, specifically mimicking the potential force patterns that would arise because of first-mode behavior. A typical expected behavior is shown in **Figure 4-2**. The behavior addressed in the first bullet is intended to account for the initial full compression force in the brace (its expected strength) that would occur during the first excursion at large lateral drifts while the behavior addressed in the second bullet is intended to account for the distribution of forces after the strength in the brace has been significantly reduced (assumed to be reduced by 30 % in AISC) after repeated cycles of significant seismic loading. The resulting forces for design are typically augmented by superimposing any forces obtained from an analysis for corresponding gravity loads. When the frame is subject to inelastic drift, the braces no longer are as effective in resisting gravity forces, which thus must be carried by the columns. Plastic-mechanism analysis addresses this redistribution from the elastic distribution of gravity forces. Where braces carry significant gravity forces in the elastic condition, the use of the elastic analysis for design of columns may require adjustment. This may be achieved by performing a separate gravity analysis without the braces and combining column forces from that analysis with ones from a lateral-load analysis that includes the braces.

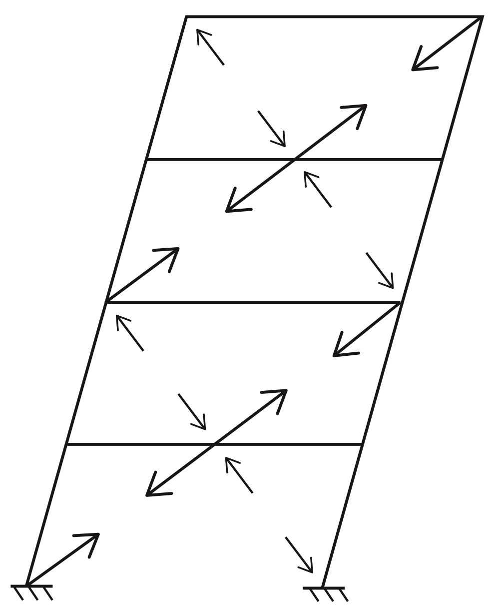


Figure 4-2 – Schematic of typical first-mode behavior of SCBF as assumed in plastic mechanism analysis for an X-brace configuration (AISC 2010a).

Plastic Mechanism Analysis in Design Practice

Many general-use analysis programs are not configured to perform analyses of the system outside of the elastic range. Designers have developed numerous "work-around" solutions to perform the required analysis (and thus obtain forces for beam and column design) using the tools available. Such methods include the following:

- use of spreadsheets to calculate brace capacity forces, as well as the vertical and horizontal components
- use of nonlinear analysis software to perform a "pushover" analysis (requires nonlinear brace element modeling, and possibly nonlinear column flexural modeling)
- use of elastic analysis programs with braces removed and brace capacity forces imposed (may require artificial lateral restraint for model stability)
- as a variant on the item above, substitution of low stiffness, high coefficient of thermal expansion material for brace elements, with temperatures imposed causing stresses corresponding to expected yield stress and expected buckling stress.

In any case, a separate gravity load analysis with the braces removed is required.

Collectors

ASCE 7 requires that floor and roof diaphragms be designed for both in-plane shear and bending stresses resulting from the analysis. Openings and edge conditions should be accounted for to ensure the shear and tension strength of the diaphragm are not exceeded. Collector elements should be provided and designed for the axial, flexural, and shear forces needed to transfer seismic forces through the structure to the special concentrically braced frames. Section 12.10 outlines the cases in which the overstrength factor should be used to ascertain the required loads for analysis. For special concentrically braced frames, the overstrength factor, Ω_0, from Table 12.2-1 in ASCE 7 equals 2. Additional discussion can be found in NEHRP Seismic Design Technical Brief No. 5 "Seismic Design of Composite Steel Deck and Concrete-filled Diaphragms" (Sabelli et al. 2011).

4.5 Nonlinear Dynamic Analysis Guidance

Nonlinear response history analysis may be used to verify the design forces in SCBFs, such as in cases where design efficiencies may be achieved through more accurate analysis, particularly for long-period structures or structures with irregularities. NEHRP Seismic Design Technical Brief No. 4 "Nonlinear Structural Analysis for Seismic Design" (Deierlein et al. 2010) presents guidance on conducting nonlinear structural analysis, including outlining different types of element formulations (e.g., concentrated plasticity versus fiber-based distributed plasticity formulations) and hysteretic constitutive formulations that are appropriate for SCBFs. The document includes guidance on modeling foundation systems, mass, and damping in structures, as well as advice on selecting appropriate ground motions. The document also provides recommendations for interpreting the results of the analysis. Of particular importance for braced frames is using an accurate model for the brace that includes inelastic yielding in tension, inelastic flexural buckling in compression, as well as the successive strength degradation that occurs because of repeated buckling in subsequent cycles of loading. Typically, the inelasticity in a brace occurs at the ends of the brace and at midspan, and so ensuring that the modeling enables inelasticity at these locations is important and required for directly modeling out-of-straightness in the plane of buckling. The columns and girders should also include proper plasticity models that account for combined flexure and axial force. Modeling the inelastic response of the connection regions, including inelastic rotations and gusset plate buckling, provides enhanced capabilities for accurately modeling the progression of damage in SCBFs. Gravity loads are applied to the frame prior to initiation of the response history analysis.

5. Design Guidance

5.1 AISC Design Procedure

AISC 341 seismic design provisions require significant ductile detailing for SCBFs. The current design method is to establish the factored force demands from ASCE 7 on members of the system and to use the AISC 360 Load and Resistance Factor Design design provisions to size the brace. Other framing members are initially sized to these factored load demands. There are several additional requirements for this member selection process:

- Satisfy limitations on bracing configuration as described in Section 3 of this document.
- Satisfy local and global slenderness limits for the brace, beam, and column.
- Design of beams in frames with V-bracing and inverted V-bracing to sustain the vertical unbalanced load that results after brace buckling.
- Design of columns and column splices to resist the maximum expected force delivered by the inelastic braces to the columns based on the plastic mechanisms discussed in Section 4.

After the initial member selection is complete, capacity design concepts are used for connections and critical members to ensure that the braces can develop their required inelastic deformation. The required resistance for the capacity-based design is the expected inelastic capacity of the brace in tension and compression (i.e., $P_{ut}=R_yF_yA_g$ in tension, and $P_{uc}=1.14F_{cre}A_g$ in compression), where, A_g is the gross cross-sectional area of the brace, R_y is the ratio of the expected yield strength to the minimum specified yield strength, F_y, and F_{cre} are the critical stress associated with brace buckling considering the expected material strength. The connections joining the brace to the frame must be designed to maintain their integrity even as the brace undergoes cyclic buckling and yielding. Numerous design requirements relate to tensile demands on connections:

- The net section resistance of the brace and gusset plate and resistance of the bolts and welds joining the brace to the gusset plate must exceed P_{ut}.
- The gusset plate thickness and the length of the brace-to-gusset interface must be sufficient to preclude block shear rupture for P_{ut}.
- Bolts or welds joining the gusset plate to the beam and column must have sufficient strength to resist force demands corresponding to the expected strength of the brace.

For compression, the requirements include a stability check for the maximum compression force the brace can deliver (e.g., its expected buckling load). Additionally, the connection must allow the brace to undergo flexural buckling without harming the connection. This is done either by designing the connection to resist the full flexural capacity of the brace or by detailing the gusset plate connection to permit flexure of the gusset plate while maintaining axial force resistance. The AISC *Seismic Design Manual* provides several illustrations of the application of member design and connection design for SCBFs.

5.2 Layout

SCBF buildings should be planned considering the post-elastic behavior of the system. Designers should consider the reduction in frame stiffness resulting from brace buckling and should mitigate potential detrimental behaviors.

Specifically, a frame changing from its elastic state to its post-buckled state may simultaneously introduce a large eccentricity between the center of mass and center of rigidity while also reducing the building torsional resistance. These effects may be mitigated by providing frames with high secondary stiffness (such as those with slender braces and high overstrength) and by providing a high degree of redundancy. (Slender braces, sized for their compression capacity, have inherently high overstrength because of the ratio of R_yF_y to the critical buckling stress, F_{cr} .)

Similarly, the different behavior of braces in tension and compression should be understood, and designers should endeavor to use braces in opposing pairs to avoid asymmetric building resistance.

At the limit of lateral drift capacity, the majority of the lateral resistance is due to the strength of braces in tension. Braces in compression may have lost a great deal of resistance because of elongation in previous cycles, local buckling, and transverse displacement. A sufficient load path must be provided considering this limit state condition. From a design point of view, providing braces in opposing pairs near each other minimizes the difference between elastic and limit state load paths and thus reduces the likelihood that the designer will overlook this effect.

Figure 5-1 shows a frame with opposing diagonals separated by two bays. The figure also shows beam axial force diagrams assuming elastic brace behavior and limit state brace behavior. (For clarity, the strength of braces in compression in the limit state condition is shown as zero.)

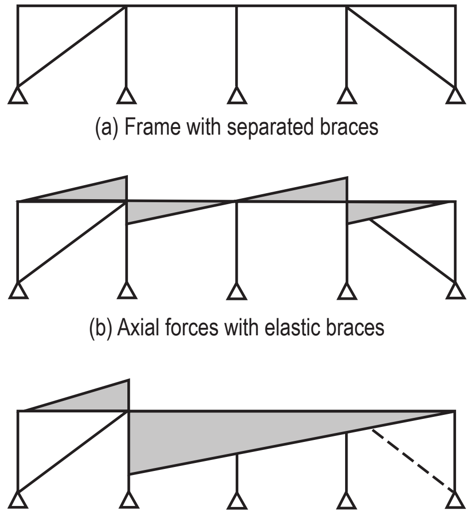


Figure 5-1 – Frames with opposing diagonals in non-adjacent bays.

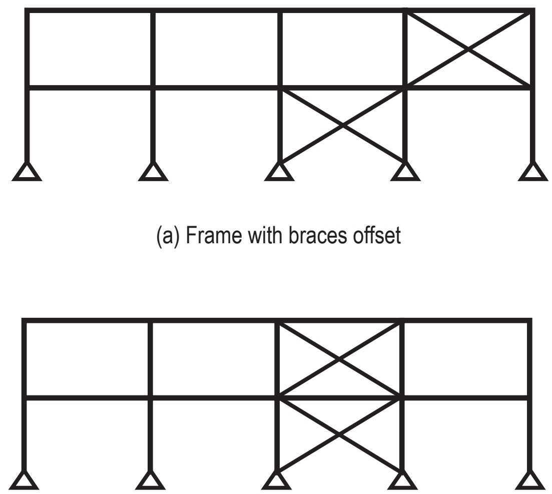


Figure 5-2 – Multi-story braced frames with and without in-plane offsets.

5.3 Configuration

Braces are typically used in opposing pairs within a bay or in adjacent bays because of the different behavior in compression and tension. Where a single diagonal is used, there is a marked tendency toward accumulation of inelastic drift in the direction corresponding to brace compression (Khatib et al. 1988). Frame configuration can affect building performance. Stacked V and inverted-V frames are somewhat less effective due to their post-elastic flexibility. The requirements in AISC 341 for the design of the beam to resist flexural forces in the post-elastic condition (with one brace buckled and the other with yielding in tension) does not ensure a high post-buckling frame stiffness. Beam flexibility may lead to a flexible condition and concentration of drift demand. The beam may also have a simple connection to the columns thus reducing the secondary stiffness once the brace buckles.

Cross-braced frames tend to increase brace rotation requirements in flexural buckling because of the increased number of connections and the corresponding reduced buckling length. Additionally, they are less economical as each connection must have the strength to resist the tensile capacity of the brace.

It is sometimes convenient to use several braced bays rather than a single stacked bay to reduce overturning demands. **Figure 5-2** shows two frames; the frames may use the same braces but frame (a) will have lower column and foundation forces. Designers must be sure to consider the complete load path for both the elastic and post-elastic conditions as described in the section above; frame (a) will have higher beam and connection forces at the discontinuity.

Design story drifts determined in accordance with ASCE 7 for tall, slender SCBFs can be dominated by global flexural deformation of the frame. End column sizes can be increased, sometimes above that required for strength, to counteract this drift contribution.

5.4 Proportioning

Proportioning of frames is fundamental in achieving adequate performance. Braces must be the fuse in the system. Beams, columns, and connections must be sized considering the brace expected strength, rather than the brace forces corresponding to the design base shear of the building. Thus, overstrength in the braces is beneficial only if there is matching (or greater) overstrength in the other elements. Where braces are oversized without a corresponding adjustment in framing member strength, the resulting frame will not be well proportioned and may not provide significant ductility.

5.5 Braces

Engineers must select brace types from a range of material and shapes. From a design point of view, braces are selected based on their compression strength. As such, HSS tend to have advantages from an economic point of view. However, as discussed in Section 3, the fracture life of wide-flange braces and of some other shapes is somewhat greater.

An additional advantage of wide-flange A992 material is that it is currently better controlled in terms of expected strength. HSS A500 material has a disadvantage in that its expected yield stress (R_yF_y, as defined in AISC 341) is higher than its specified minimum yield stress (F_y, as defined in AISC 360).

As braces are sized using design strengths based on specified minimum yield stress and bracing connections and as other elements are required to be designed to resist forces based on the expected yield stress, the ratio of the two (R_y) is an index of efficiency, with high values indicating neglected strength. Furthermore, A500 material is available in multiple grades, and it is possible that members are certified for multiple grades, adding a degree of overstrength that may not be accounted for in the design. Such overstrength in the brace is detrimental if it creates a condition in which connection rupture occurs prior to brace yielding.

Another preliminary decision that must be made in the design of braced frames is the selection of the plane of buckling. This is achieved through a combination of member selection and connection detailing. The section type may have similar or identical properties in its two transverse axes (as is the case for square and round shapes) or a distinct difference, creating a strong axis and a weak axis (as is the case for rectangular or wide-flange sections). In combination with this, end connections may provide different degrees of rotational restraint in each axis. Together, the section properties and end restraint can be used to favor buckling in the plane of the frame or perpendicular to that plane.

There is no inherent structural advantage for buckling in one plane compared to the other. However, the anticipated brace transverse displacements must be accommodated without impacting adjacent building components and causing unsafe conditions, such as falling hazards or blocked egress. This may cause different detailing challenges for different planes of buckling. Singly-symmetric and mono-symmetric sections are not typically used in SCBFs due to the coupling of flexural and torsional buckling modes. The effect of flexural-torsional buckling on gusset connections is not well understood.

5.6 Connections

The SCBF system was developed with the intention of maximizing the inelastic drift capacity that could be obtained based on brace buckling and yielding while maintaining lateral resistance. As such, connection rupture is to be avoided. This is achieved by requiring connections to be designed for forces corresponding to the expected strength of the brace as it undergoes inelastic axial deformations (yielding in tension and buckling in compression.)

In tension, the required strength of the connection is the expected yield strength of the brace, including material overstrength. This required strength applies to all limit states, including local limit states within the brace itself.

For bolted connections there is a reduced section through the bolt holes. For welded slotted brace connections, there typically is a reduced section caused by the slot. Additionally, the connection configuration may necessitate the consideration of an effective net section smaller than the net section because of shear-lag effects.

In the above cases, the brace section requires reinforcement to avoid net-section rupture at low drift levels. This reinforcement serves as a bypass to reduce the stress at the critical section. Thus the reinforcement is most effective close to the plane of the brace-to-connection force transfer. If the reinforcement must be located away from that force transfer, the calculation of the effective net section of the reinforced section reveals the reduced efficacy.

AISC 341 and AISC 360 do not specifically discuss the appropriate method for determining the force transfer from brace to reinforcement and back from the reinforcement to the brace. Practice has been to fully develop the strength of the reinforcement via welds, as illustrated in the AISC *Seismic Design Manual*. Other approaches may be considered as well.

Reinforcement material should have strain compatibility with the brace material. Higher-strength material may be used as reinforcement, but its full strength may not be realized at the limit state of net-section rupture. Lower-strength material may be used, but strains may reach the yield level earlier in the reinforcement than in the brace.

Brace connections must also resist large compression forces corresponding to the brace compression capacity. This compression capacity is lower than the expected tension yield strength, but only modestly so for braces of low slenderness. For intermediate and slender braces, AISC 341 provides a modified expected compression strength based on expected material strength and uses an adapted formula to remove some of the conservatism appropriate for determining the lower-bound member design strength.

Where gusset plates are used as part of the bracing connection, their compressive strength may be determined using a variety of methods. Dowswell (2006) provides specific methods appropriate for different gusset restraint conditions.

Brace flexural buckling entails the formation of three concentrated points of rotation as the brace axial length decreases corresponding to large seismic drifts. Braces form plastic hinges at the mid-length between connections and at each end. These end rotation points are in the brace itself if the end connection's flexural strength exceeds that of the brace. Conversely, if the brace flexural strength exceeds that of the connection, the concentrated rotation demand will be in the connection itself.

Minor in-plane eccentricity may be included in the layout of the connections to reduce connection size or make the geometry of the joints easier for fabrication and erection. Such eccentricities result in flexural forces in beams and columns; these flexural forces are determined using forces

Detailing for Buckling

Where connections do not have the flexural strength in the plane of buckling to force rotation corresponding to brace buckling to occur in the brace, the connections must be detailed to accommodate significant rotations.

Several approaches to providing rotation capacity in the connection have been proposed and verified through testing. One common approach is to provide a hinge zone in a single gusset plate. This approach was developed based on research by Astaneh-Asl et al. (1986). This hinge zone is oriented perpendicular to the brace axis, with a minimum width (in the direction of the brace axis) of twice the gusset thickness (t_p). No stiffeners or other restraint should intrude into this zone and thus hinder the free rotation. Rotation occurs out of the plane of the frame in this connection.

Recent research has shown that there are substantial limitations with the current connection design methods (Lehman et al. 2008; Roeder et al. 2011). The $2t_p$ linear clearance method provides relatively compact plates with tapered gusset plates, but when used with rectangular or minimally tapered gussets, the method leads to relatively thick gussets. The thickness of these large gusset plates is typically controlled by gusset plate buckling, and the increased thickness of the gusset increases the rotational restraint at the end of the brace and decreases the inelastic deformation capacity achieved prior to brace fracture.

More recently, an approach based on the use of thinner gussets and an elliptical hinge has been developed, based on work by Roeder et al. (2011). This approach traces an elliptical hinge zone on the gusset and allows the brace to extend nearer to the beam and column members. The elliptical hinge zone is eight times the thickness of the gusset. Simple geometric formulas are used to establish gusset dimensions and stand-off distances. Rotation occurs out of the plane of the frame in this connection as well. Connections designed with these clearance models are likely to be controlled by block shear and tensile yield. The gusset plates are smaller and thinner, and frequently provide increased inelastic deformation capacity of the SCBF system.

A third alternative provides rotation in the plane of the frame by means of introducing a knife plate perpendicular to the gusset plate. Rotation occurs in the knife plate in a hinge zone that is three times the plate thickness.

Figure 5-3 shows these three types of details configured to provide brace buckling rotation capacity.

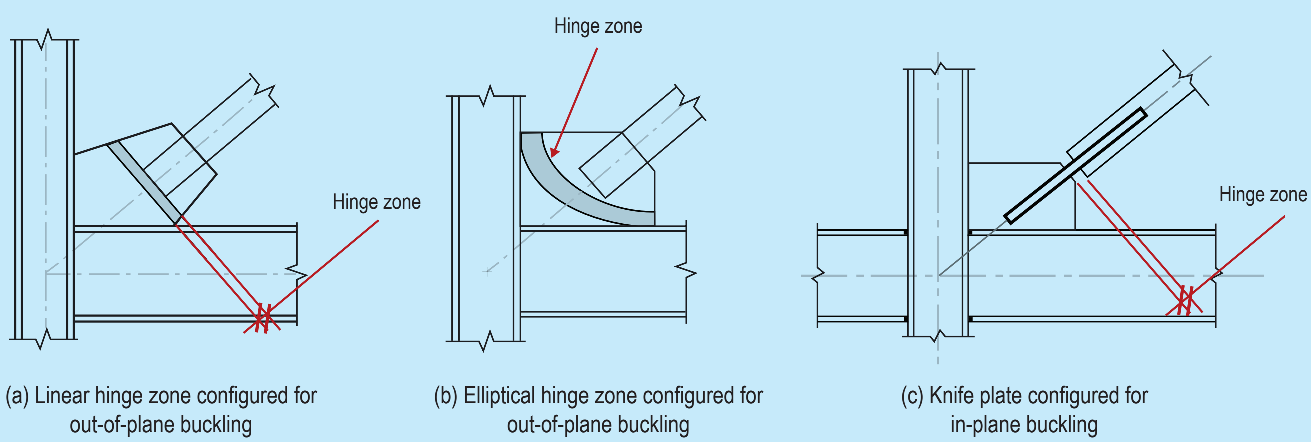


Figure 5-3 – Details configured to provide brace buckling rotation capacity. The AISC Seismic Design Manual has examples of each of these types.

corresponding to brace expected tension and compression strengths. The design of beams and columns to resist these flexural forces, in combination with axial forces corresponding to brace expected tension and compression strengths, ensures that the primary source of inelastic drift capacity is brace buckling and tension yielding.

5.7 Gusset Plate Design Methods

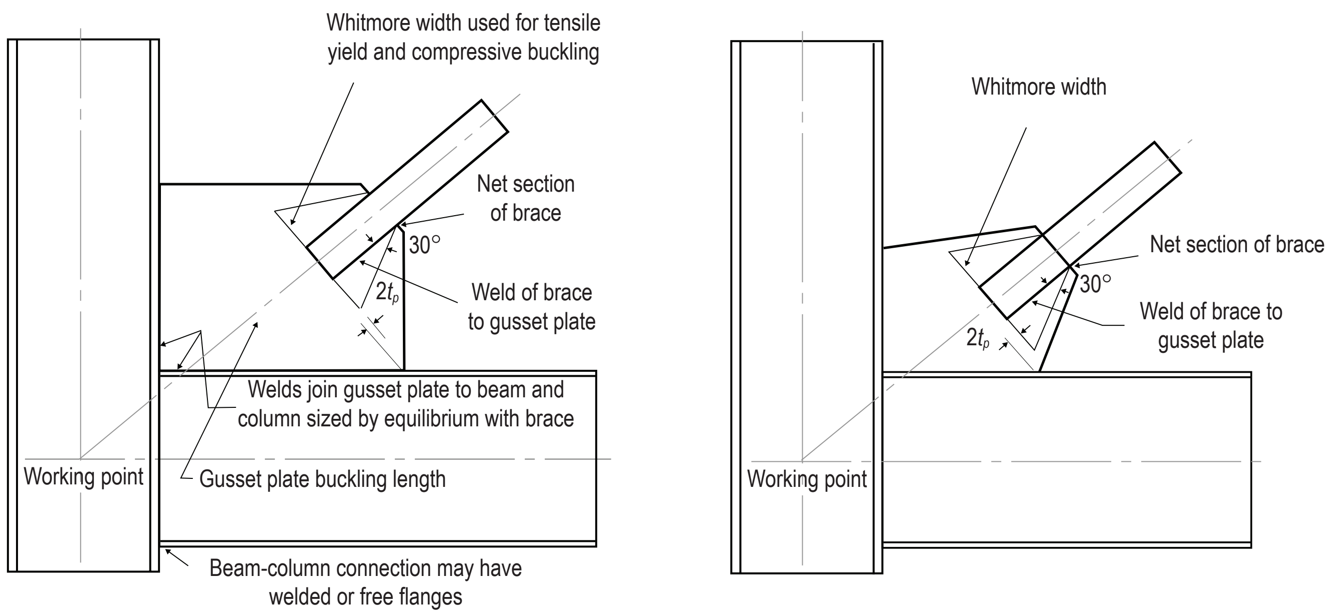
Current Design Method

As noted earlier, the gusset plate connecting the beam, column, and brace must be designed by capacity design procedures such that its resistance exceeds expected inelastic capacity of the brace in tension and compression. A number of limit states must be checked to verify sufficient strength. A complete illustration is found in the AISC *Seismic Design Manual*.

The thickness of the gusset plate must be sufficient to resist both the brace expected tension strength and to resist buckling when subjected to the expected brace compression strength. In practice, this may lead to thick gusset plates. Additionally, gussets are configured to accommodate brace buckling, as discussed above.

Beam-column connections must be designed to be consistent with gusset plate design. That is, any forces transferred to the beam by the braces through the gusset plate design or delivered to the braced bay by drag struts or other structural elements must be considered in the design of the beam-column connection. Typically, welds joining the gusset plate to the beam and column are sized to resist forces derived from the expected brace strength using a gusset analysis method. The most commonly employed method is the UFM, as illustrated in the AISC *Seismic Design Manual*.

The geometry of the gusset plate may be rectangular or tapered (**Figure 5-4**) with typical design checks noted. The tapered gusset plate (**Figure 5-4b**) has similar checks but different geometry.



(a) Rectangular gusset plate

(b) Tapered gusset plate

Figure 5-4 – Failure mode design checks for gusset plate connections.

Recommendations for Improvement of Gusset Plate Connection Design

Recent research has shown that welds joining the gusset plate to the beam and column should be sized to develop the expected yield capacity of the gusset plate rather than sized to requirements of the UFM (Roeder et al. 2011). Crack initiation invariably occurs in gusset plate connections because of substantial deformations experienced by the gusset plate, but the weld cracks remain stable if the welds are sized to these criteria and if demand-critical weld material is used. Welds that do not have sufficient strength to develop the strength of the gusset plate have experienced dramatic brittle fractures at small frame deformations, regardless of whether they have sufficient strength to resist the expected brace yield strength.

As noted earlier, gusset plates are designed to the expected load capacity of the brace, and these design forces are much larger than the factored loads required of the design. Prior research has recommended a balanced design procedure that permits limited inelastic deformation in the gusset plate and increases the inelastic deformation capacity of the SCBF system. In particular, limited yielding in the connection is encouraged by liberalization of the block shear, net section, and tensile yield design criteria (Roeder et al. 2011). The combination of these recommended improvements has been shown to substantially increase the inelastic deformation capacity of the SCBF system prior to initial brace fracture. These recommendations require variations from current AISC seismic design provisions, because more liberal stress levels are recommended for designing the gusset plate for tensile yielding over the Whitmore width and for block shear.

These recommendations are rational because the design loads for the gusset plate are typically two to three times the factored design loads for which the resistance factors were developed. This recommended procedure encourages thinner, more compact gusset plates that facilitate end rotation caused by buckling of the brace. Current design methods frequently encourage larger, thicker gusset plates, which provide greater restraint to the end rotation and reduce the inelastic deformation capacity of the brace prior to brace fracture. This can cause earlier and more severe local damage to the beams and columns adjacent to the gusset plate. This increased local damage increases the repairs required after earthquakes and reduces the performance advantages of the SCBF system (Yoo et al. 2008a and 2008b). The gusset plate will yield because of the deformations caused by brace buckling, regardless of the conservativeness of its design. Conservatively designed gusset plates are usually significantly thicker than the webs of beams and columns. The stress in the gusset plate resulting from its deformation is transferred to the webs of the beam and column, and significant local inelastic deformation must be expected in the beam and column if the web is significantly thinner than the gusset plate (Palmer 2012). This reduced beam and column damage during earthquake loading reflects one of the major benefits of the proposed balanced design procedure.

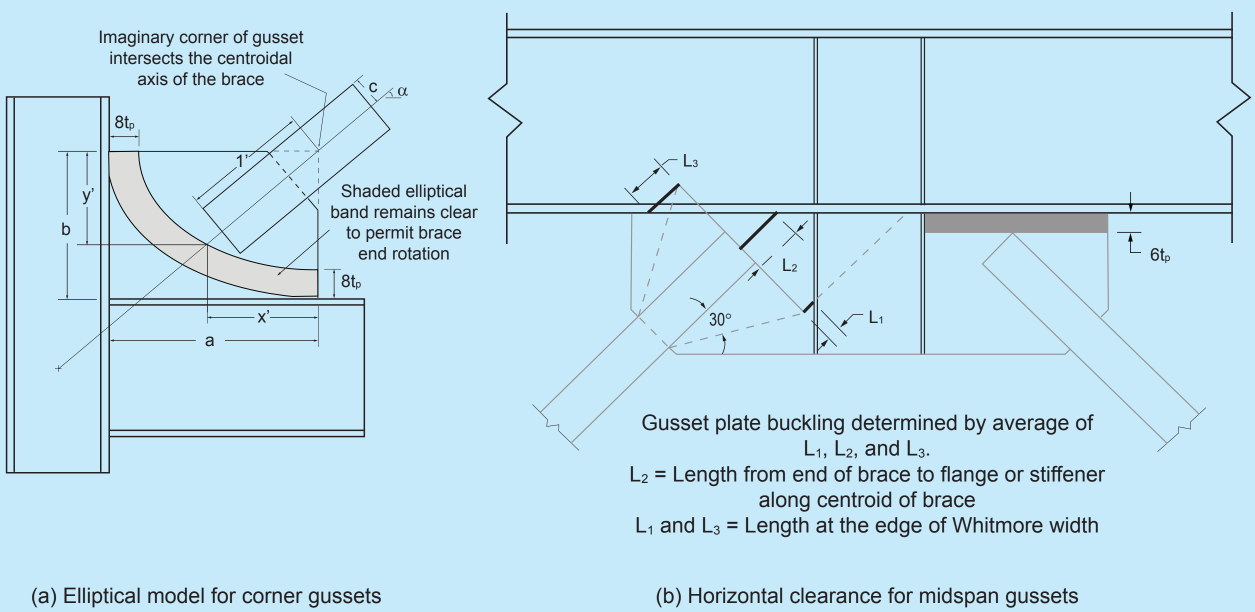


Figure 5-5 –Improved Gusset Plate Clearance Models.

5.8 Frame Deformations

Bracing connections must also be designed to accommodate the large inelastic drifts that the frame is expected to undergo. This is problematic where gusset plates connect to both the beam and the column, forming a haunch. AISC 341 requires that attention be given to this configuration. The connection consists of all of the connected pieces: the beam, the column, the brace, and the gusset. The stiffening effect of the gusset in the assembly is significant.

AISC 341 allows two approaches to accommodating drifts in these connections: providing rigid connections of sufficient strength; and providing connections capable of sufficient rotation. Within the first approach, the designer has two options to accommodate the demands that large drifts impose on these beam-column-gusset assemblies. In the first option, the connection is designed as rigid. It is assumed that at large drifts, flexural yielding of the beam or the column will have occurred and that the connection is designed to resist moments corresponding to the flexural yielding of the beam or the column. In this method, it is implicitly assumed that the brace contributes negligible flexural demand or resistance to the connection.

In the second option, the rigid-connection method, the designer may explicitly evaluate the effects of this flexural yielding on the connection components and joints. Alternatively, AISC 341 considers a beam-to-column moment connection meeting the requirements for Ordinary Moment Frames to be sufficient for these moments. The addition of the gusset plate is assumed to strengthen the connection and thus the assembly is deemed to comply with the rigid-connection requirements.

Web Proportioning

Recent research indicates that web failures in rigid connections may not be precluded by use of current design methodologies (Lumpkin 2009). While the local web strength limit states and design strengths in the AISC Specification are adequate for many conditions, in this case the forces normal to the flanges calculated based on brace axial forces do not adequately capture maximum demands on beam and column webs. In some conditions, frame deformations cause the gusset plate to act as a haunch to deliver significant moments across the connection. These forces may reach the level of gusset yielding. Where webs are much thinner than gussets, failures in the web may result. Palmer (2012) provides a basis for providing a web at least 3/4 of the gusset plate thickness (or, conversely, limiting the gusset thickness to 4/3 the web thickness).

The second approach to accommodate the rotational demands on these assemblies is to provide a connection that allows significant relative rotation between elements. This rotation demand is defined by AISC 341 as 2.5 %.

The rotation demand can be accommodated by providing flexible joints between the beam and column and between the gusset and column, provided that the rotation demand, projected over the connection depth, can be accommodated in the joints. Alternatively, a flexible splice in the beam can be provided to allow for relative rotation of the beam and the connection, which moves rigidly with the column. In this latter approach, the rules for simple connections in AISC 360 may be applied to ensure rotation capacity. Because of the large horizontal forces that typically must be resisted in these connections, typical simple connections are rarely adequate, although some of the principles employed to allow inelastic rotation, such as ensuring that ductile modes govern, are applicable to this connection design option. The AISC *Seismic Design Manual* shows such an approach.

It is generally recognized that fixed connections provide the system with beneficial strength and stiffness. However, they may also be more subject to unfavorable behaviors that limit drift capacity. Designers wishing to provide secondary strength and stiffness may consider providing it in adjacent bays. Often, welded flanges are used to resist large collector forces; moment connections in collectors can serve the dual function of providing axial and flexural strength.

5.9 Base Connections

Base connections are often modeled as pinned for design because the flexural resistance may not result in a significantly reduced required strength of the members, and thus, pinned-base and fixed-base models generally lead to the selection of identical member sizes. Nevertheless, typical connections of braces to the column-to-base assembly provide high flexural stiffness and may have limited rotation capacity. With relatively little research guidance on the behavior of these assemblies, designers must rely on largely untested methods to achieve the required drift capacity.

Moderate rotation capacity can be achieved through one of the following methods (or a combination thereof):

- elongation capacity of ductile anchor rods
- foundation rotation
- column inelastic rotation

Base plate flexibility may also provide rotation capacity, but it may be incompatible with providing the required tension and compression strength.

The anchorage of large tensile forces, as required for SCBF base assemblies, generally falls outside the bounds of ACI 318 Appendix D and its supporting research (ACI 2011). An effective approach to designing for large tension anchorage forces is to embed a plate similar to the base plate deep enough into the foundation such that punching shear resistance is sufficient to resist the force. Embedded base plates are most effective when they are located beneath the bottom reinforcement of a pile cap or spread footing. **Figure 5-6** shows a base connection with an embedded plate (note the thickening of the foundation below the embedded plate).

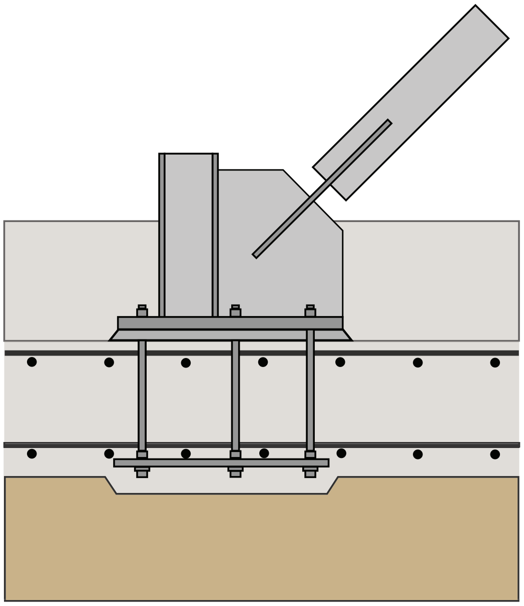


Figure 5-6 – Base connection with an embedded plate.

Another important requirement of base connections is the capacity to transfer horizontal forces between the brace and the foundation. Practice varies substantially in this regard, and little research is available to provide guidance on the efficacy of methods employed.

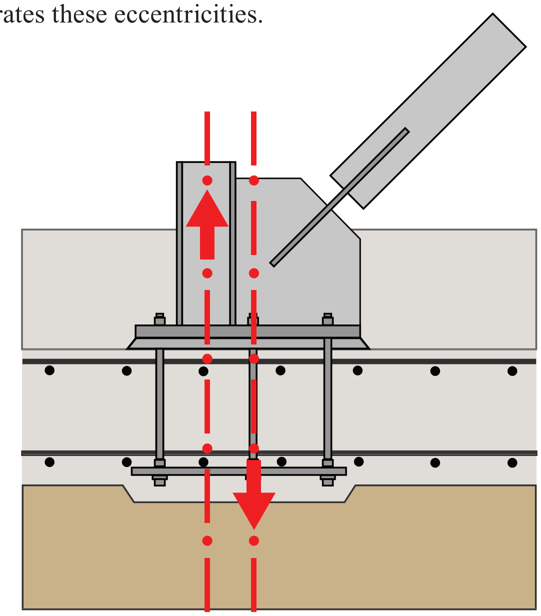
Shear methods that may be considered include the following:

- anchor rods in shear (combined with tension)
- reinforcement parallel to the direction of the shear force welded to the base plate or gusset
- shear lugs below the base plate
- shear studs below the base plate
- bearing on concrete (for a base plate and column embedded into the foundation or a slab above it)
- added horizontal members resisting horizontal forces and transferring them into the foundation away from the base plate

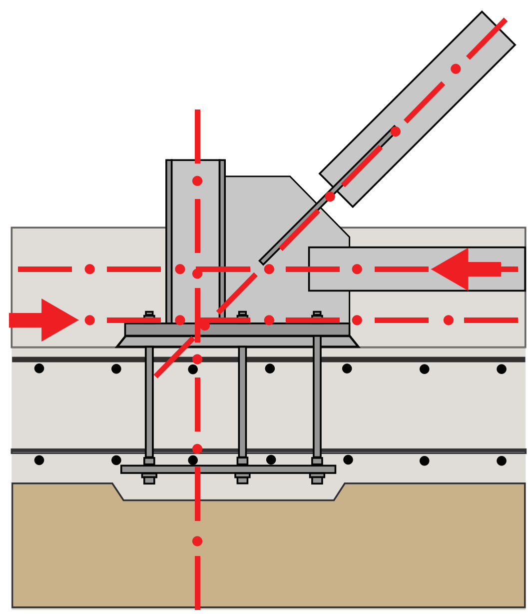
In all of these methods, the load path of the horizontal forces into the foundation and soil must be followed, and any forces resulting from eccentricity must be considered.

There are two common eccentricities that affect the distribution of design forces at base plate connections. The first is the eccentricity of the base plate with respect to the column. Base plates are typically extended for attachment of the gusset plate on one side. If the anchor rod group is centered on such a gusset plate, there is a significant eccentricity between the centroid of the group and the column centerline.

The second potential eccentricity occurs in conditions in which the horizontal force resistance is provided at an elevation other than the elevation of the intersection of brace and column centerlines. Many of the shear-transfer mechanisms discussed above can have such eccentricities. **Figure 5-7** illustrates these eccentricities.



(a) Anchor rod eccentricity for vertical forces



(b) Shear transfer eccentricity for horizontal forces

Figure 5-7 – Potential eccentricities at base plate connections.

Designs may have both of these eccentricities. **Figure 5-7b** shows the eccentricities result in opposing moments. Designers may minimize the moment by adjusting work points.

5.10 Midspan Connections

Midspan connections, in which two or more braces at a level connect to the beam at or near the center of the bay, are in many respects simpler than other bracing connections. As AISC 341 specifies the forces to be assumed in each brace in the design of the connections, the midspan connection is statically determinate. The two-story-X condition, in which a pair of braces above the beam and a pair of braces below the beam all come to the midspan connection, does not add complexity, and most limit states may be considered independently for connections above and below the beam unless the adjacent story heights are considerably different.

Midspan connections may rely on local beam web shear to transfer forces. As such, thin webs may be problematic. If the beam web-shear strength is insufficient for the portion of the vertical component of the brace force that the connection analysis assigns to the web, the web may be reinforced or the brace work points may be adjusted. This latter approach reduces the connection size but results in moment in the beam. It is generally advantageous to consider these forces in the selection of the beam.

6. Additional Requirements

6.1 Mixing Bolts and Welds

Most SCBF connections employ welded joints between the gusset plate and the beam and the column. However, a wide variation in beam-column connections has been employed. AISC 341 seismic design criteria expressly prohibit the use of combined bolts and welds to resist any force across a given interface. This is rational because bolts and welds resist load differently. Bolts may resist load with no slip and minimal deformation until friction on the faying surface is overcome. However, friction is highly variable, and joint slip occurs after friction is overcome. Welds resist loads with virtually no deformation. Bolts and welds may work together prior to initial bolt slip, but seismic loads require large inelastic deformation, and slip is probable. As a result, bolts and welds cannot reliably work together at these deformations, and load sharing is prohibited.

Strict interpretation of this rule severely limits or prohibits many braced frame connections (**Figures 6-1a and 6-1b**), because the shear and axial force is transmitted to the column by a combination of bolts and welds. Nevertheless, connections of these types are frequently used. Engineers may satisfy the specific requirements by sizing welds (or bolts) by an appropriate application of an equilibrium force distribution. Such practices should theoretically be safe by the lower-bound plasticity theorem if all elements are appropriately designed to this equilibrium stress distribution and ductile behavior is achieved, thus allowing forces to redistribute to match the strength distribution provided. Experiments show that connections such as shown in **Figures 6-1a** and **6-1b** may develop the full resistance of the brace and the SCBF system, but research also shows that connections such as those of **Figure 6-1a** achieve less inelastic deformation capacity than connections with fully-restrained beam-column connections such as **Figure 6-1c** (Roeder et al., 2011).

6.2 Foundation Design

To achieve the goals of SCBF design, it is essential that the foundations be capable of developing the full resistance and deformation capacity of the braced frame. Unfortunately, current foundation design criteria do not ensure that the foundation will develop the required resistance. If the foundation is understrength, uplift may occur. Uplift may attenuate the seismic response, and it may aid in assuring life safety and collapse prevention. However, uplift may also cause significant damage to floor diaphragms and a whole range of nonstructural elements. This damage may also present life safety or collapse issues. Design guidance has been proposed for controlling uplift in structural design, guidance that goes well beyond the scope of this Guide. Reliance upon uplift without employing the rather extensive requirements needed to control uplift appears to be unwise for new construction. Hence, it is prudent to design the foundation to fully develop the strength of the braced frame.

Repair and retrofit of existing braced frames is a common engineering concern. It is costly to repair or retrofit an existing building, and foundation upgrade is even more costly and difficult. Hence, relying upon uplift on existing braced frames may be a more acceptable solution because the benefits of uplift may reduce the seismic risk relative to the existing condition of the braced frame system.

6.3 Composite SCBF

ASCE 7 and AISC 341 both permit the use of composite SCBF (C-SCBF) systems. In C-SCBFs, the columns may be concrete-encased composite columns or filled composite columns, while the beams may be either structural steel or steel girders with composite floor slab. The braces are either structural steel or filled composite members.

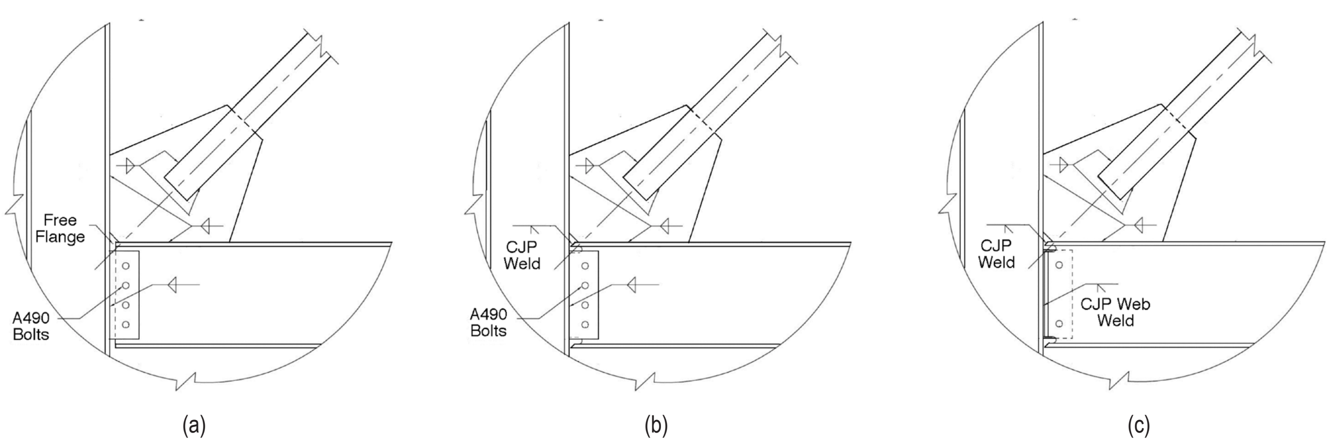


Figure 6-1 – Typical braced frame connections.

Composite braced frames have been constructed primarily using filled composite columns and either steel or filled composite braces. Filled composite columns are especially attractive in braced frames in taller structures having large axial compression forces in the columns or in braced frames where added stiffness is required.

The basis of design and analysis are comparable to that of SCBFs. Composite member design defers primarily to AISC 360, and added provisions are included in AISC 341 for detailing of the composite connections and splices.

6.4 Brace Transverse Displacement

Braces have been observed to develop significant transverse displacement upon buckling (especially after tension elongation), much larger than the axial deformation imposed (Uriz and Mahin 2008). Proper consideration of this deformation and its potential effects on other systems is necessary to ensure proper performance. Out-of-plane deformation of typical nonstructural walls may be considered acceptable and is commensurate with brace buckling. However, loss of gravity support for cladding and interference with exiting may represent unacceptable conditions. In such cases, sufficient separation should be provided, or frames should be configured to avoid damaging contact.

7. Detailing and Constructability

7.1 Buckling Deformation

Braces are anticipated to undergo significant axial shortening, which results in significant transverse deformation in the plane of buckling on the order of 10 % of the brace length (Tremblay 2002). Although building codes do not explicitly require consideration of this transverse displacement, it may create hazards in certain circumstances, such as braces located near windows configured to buckle out of plane. Designers should consider the effects of building layout and braced frame configurations with reference to such post-buckling secondary hazards.

7.2 Interaction with Architecture

Braced frames are often located within architectural walls. In such cases, the designer must provide for room for the anticipated brace buckling. If braces are configured to buckle out of the plane of the frame, the architectural wall may be terminated above and below the brace. (See the discussion of the "protected zone" below for other concerns.) If braces are configured to buckle in the plane of the frame, the architectural wall may be built as two walls with a cavity for the brace between.

If architectural elements restrain the brace from buckling, the maximum brace compression force may be higher than predicted. This may result in buckling of gussets or webs. Where the restraint cannot be avoided, the possibility of such modes can be eliminated by designing for a maximum brace compression force considering the architectural restraint.

Gussets configured to provide a pinned end for the brace may be restrained from providing rotation capacity by concrete fill at floors. Providing a blockout in the concrete or configuring the gusset to provide rotation capacity above the concrete should be considered.

7.3 Protected Zone

Where structural steel members are providing the inelastic drift capacity through inelastic strain in the steel, attachments to those regions are restricted. Low-toughness welds, shot pins, and similar potential crack initiators are not allowed in these "protected zones."

Braces in SCBFs may be subject to concentrated inelastic strain in regions where plastic hinging occurs as part of buckling. (The distributed inelastic strain entailed in tension yielding is expected to be significantly lower.) These regions of potential plastic hinging include the brace midspan, and, at the brace ends, either the ends of braces (for fixed-end braces) or the gusset plates (for gussets configured to facilitate rotation.

For exposed braces the restrictions on connecting to protected zones do not entail much complexity. Where braced frames are enclosed in an architectural wall, special attention is required to exclude attachments in the protected zone, which can reduce ductility.

There is ongoing research into the effect of attachments in the protected zone. Some (as yet unpublished) observations indicate that certain types of connection within the protected zone may be acceptable without significantly reducing member ductility, but further research is needed (Watkins et al. 2013).

7.4 Brace Connection Tolerances

Braces require some maneuverability and some construction tolerance for erection. Welded field connections generally provide some tolerance. However, this tolerance should be specified and accounted for in the design. For example, the buckling length of gussets may be increased considering this tolerance, and gussets and reinforcement plates should be detailed so that they are adequate through the range of permitted brace end locations.

Brace slots for slotted connections are typically fabricated $^1/_8$ inch wider than the gusset plate thickness and with two inches of length beyond the nominal edge of gusset. These tolerances typically provide the maneuverability needed for erection.

Bolted connections are often preferred in the field for economic reasons. For ease of erection, bolted connections of braces generally require oversize holes, which entail reduced design strength.

7.5 Direct-Welded Brace Connections

Direct-welded to the brace connections (braces welded directly to the beam, to the column, or to both) offer some economic advantages by reducing the number of force transfers. Such connections are difficult for HSS braces, especially round sections, due to the changing geometric conditions around the HSS perimeter. These connections may be challenging to configure for any shape at beam-column intersections. Where direct-welded connections are used, the beam-column connection assembly must be strong enough to resist the brace flexural plastic-hinge moment in the plane of buckling. This is of particular concern for out-of-plane buckling.

7.6 HSS Availability

Relatively few square and rectangular HSS meet the b/t limits in AISC 341. A great many round HSS meet those limits. However, many round HSS shapes are not frequently produced. Designers using round HSS braces (other than those that match pipe sections) should verify the availability of shapes with fabricators or service centers.

ASTM International, formerly known as the American Society for Testing Materials, has announced a new standard that has a minimum yield stress of 50 ksi and a maximum of 70 ksi (Melnick 2013). The specification will likely result in a more controlled product with a higher yield stress used for design and the same expected strength, thus reducing the material overstrength factor R_y.

8. References

ACI (2011). *Building code requirements for structural concrete (ACI 318-11) and commentary*, American Concrete Institute, Farmington Hills, MI.

AIJ (1995). *Preliminary reconnaissance report of the 1995 Hyogo-ken Nanbu Earthquake*, (English Edition) Architectural Institute of Japan, Tokyo, p. 216.

AISC (1997). *Seismic provisions for structural steel buildings*, American Institute of Steel Construction, Chicago, IL.

AISC (2010a). *Seismic provisions for structural steel buildings (AISC 341-10) and commentary*, American Institute of Steel Construction, Chicago, IL.

AISC (2010b). *Specification for structural steel buildings (AISC 360-10) and commentary*, American Institute of Steel Construction, Chicago, IL.

AISC (2012). *Seismic design manual*, American Institute of Steel Construction, Chicago, IL.

ASCE (2010). *Minimum design loads for buildings and other structures (ASCE/SEI 7-10)*, American Society of Civil Engineers, Reston, VA.

Astaneh-Asl, A., Goel, S.C., and Hanson, R.D (1986). "Earthquake design of double-angle bracing," *AISC Engineering Journal*, Vol. 23, No. 4, 4th Quarter, 1986, pp. 133-147.

Bruneau, M., Uang, C.M., and Sabelli, R. (2011). *Ductile design of steel structures*, McGraw-Hill.

Deierlein, G.G., Reinhorn, A.M., and Willford, M.R. (2010). "Nonlinear structural analysis for seismic design," *NEHRP Seismic Design Technical Brief No. 4*, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 10-917-5.

Dowswell, B. (2006). "Effective length factors for gusset plate buckling," *Engineering Journal*, AISC, Vol. 43, No. 2, 2nd Quarter, pp. 91–101.

EERI (1990a). "5 Buildings," *ES6 Supplement, Loma Prieta, California Earthquake of October 15, 1989, Earthquake Spectra*, Vol 6, Earthquake Engineering Research Institute, Oakland, CA, pp. 127-149.

EERI (1990b). "7 Industrial Facilities," *ES6 Supplement, Loma Prieta, California Earthquake of October 15, 1989, Earthquake Spectra*, Vol 6, Earthquake Engineering Research Institute, Oakland, CA, pp. 189-238.

EERI (1996). "2 Steel Buildings," *Supplement C to Vol. 11, Northridge Reconnaissance Report, Earthquake Spectra*, Vol. 11, Earthquake Engineering Research Institute, Oakland, CA, pp. 25-48.

Hsiao, P-C, Lehman, D.E., and Roeder, C.W. (2012). "Improved analytical model for special concentrically braced frames," *Journal of Constructional Steel Research*, Vol. 73, pp. 80-94.

Hsiao, P-C, Lehman, D.E., and Roeder, C.W. (2013a). "A model to simulate special concentrically braced frames beyond brace fracture," *Earthquake Engineering and Structural Dynamics*, Wiley, Vol. 42, pp. 183-200.

Hsiao, P-C, Lehman, D.E., and Roeder, C.W. (2013b). "Evaluation of response modification coefficient and collapse potential of SCBFs," *Earthquake Engineering and Structural Dynamics*, Wiley, DOI: 10.10002/eqe.2286.

IBC (2012). *International building code*, International Code Council, Washington, D.C.

Khatib, I. F., Mahin, S. A., and Pister, K. S. (1988). "Seismic behavior of concentrically braced steel frames," *Report No. UCB/EERC-88/01*. Berkeley: Earthquake Engineering Research Center, University of California.

Lehman, D.E., Roeder, C.W., Herman, D., Johnson, S., and Kotulka, B. (2008). "Improved seismic performance of gusset plate connections," ASCE, *Journal of Structural Engineering*, Vol. 134, No. 6, Reston, VA, pp. 890-901.

Lumpkin, E.J. (2009). "Enhanced seismic performance of multi-story special concentrically braced frames using a balanced design procedure," a thesis submitted in partial fulfillment of the MSCE degree, University of Washington, Seattle, WA, p.461.

Melnick, S. (2013). "Editor's note," *Modern Steel Construction*. May, 2013.

Okazaki, T., Lignos, D.G., Hikino, T., and Kajiwara, K. (2013). "Dynamic response of a chevron concentrically braced frame," *Journal of Structural Engineering*, ASCE, Reston, VA, DOI: 10.1061/(ASCE)ST.1943-541X.0000679

Palmer, K.D. (2012). "Seismic behavior, performance and design of steel concentrically braced framed systems," Ph.D. thesis, University of Washington.

Roeder, C.W., Lumpkin, E.J., and Lehman, D.E. (2011). "Balanced design procedure for special concentrically braced frame connections," Elsevier, *Journal of Constructional Steel Research*, Vol. 67 No. 11, pp. 1760-72.

Uriz, P., and Mahin, S. A., 2008. "Toward earthquake-resistant design of concentrically braced steel-frame structures," *Pacific Earthquake Engineering Research Center Report No. PEER 2008/08*, Berkeley: University of California, p. 401.

Sabelli, R., Sabol, T.A., and Easterling, S.W. (2011). "Seismic design of composite steel deck and concrete-filled diaphragms," *NEHRP Seismic Design Technical Brief No. 5*, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 11-917-10.

SEAOC (2013). *2012 IBC structural/seismic design manual*, *Vol. 4: Example for steel-framed buildings*, Structural Engineers Association of California, Sacramento, CA. (in press)

Tremblay, R. (2002). "Inelastic seismic response of steel bracing members," *Journal of Constructional Steel Research*, Vol. 58, pp. 665-701.

Watkins, C.E., Toellner, B.W., Laknejadi, K., Abbas, E., and Eatherton, M.R. (2013). "Effect of powder actuated fasteners on the seismic performance of steel moment frame connections," paper presented at the ASCE Structures Congress 2013.

Yoo, J.H., Roeder, C., and Lehman, D. (2008a). "Analytical performance simulation of special concentrically braced frames," ASCE, *Journal of Stuctural Engineering*, Vol. 134 No. 6, pp. 190-198.

Yoo, J.H., Lehman, D., and Roeder, C. (2008b). "Influence of gusset plate parameters on the seismic resistance of braced frames," *Journal of Constructional Steel Research*, 64, pp. 607-623.

9. Notations and Abbreviations

Specific units and definitions are found in the referenced documents.

A_g	gross cross-sectional area of the brace
b	width
C_d	deflection amplification factor
D	effect of dead load
e	eccentricity created by diaphragm step or depression
E_v	effect of vertical seismic input
f'_c	specified compressive strength of concrete
f_y	specified yield strength of reinforcement
f_l	live load factor, taken as 0.5 except taken as 1.0 for garages, areas occupied as places of public assembly, and all areas where L is greater than 100 psf
E	effect of horizontal seismic (earthquake-induced) forces
F_{cre}	flexural buckling stress or critical stress of the brace determined using expected yield stress
F_y	specified minimum yield stress
h_x	the height above the base to Level x
H	effects of soil, water in soil, or other materials
I	the importance factor
K	effective length factor for a compression member
k	distribution exponent for design seismic forces
L	span of diaphragm or diaphragm segment
P	axial force
P_{uc}	required axial strength in compression
P_{ut}	required axial strength in tension
R	response modification coefficient
R_i	reaction force in slab at vertical element i
R_y	ratio of expected yield stress to specified minimum yield stress
S_a	spectral response pseudo-acceleration, g
S_m	elastic section modulus
S_{DS}	design, 5 percent damped, spectral response acceleration parameter at short periods

t	thickness
t_p	thickness of panel zone
T	the fundamental period of the building
w_x	portion of effective seismic weight of the building that is located at, or assigned to, Level x
Δ	story drift
δ	member eccentricity from a straight line due to initial imperfection or deformation
ϕ	strength reduction factor
ρ	a redundancy factor based on the extent of structural redundancy present in a building
Ω_0	amplification factor to account for overstrength of the seismic force-resisting system defined in ASCE 7

Abbreviations

ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
ASTM	Formerly American Society for Testing and Materials, now ASTM International
ATC	Applied Technology Council
BRBFs	Buckling-Restrained Braced Frames
C-SCBF	Composite Special Concentrically Braced Frames
CBF	Concentrically Braced Frame
CBFs	Concentrically Braced Frames
CJP	Complete Joint Penetration
CUREE	Consortium of Universities for Research in Earthquake Engineering
HSS	hollow structural section
IBC	International Building Code
OCBFs	Ordinary Concentrically Braced Frames
SCBF	Special Concentrically Braced Frame
SCBFs	Special Concentrically Braced Frames
UFM	Uniform Force Method

10. Credits

Cover photo	Image courtesy of William A. Andrews, Walter P Moore
Figure 3-1	Image courtesy of Charles Roeder, University of Washington
Figure 3-2	Image courtesy of Charles Roeder, University of Washington
Figure 3-3(a)	Image courtesy of Charles Roeder, University of Washington
Figure 3-3(b)	Image courtesy of Degenkolb Engineers
Figure 3-3(c)	Image courtesy of Earthquake Engineering Research Institute
Figure 3-3(d)	Image courtesy of Earthquake Engineering Research Institute
Figure 3-3(e)	Image courtesy of Charles Roeder, University of Washington
Figure 3-3(f)	Image courtesy of Charles Roeder, University of Washington
Figure 3-4	Image courtesy of Charles Roeder, University of Washington
Figure 3-5	Images courtesy of Charles Roeder, University of Washington
Figure 4-1	Images courtesy of Charles Roeder, University of Washington
Figure 4-2	Image courtesy of the American Institute of Steel Construction Copyright © 2010 by American Institute of Steel Construction Reprinted with permission. All rights reserved.
Figure 5-1	Images courtesy of Rafael Sabelli, Walter P Moore
Figure 5-2	Images courtesy of Rafael Sabelli, Walter P Moore
Figure 5-3	Images courtesy of Rafael Sabelli, Walter P Moore
Figure 5-4	Images courtesy of Charles Roeder, University of Washington
Figure 5-5	Images courtesy of Charles Roeder, University of Washington
Figure 5-6	Image courtesy of Rafael Sabelli, Walter P Moore
Figure 5-7	Images courtesy of Rafael Sabelli, Walter P Moore
Figure 6-1	Images courtesy of Charles Roeder, University of Washington