State-of-the-Art Review on Seismic Design of Steel Structures

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Abstract: This state-of-the-art review provides an overview of the evolution of seismic design requirements for main steel building seismic force–resisting systems, as driven by new developments, the 1994 Northridge, California, earthquake, and changes in earthquake engineering practice in the United States. Important aspects of these systems in terms of ductility design and capacity design are highlighted. Recent developments in practice related to innovative systems are touched upon. The work presented here is intended to provide the reader with an appreciation of why the current seismic design requirements for steel structures are as framed, highlighting in the process several unresolved issues and inconsistencies that will require attention in future research. Implications of the Christchurch, New Zealand, rebuilding after the 2010–2011 earthquakes there for future U.S. seismic code development are also presented. **DOI: 10.1061/(ASCE)ST.1943-541X.0001973.**

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Introduction

Seismic design of steel structures is a broad topic. For example, in the United States the latest edition of "Seismic Provisions for Structural Steel Buildings" [more contemporarily referred to as AISC-341-16 or ANSI/AISC-341-16 (AISC 2016b)] has grown to rival in size a separate document, "Specification for Structural Steel Buildings" used for nonseismic design (ANSI/AISC 360-16). Thus, detailed review of all design and detailing requirements for the seismic design of steel structures, and the reasons for their existence, goes far beyond the scope of a technical paper—books, book chapters, design guides, and continuing-education courses already provide such comprehensive presentations (e.g., Bruneau et al. 2011; Hamburger and Malley 2016; Kersting et al. 2015; Sabelli et al. 2013; Naeim 2001).

This state-of-the-art review, in a complementary manner, ventures to provide an overview of how the philosophy of steel seismic design has evolved in recent decades, as driven by new developments, the occurrence of significant earthquakes, and changes in earthquake engineering practice. Following a brief look at how design codes have generally approached seismic design over the past decades, and a description of how the 1994 Northridge, California, earthquake represents a pivotal point in the seismic design of steel structures, the paper briefly describes the main structural steel systems commonly used in seismic design and highlights important aspects of the ductility requirements for all such systems. Also described are issues related to capacity design. Recent developments in practice related to proprietary and innovative systems are touched upon.

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Much of the focus here is on historical perspectives, given the legacy that has driven many aspects of the seismic design of steel structures and that explains some limitations in current knowledge. This is done with the caveat that such an introspective review, given the space limitations of a technical paper, risks being incomplete and somewhat (unintentionally) subjective. Nonetheless, although a comprehensive review of all research and perspectives on this broad topic is beyond the scope of this paper, the objective here is to provide the reader with an appreciation of the current seismic design requirements for steel structures as currently framed. This state-of-the-art review is linked to the design requirements of the AISC Seismic Provisions for the sake of anchoring its observations in a practical framework; it should not be construed as a negative critique of them. For simplicity, the review focuses only on U.S. practice, but is broader in reach because international practice and developments have generally traveled a similar path; many of the unresolved design questions highlighted here are not unique to the United States.

Seismic Steel Design Eras in Building Codes

The evolution of the seismic design of steel structures in the past half-century can be roughly divided into three eras. Prior to 1988, both general seismic loading provisions and material-related seismic design provisions (including those for steel structures) were typically integrated in a single document: the locally adopted model building code. In those days, seismic design of low-rise and midrise buildings without major irregularities used the "equivalent lateral force" method. For example, in the 1985 Uniform Building Code (UBC) (ICBO 1985) the base shear for working stress design was specified as

$$V_w = (ZIKCS)W \tag{1}$$

where the horizontal force factor, K = factor accounting for the relative ductility and energy dissipation of building systems.

Buildings were grouped into four types based on their earthquake-resisting elements, with K equal to 0.67, 0.80, 1.0, and 1.33 for seismic force–resisting systems (SFRSs), respectively, defined as moment-resisting frame systems, dual systems, building

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frame systems (for all systems not in the other three categories), and bearing wall systems. For steel buildings, only the ductile moment-resisting space frame (DMRSF) could be designed for the lowest seismic base shear, with $K = 0.67$. For application in high seismic regions, additional design requirements were few. The only requirement for members of steel DMRSFs was that "Members in which hinges will form during inelastic displacement of the frames shall comply with the requirement for plastic design sections." For capacity design, it was specified that "Each beam or girder moment connection to a column shall be capable of developing in the beam the full plastic capacity of the beam or girder."

For braced frames in moderate to high seismic regions, all members needed to be designed for 1.25 times the prescribed seismic forces; connections needed to be designed to develop either the full capacity of the members or the prescribed seismic forces without the one-third increase usually permitted in working stress design for stresses resulting from earthquake forces. Significantly, these requirements were based on the SEAOC recommendations (SEAOC 1980).

The 1988 edition of the UBC (ICBO 1988) began the second era of seismic steel design. On the seismic loadings side for base shear calculation, this edition abandoned the format of Eq. (1), that relied on empirical K factors. Instead, a response modification factor, R_w, was used to calculate V_w for working stress design, where

$$V_w = (ZIC/R_w)W \tag{2}$$

Based on the vertical components in the SFRS, each category was further divided into several classes with their associated R_w values. More significantly, specific ductility design requirements for special moment-resisting space frames (SMRSFs), concentrically braced frames (CBFs), and eccentrically braced frames (EBFs) were provided; a prescriptive girder-to-column moment connection for SMRSFs was codified; and the concept of capacity design was introduced (e.g., prescribing the use of amplified seismic load to protect columns from global failure). Based on the 1988 UBC, the first edition of the AISC Seismic Provisions was published in 1990 (AISC 1990). This seismic steel design practice continued through the 1994 UBC edition.

The trigger for the third era, as far as seismic steel design is concerned, was the Northridge, California, earthquake that occurred on January 17, 1994. This event drastically changed the seismic research, design, and construction of steel buildings in the United States and is fully addressed in a subsequent section.

For most of the previous century, three model building codes were predominantly and broadly used in different parts of the United States (a model building code serves as a reference, being adopted with or without modifications by local building codes in the United States, which has no mandatory national code). In the 1990s, there was an effort to unify all three codes into one, resulting in the creation of the International Building Code (IBC), which references ASCE 7 (ASCE 2016) for its design earthquake loads and the AISC Seismic Provisions for its seismic steel design requirements. To provide specific design requirements for the SFRSs listed in ASCE 7, starting with the 2010 edition, the AISC Seismic Provisions grouped steel SFRSs into the two categories: moment frame systems as well as braced frame and shear wall systems.

Although it appears that the ASCE 7 standard covers the required seismic forces (i.e., the load effect side) and that the AISC Seismic Provisions deal with design strengths (i.e., the resistance side) of steel members and components, these two standards are related to each other in an implicit, yet significant, way through the response modification factor, R. Furthermore, because the seismic load effect is also coupled with the actual strengths of the members and the entire structure, the AISC Seismic Provisions cover requirements not only for the resistance side but also for the required earthquake load effect side. Ductility design and capacity design are two pillars of the seismic design of structures. To pave the way for the presentation to follow, the relationship between these two design concepts and the R factor approach is briefly presented next.

Ductility Design, Capacity Design, and *R* Factor

Fig. 1 shows the expected structural response of an SFRS designed for a design earthquake. Point E represents the required seismic force level if the structure remains elastic. Because this force level can be high, say above 1 g times the reactive seismic mass of the building in high seismic regions, modern seismic codes accept the concept that damage is allowed for economic considerations. To facilitate routine elastic design (equivalent lateral load or modal response spectrum analysis), the seismic force at Point E is reduced to that at Point S by a response modification factor, R, for strength design; Point S represents the first significant event (e.g., plastic hinge formation in a beam in a moment frame, or brace buckling in a concentrically braced frame) beyond which the structure responds in the inelastic range (for working stress design, Point E is further reduced to Point W by an R_w factor). When the structure is redundant, with ductility built into members that are expected to undergo inelasticity, the structure deforms further beyond Point S to its maximum strength at Point M and degrades to Point U if strength degradation due, for example, to member buckling or the P-Δ effect, occurs. Therefore, the R factor is mainly composed of two components (Freeman 1990; Uang 1991; NIST 2012)

$$R = R_\mu \Omega_o \tag{3}$$

where R_μ $(=C_e/C_y)$ = ductility reduction factor at the system level; and Ω_o $(=C_y/C_s)$ = system overstrength factor.

A few observations can be made about the R-factor design procedure:

- The structure is expected to deform into the inelastic range because damage is accepted in a design earthquake for economic reasons;
- The R factor was developed mainly for convenience in routine design because elastic analysis is still valid for structural performance evaluation at Point S;
- Ductility (i.e., inelastic deformation capacity) is needed for structural components expected to experience inelasticity; and

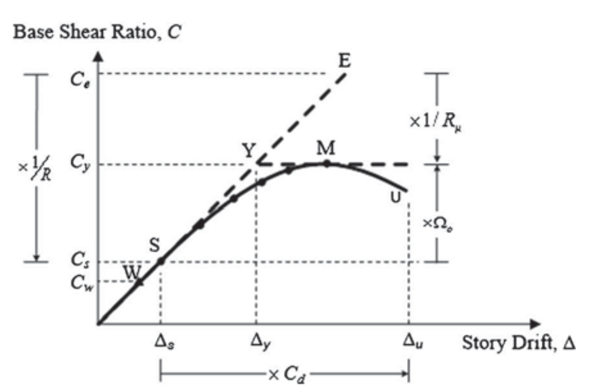

Fig. 1. Typical structural response and system performance factors

- Ultimate lateral strength at the system level (Point M) can be much higher than that at Point S, where an elastic analysis is performed.

Regarding the first bullet entry, seismic codes set a target plastic mechanism for each SFRS for energy dissipation that minimizes inelastic deformation demand while maintaining the gravity load–carrying capacity. Taking moment frame design as an example, a classical plastic design may allow plastic hinges to form in either beams or columns if the ultimate strength of the frame is no less than that required. For seismic design, however, codes aim to achieve a more desirable plastic mechanism that limits the premature formation of story mechanisms by promoting hinging in beams and limiting it in columns. Although the R factor associated with the target plastic mechanism of each SFRS is given in ASCE 7, the designer relies on the AISC Seismic Provisions to ensure that sufficient ductility capacity in the third bullet entry just given is built into these systems.

For structural elements that are designed to remain essentially elastic and do not undergo energy dissipation in a seismic event, the fourth bullet entry implies that the horizontal seismic load effect is significantly higher than that computed in accordance with the second bullet entry. The capacity design principles are then used to compute the seismic load effect in these structural components. Because the required seismic forces needed for capacity design correspond to the seismic force level at Point M, conceptually a nonlinear analysis is needed. To bypass nonlinear response analysis for routine design, ASCE 7 uses an empirical overstrength factor, Ω_o, to amplify the seismic force effect from Point S to Point M. This amplified effect, equal to the value obtained from the code-specified earthquake load, E_h, multiplied by Ω_o and termed E_{mh}, has been specified as not needing to be larger than the value computed from a plastic analysis using the realistic expected values of material strengths, termed E_{cl} (in 2016 ASCE 7 terminology). Although convenient, multiplying E_h by Ω_o has been recognized as flawed because it fails to capture the redistribution of forces that typically occur during nonlinear response and that may affect demands on the elements that are intended to be protected by capacity design principles. Whenever possible, procedures to compute E_{cl} have been provided in the AISC Seismic Provisions, and in the 2016 edition AISC requires that E_{cl}, when specified, be used as the value of E_{mh} for capacity design even if it exceeds that computed based on the Ω_o factor.

Impact of the 1994 Northridge Earthquake

The 1994 Northridge, California, earthquake might have had more impact on seismic steel research, design, and construction practice than any other seismic event in the United States. Prior to it, steel special moment-resisting space frames, or simply special moment frames (SMFs), were thought to be a premier system for earthquake resistance, with the highest R_w value (or the lowest K value) since the 1960s. It was expected that ductile response in the form of beam flexural hinging at the column face, shear yielding in the column panel zone, or a combination of the two would occur such that a ductile plastic mechanism would form in the frame. However, during the Northridge event, with little sign of plastic deformation in the beams, brittle fracture of beam-to-column moment connections occurred in many multistory steel buildings; those with reported connection damage ranged in height from single story to 26 stories and in age from new at the time of the earthquake to 30 years. The connections in question had beam flanges welded to the column flange with complete-joint-penetration (CJP) groove welds and beam web bolted to the column flange with a shear plate

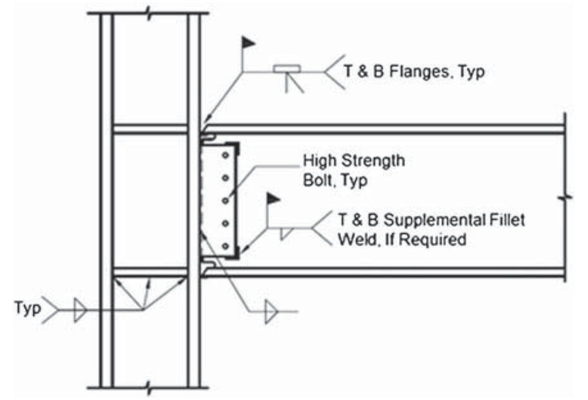

Fig. 2. Typical special moment frame (SMF) beam-to-column connection prior to 1994

(FEMA 2000j). Based on limited test data (Popov and Stephen 1970; Krawinkler et al. 1971; Popov et al. 1985; Popov and Tsai 1989), this type of welded flange–bolted web connection (Fig. 2) became popular on the West Coast. It was not until the 1988 UBC that this so-called pre-Northridge connection was approved as the only one prequalified for SMF construction. Based on testing of connections with W30 beams, Englehardt and Hussain (1993) published a paper shortly before the earthquake that raised concerns about the reliability of this type of moment connection.

Immediately after the Northridge earthquake, a multiyear U.S. effort known as the SAC Joint Venture, combining the expertise of design engineers, academics, fabricators, and steel and welding consumables producers, was initiated to investigate the cause of the Northridge fractures and to develop better design and construction practice. The outcome of this effort was documented by FEMA (FEMA 2000b, c, d, e) in the design of moment frames in new structures, the upgrade and repair of existing structures, and quality control.

FEMA-350 (FEMA 2000c) summarizes the main factors that contributed to the fractures, which occurred primarily at the beam bottom flange level:

- Design: The assumption held at the time that welded beam flanges transfer moment and bolted webs transfer shear to the column was inappropriate given that a significant amount of beam shear actually transferred through the beam flanges in these connections, and allowing a weak panel zone in design exacerbated the problem; the consequence of using deeper and heavier beams with fewer bays in SMFs commonly designed in the 1980s and 1990s was also not well understood; the effect of triaxial stress demands on the ability to develop yielding in beam flanges welded on the column face had also been under-estimated;
- Materials: The impact of increased steel yield and tensile strengths on the cyclic performance of steel connections, due to a change in steel production in the 1980s, was neither recognized nor considered in design; and
- Welding: The significance of welding details, processes, consumables, workmanship, and inspection was overlooked by researchers and design engineers.

Regarding design, improved seismic design procedures that avoid similar brittle fracture were developed based on the SAC study (FEMA 2000c). In particular, several prequalified moment connections were proposed that relied heavily on full-scale experimental verification and associated finite-element analyses. Each connection type has specific ranges of applicability (e.g., size, weight, beam and column slenderness parameters, span–to–beam depth ratio, steel grade). These recommended prequalified connections also formed the basis of the AISC 358 standard "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications" (AISC 2016a), which was first published in 2005. Although the recommended design criteria developed by SAC were intended for ductile moment frames only, the same methodology impacted the development of all other steel SFRSs in several aspects. For example, A36 and A572 Grade-50 steels were routinely used for the construction of pre-Northridge SMFs. However, they did not have an upper bound on yield stress. In response to the need to cap yield stress for capacity design purposes, a new grade of steel (ASTM A992) was developed in 1998 to replace A36 and A572 for W-shaped members. Material ductility for ASTM A992 is well defined because a maximum yield–to–tensile strength ratio of 0.85 is specified, and weldability is improved because a maximum carbon equivalent value of 0.45 (0.47 for Group-4 and Group-5 shapes) is required. For capacity design, the concept of expected material strength and strain-hardening factors were also introduced for the design of all SFRSs.

The Northridge earthquake also drastically changed experimental seismic steel research in the United States. Previously, it was not uncommon for steel researchers to test small-scale steel components and then extrapolate test results to much larger members. Without realizing the significance of welding, it was also not uncommon for important welding information on fabrication and erection of test specimens (such as process and position, electrode used, qualification of field and shop welders, inspection) not to be documented. The Northridge earthquake changed this practice. Nowadays, full-scale testing of structural members, connections, and subassemblies is routine and test results with reduced-scale models are viewed with suspicion. When welded joints are involved, especially when field-welded CJP welds are used, welding is routinely done in the test laboratory to simulate actual conditions. Prior to the earthquake, there was no welding code specifically for seismic applications. In response to the need for one, the American Welding Society published the first edition of AWS D1.8 (AWS 2005), "Structural Welding Code—Seismic Supplement," in 2005 as a supplement to AWS D1.1 (AWS 2015), "Structural Welding Code," to be used in conjunction with the AISC Seismic Provisions and AISC 358.

Steel Seismic Force-Resisting Systems

For completeness, this section provides a brief description of the main steel (i.e., noncomposite) structural systems included in the AISC Seismic Provisions (Fig. 3). Malley (2010) provides a brief history of the development of the provisions.

Moment Frame Systems

Steel moment frame construction, despite its higher construction cost when compared with that of braced frame construction, remained popular even after the Northridge earthquake because it provided architects and building owners with open-space floor plans. For moment frame systems, both special moment frames (SMFs) and ordinary moment frames (OMFs) from the previous editions of the model codes were included in the 1990 AISC Seismic Provisions. In 1997, steel intermediate moment frames (IMFs) were introduced, intended for application in low to moderate seismic regions where, by specifying a lower value for the R factor of 6 (compared with 8 for SMFs), less inelastic deformation was expected. Whereas an SMF was to provide significant inelastic deformation capacity through flexural yielding in the beams and limited yielding of column panel zones, column hinging was also allowed in an IMF. However, a study conducted by SAC (FEMA 2000c) concluded that the inelastic deformation demands on IMF systems were actually like those on SMF systems and therefore that the reduction in design criteria associated with IMF systems was not justified. This led the 2002 edition of ASCE 7 to reduce the value of R from 6 to 4.5, reduce the building height limit, and severely restrict the use of IMFs in higher seismic design categories.

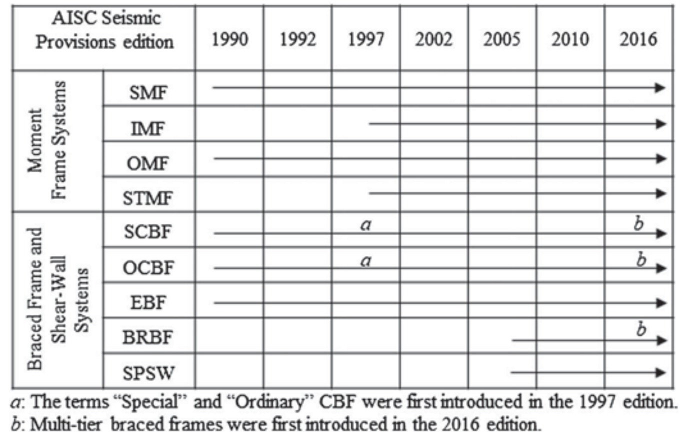

Fig. 3. History of steel seismic force–resisting systems

Introduced initially as a moment-resisting system, the truss moment frame is used for relatively long bay widths. Early editions of the UBC allowed the use of trusses as horizontal members in an SMF if the trusses were designed to be stronger than the columns (i.e., the strong truss–weak column concept). Based on research conducted by Goel and Itani (1994), special truss moment frames (STMFs) were introduced in 1997 in the UBC and the AISC Seismic Provisions. The purpose of STMFs is the design of a special segment inside the truss to yield in a ductile manner while the other members outside the special segment stay in the elastic range.

Concentrically Braced Frames

Whenever diagonal braces in a building are acceptable from the architectural perspective, concentrically braced frames (CBFs) are a popular option. Some aspects of capacity design for CBFs were first incorporated in the 1988 UBC and 1990 AISC Seismic Provisions, but the terms *special concentrically braced frame (SCBF)* and *ordinary concentrically braced frame (OCBF)* were not introduced until the AISC Seismic Provisions released in 1997. Early research at the brace and brace connection level (e.g., Gugerli and Goel 1980; Zayas et al. 1980; Popov and Black 1981; Astaneh-Asl and Goel 1984; Astaneh-Asl et al. 1985, 1986; Aslani and Goel 1991; Goel 1992) and at the system level (Ghanaat and Clough 1982; Foutch et al. 1987; Bertero et al. 1989) formed the basis of CBF design provisions in model codes starting in 1988. Subsequent research (Tremblay et al. 2003; Shaback and Brown 2003; Tremblay and Filiatrault 1996; Tremblay 2002; Celik et al. 2005, to name a few) further enhanced knowledge of CBF cyclic inelastic deformations. Before the SAC Joint Venture published its design recommendations for SMFs in 2000, many design engineers (and architects) turned to CBFs, especially SCBFs. Various code committees have attempted to eliminate OCBFs, but these frames have been kept with strict limitations on building height (10.7 or 20.9 m for single-story buildings with a dead load restriction) and a low R value (3.25).

Interest in SCBF research was renewed in the early 2000s (Yang and Mahin 2005; Uriz and Mahin 2008; Lehman et al. 2008; Roeder et al. 2011; Lai and Mahin 2014) Some researchers have also turned their attention to CBF design with so-called $R = 3$ buildings. These are a type of construction commonly used in moderate seismic regions or representative of pre-1988 CBF construction in high seismic regions (Hsiao et al. 2012a; Bradley et al. 2017), partly because, in moderate seismic regions ASCE 7 does not require buildings classified as Seismic Design Category B or Seismic Design Category C to comply with AISC Seismic Provisions as long as $R = 3$ is used to compute earthquake loads. Because the AISC Seismic Provisions were developed with a heavy bias toward applications in high seismic regions, it is not clear if a CBF designed with $R = 3$, and so not meeting ductility and capacity design requirements, can provide the same margin against collapse as that provided by a ductile system. To determine if there is sufficient reserve strength to offset the a system's low ductility capacity, Hines et al. (2009), Stoakes and Fahnestock (2011), and Bradley et al. (2017) tapped into reserve lateral strengths generally ignored in a routine design (e.g., semirigid connections in the gravity frame system, connections in the braced frame system, column continuity, base fixity, brace reengagement).

Eccentrically Braced Frames

Eccentrically braced frames were originally developed in Japan (Fujimoto et al. 1972; Tanabashi et al. 1974). In the United States, they were first studied by Roeder and Popov (1978). In the 1980s, numerous studies on link behavior provided insight into the cyclic response of EBFs (Manheim and Popov 1983; Hjemstad and Popov 1983, 1984; Malley and Popov 1984; Kasai and Popov 1986a, b; Ricles and Popov 1989; Engelhardt and Popov 1989, 1992). Experimental studies to verify EBF response at the system level were also conducted from the mid-1980s on (Yang 1985; Roeder et al. 1987; Whittaker et al. 1989). These studies led to design provisions in the 1988 UBC and later in the AISC Seismic Provisions.

Further studies have been conducted in the last two decades, including full-scale testing of large links not only for buildings but also for bridges (Dusicka and Itani 2002; Itani et al. 2003; Zahrai and Bruneau 1999; McDaniel et al. 2003; Sarraf and Bruneau 2004). Recent research on links has also extended from I-shaped rolled links to built-up sections [including I-shaped and double-C (Mansour et al. 2011) and boxed sections (Berman and Bruneau 2008b, c)]. Boxed sections were incorporated in the 2010 AISC Seismic Provisions. Replaceable links have also been explored given increasing emphasis on performance-based design (Ramadan and Ghobarah 1995; Mansour et al. 2011; Dusicka and Lewis 2010).

Buckling-Restrained Braced Frames

Originally developed in Japan in the 1970s, buckling-restrained braced frames (BRBFs) have been used extensively for seismic applications in that country since the 1995 Kobe earthquake (Reina and Normile 1997). These are a type of concentrically braced frame, except that the buckling-restrained braces (BRBs) are specially detailed to prevent global buckling and significant strength loss. A BRB is composed of a ductile steel core designed to yield in both tension and compression (Watanabe et al. 1988). To preclude global buckling in compression, the steel core is first placed in a steel casing [e.g., a hollow structural section (HSS)], which is then filled with mortar or concrete. Before the mortar is cast, an unbonding material or a small air gap is provided between it and the steel core to minimize or, if possible, eliminate the transfer of axial force from the core to the mortar and the HSS. Because the Poisson effect causes the steel core to expand under compression, this small gap is needed to allow for expansion. Research on BRBs and BRBFs, especially in Asia, has been active for 15 years. Uang et al. (2004), Xie (2005), and Bruneau et al. (2011) provide more detailed information on the development of BRBFs.

The BRBF system for high seismic applications is relatively new in the United States, only having gained rapid acceptance a few years after the 1994 Northridge earthquake. Although BRBs are treated as supplemental energy dissipation devices usually inserted into a moment-resisting frame in Japanese design (Iwata et al. 2000), one major factor contributing to their U.S. acceptance was the conversion of the system into a new kind of braced frame with an R factor–based elastic design procedure that designers were familiar with (López and Sabelli 2004). The BRBF has been codified as an SFRS with the highest R value (8) since the 2005 editions of ASCE 7 and the AISC Seismic Provisions.

Because a BRB does not buckle, its yielding steel core is much smaller than the corresponding brace area in a concentrically braced frame. Therefore, BRBFs are more flexible than SCBFs and code-specified drift limits may govern their design. Although the hysteretic response of a BRB is full and stable, its postyield stiffness (or hardening ratio) is relatively low. Therefore, BRBFs tend to cause a concentration of damage in specific stories and produce large residual drifts (Sabelli et al. 2003; Kiggins and Uang 2006; Sahoo and Chao 2015). Research on mitigation of damage concentration not only in BRBF but also in braced frames in general is presented in the section "Capacity Design."

Shear Wall Systems

The special plate shear wall (SPSW) is the second of the two SFRSs first introduced in the 2005 AISC Seismic Provisions (several types of steel-concrete composite walls are included in the provisions, but are not addressed here because composite structures are beyond the scope of this paper). Steel shear wall systems were used from time to time starting in the 1970s for various purposes (Sabelli and Bruneau 2007). The design philosophy, still used in Japan, was to stiffen the steel plates, using either stiffeners or concrete cover, to prevent their buckling. The idea of unstiffened steel plate shear walls that rely on postbuckling tension-field action was first advocated in the early 1980s (Timler and Kulak 1983; Thorburn et al. 1983; Tromposch and Kulak 1987). The structural behavior of the SPSW is conceptually similar to that of a cantilever plate girder, where the steel infill panels serve as the web, the columns serve as the flanges, and the beams serve as the intermediate stiffeners. Bruneau et al. (2011) provide a review of the historical development of this structural system. Early research efforts led to a simplified design method (known as diagonal strip method) that was incorporated into the Canadian steel design standard CAN/CSA S16-95 (CSA 1994).

The 2005 editions of ASCE 7 and the AISC Seismic Provisions essentially built on the Canadian procedure with modifications in format and terminology. Significant research was conducted in the 1990s and early 2000s that resulted in enhancement of SPSW design provisions from 2005 to 2016 (Zhao and Astaneh-Asl 2004; Behbahanifard et al. 2003; Berman and Bruneau 2003, 2008a; Caccese et al. 1993; Dastfan and Driver 2008; Driver et al. 1998; Elgaaly 1998; Grondin and Bahbahannifard 2001; Lee and Tsai 2008; Lin et al. 2010; Lubell et al. 2000; Qu and Bruneau 2009, 2010a, b; Bruneau and Qu 2011; Qu et al. 2008; Purba and Bruneau 2009, 2010; Rezai 1999; Roberts and Sabouri-Ghomi 1991, 1992; Sabelli and Bruneau 2007; Vian et al. 2009a, b), The contributions of this research include models to determine capacity design demands in columns and beams, and modified design requirements to ensure adequate ductile response, as presented in Bruneau et al. (2011).

Ductility Design

First and foremost, a ductile material is at the root of ductile design. Steel has served as the primary ductile material in structural engineering (as steel members or as reinforcement in concrete and masonry). However, a number of issues related to metallurgy (material toughness, temperature, strain rate, welding procedures and electrodes, fabrication and detailing, sharp discontinuities, chemical exposure, and many more) greatly reduce or eliminate the ductile behavior of steel structures (Barsom and Rolfe 1999; FEMA 2000f, j; Bruneau et al. 2011). Although this state-of-the-art review is focused mainly on performance at the structural component and system levels, it is advisable not to lose sight of these critical issues and thereby take ductile steel for granted.

Relying on the development of a yield mechanism to survive a significant seismic event requires the ductile design of SFRSs with specific members designated to experience inelastic action (e.g., beams and panel zones in SMFs, diagonal braces in SCBFs, links in EBFs). These members must be sized to have sufficient strengths for the basic seismic loading combinations, where the horizontal seismic effect corresponds to the R factor–reduced force at Point S in Fig. 1. As indicated in Eq. (3), one component of R is the system reduction factor, R_μ, which is related to ductility capacity (μ) at the member level. Because the relationship between system-level R_μ and member-level μ for a multistory building is complex, depending on, among many things, elastic stiffness and development of the system's plastic mechanism (partial versus full), values of R specified in ASCE 7 were originally established mainly by converting from the UBC's empirical K factor (Eq. 1). In other words, the explicit μ-R_μ relationship for each SFRS has typically not been spelled out in the seismic provisions of U.S. building codes.

Member ductility or inelastic deformation capacity of steel members is generally governed by slenderness parameters, material properties, and loading patterns. For a compact flexural member, for example, local buckling and out-of-plane lateral-torsional buckling may occur soon after the compression flange has yielded and strain-hardened (ASCE-WRC 1971; Kato 1990; Ziemian 2010). However, modern steel design codes usually simplify this complicated phenomenon by treating flange local buckling (FLB), web local buckling (WLB), and lateral-torsional buckling (LTB) as separate limit states, even though they interact in reality. Ideally, because SFRSs designed per ASCE 7 are expected to experience inelastic response, the inelastic deformation demands of the structural components should be compared with those allowed. Nevertheless, except for a very limited number of cases to be discussed later, the AISC Seismic Provisions take an indirect approach by setting limiting values that are functions of the slenderness parameters and material properties. The following section describes this approach and how it has evolved over the years for various steel SFRS systems.

Moment Frames

For local buckling control of beams and columns in SMFs, design codes specify limiting slenderness ratios to ensure sufficient plastic rotations. The Appendix summarizes the evolution of local buckling requirements for seismic design. Prior to the 1988 UBC, seismic section compactness criteria were borrowed directly from plastic design (ASCE-WRC 1971), even though plastic design does not consider the effect of cyclic loading. Plastic rotation capacity, which is the ratio between plastic rotation and rotation at yield and is on the order of 6 to 7 under monotonic loading (AISC 2002), is defined by the largest rotation reached on the response curve after reaching maximum strength and before buckling degrades flexural strength below the plastic moment, M_p. For seismic design, the argument appears to be that it is acceptable to allow strength degradation under cyclic loading if the rate of degradation is gradual.

For flange local buckling control, the tabulated limits for the width-thickness ratio in the UBC up to the 1985 edition can be traced back to the plastic design provisions in the 1969 AISC (AISC 1969) specifications. Over the years, practically the same limiting values have been converted to a different form as a function of $\sqrt{F_y}$, $\sqrt{E/F_y}$ or, in the 2016 AISC Seismic Provisions, $\sqrt{E/(R_yF_y)}$ for an assumed R_y value of 1.1; these alternate forms are not based on new research findings.

The limiting seismic width-thickness ratio, h/t_w, for web local buckling control has, with one exception, also been strongly influenced by plastic design provisions. The rightmost column in the Appendix shows an evolution, where C_a $[=P_u/(\phi_c P_y)]$ is the normalized axial compression force. Without an axial load, the limiting ratio for the beam web in the AISC Seismic Provisions before the 1994 Northridge earthquake was $520/\sqrt{F_y}$ (equivalent to $3.05\sqrt{E/F_y}$ in ksi units). After the earthquake, the reduced beam section (RBS) connection, first introduced in the late 1980s (Plumier 1997), began to receive much attention in the United States, becoming one the most researched connections in the last two decades. Testing showed that WLB usually starts first, interacting with FLB, and is followed by LTB (Engelhardt et al. 1998; Yu and Uang 2001). To evaluate the effect of reduced flanges, Uang and Fan (2001) used a procedure similar to that recommended by Kemp (1996) to consider the interaction of all three buckling modes

$$\theta_p = C\left(\frac{b_f}{2t_f}\right)^\alpha \left(\frac{h}{t_w}\right)^\beta \left(\frac{L_b}{r_y}\right)^\gamma (F_y)^\delta \tag{4}$$

where θ_p = plastic rotation of the connection. A regression study of 55 full-scale RBS connections tested cyclically showed that WLB is the governing limit state when beam lateral bracing complies with the AISC Seismic Provisions. Setting $\theta_p = 0.03$ rad, which was the required value for SMF in the 2002 AISC Seismic Provisions, the following criterion was established:

$$\left(\frac{b_f}{2t_f}\right)^{1/8} \left(\frac{h}{t_w}\right)^{1/2} = \frac{202}{\sqrt{R_yF_y}} \tag{5}$$

To simplify further, a lower-bound limiting ratio of h/t_w was set at $418/\sqrt{F_y}$ (or $2.45\sqrt{E/F_y}$), where F_y is in ksi units. Although this ratio was derived for the beam web when an RBS was used, the AISC Seismic Provisions since the 2002 edition apply it to all SFRS beams, irrespective of whether an RBS is used. For column webs, however, the limit for $C_a = 0$ (i.e., no axial load) remained unchanged at $3.14\sqrt{E/F_y}$, creating an inconsistency between beams and columns that was resolved in the 2010 edition by artificially anchoring the expression for the highly ductile limit of column webs, λ_{hd}, to $2.45\sqrt{E/F_y}$.

To control lateral-torsional buckling of SMF beams that are expected to develop plastic hinges, the UBC in its 1985 and earlier editions did not provide any specific requirement for lateral bracing, but the connections requirement did state that the beams were expected to develop full plastic capacity. The 1986 AISC LRFD Specification (AISC 1986) required the following for the maximum unbraced length, L_{pd}, for plastic design to ensure a minimum rotational ductility capacity of 4:

$$\frac{L_{pd}}{r_y} = \frac{3600 + 2200(M_1/M_p)}{F_y} \tag{6}$$

which assumes that a plastic hinge forms at one end of the unbraced segment with the plastic moment, M_p, that M_1 is the smaller moment at the other end of the segment, and that (M_1/M_p) is positive when moments cause reverse curvature. In a seismic event, beams in a moment-resisting frame are almost always in reverse curvature between columns unless the other end is pinned. Conservatively assuming the latter case (i.e., $M_1 = 0$), Eq. (6) becomes

$$\frac{L_{pd}}{r_y} = \frac{3600}{F_y} \tag{7}$$

At the time when it was common to specify A36 steel for beams, this limiting requirement further reduced to $L_{pd}/r_y = 100$. The UBC since 1988 has therefore specified a limiting value of 96 for L_{pd}/r_y. The AISC Seismic Provisions since 1990 have considered Eq. (7) to reflect the effect of yield stress. Considering the uncertainty of plastic hinging locations in the beam, however, AISC conservatively reduced the numerator from 3,600 to 2,500

$$\frac{L_{pd}}{r_y} = \frac{2500}{F_y} \tag{8}$$

Eq. (8), which is still used in 2016 AISC Seismic Provisions but is expressed as $0.086E/F_y$ or $0.095E/(R_yF_y)$, is equivalent to Eq. (6) by assuming that the unbraced segment is in a single curvature and that M_1 is equal to $0.25M_p$. In other words, the seismic lateral bracing requirement has been adapted from the original plastic design requirement, not from seismic research that considers the cyclic nature of the expected structural response. Nakashima et al. (2002) showed through numerical simulation that this lateral bracing requirement is conservative.

The panel zone design philosophy has changed significantly over the years. Because panel zones are expected to yield in shear, conceptually they need to be designed for the shear force obtained from R factor-reduced seismic forces (or, in the 1988–1994 editions of the UBC, R_w factor-reduced seismic forces). This was indeed the case for seismic codes (e.g., 1988 UBC and 1992 AISC Seismic Provisions) before the 1994 Northridge earthquake, except that the required shear force did not need to exceed that determined from $0.8\sum M_p$ of the beams framing into the column flanges at the connection. The 0.8 factor was intended to account for the gravity load effect (Popov 1987). This relaxation in the design requirement led to weaker panel zones, which became one of the many factors that contributed to the brittle fractures of pre-Northridge moment connections (FEMA 2000k).

After the Northridge earthquake, the AISC Seismic Provisions specified that the required panel zone shear strength be determined from the extrapolated beam moment at the column face—an approach similar to that reserved for the capacity design of components not expected to yield. Furthermore, the 0.8 factor was deleted because the beneficial effect from gravity loads in preventing beam yielding may not always exist (El-Tawil et al. 1999). The goal of the panel zone design requirements in the latest AISC Seismic Provisions is to avoid excessively weak zones but allow limited yielding; notably, the panel zone design strength provided by the equation specified in AISC 360 is only developed when the panel zone deforms to four times the shear yield strain (Krawinkler 1978), which is arguably a large ductility demand in a structural element designed per capacity design principles.

Although the panel zone has been shown to be a reliable source of energy dissipation, one well-known argument against an SMF design approach with weak panel zones is that large shear deformation in them causes kinking in the column flanges and such kinking can trigger fracture of the beam flange CJP welds. Unfortunately, little research has been conducted to quantify the threshold beyond which such concern is applicable. When notch-tough CJP welds are used, Kim et al. (2015) postulated that the ultimate panel zone deformation capacity corresponds to that when each column flange is fully yielded at the kink locations. They showed that deformation capacity is a function of the ratio of beam depth to column flange thickness. However, the proposed equation was verified experimentally by only two full-scale moment connection specimens. Because many SMFs constructed before the 1994 Northridge earthquake have weak panel zones, more research is needed to establish critical panel zone deformation capacity for retrofit purposes.

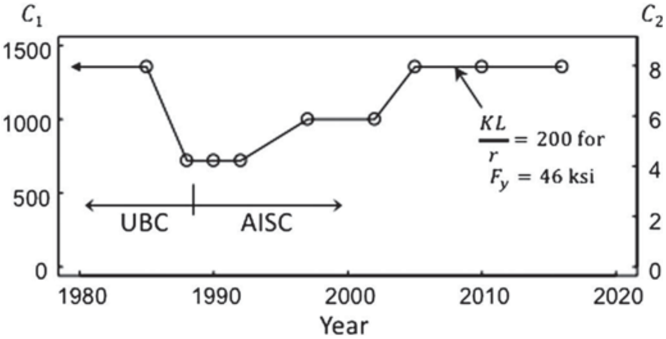

Fig. 4. Evolution of slenderness limiting KL/r ratio for special concentrically braced frame (SCBF) braces

Concentrically Braced Frames

A concentrically braced frame relies on compressive buckling and tensile yielding of the diagonal braces to dissipate energy. Compared with flexural bending or tensile yielding, Euler-type flexural buckling is a much less desirable energy dissipation mechanism. Axial compressive strength also drops rapidly in the postbuckling range, which affects internal force distribution and the required forces for the design of beams, columns, and gusset connections. Prior to 1988, no special requirement was provided for the seismic design of braced frames. In the 1988 UBC, a few key concepts for ductile design of braced frames in high seismic zones were introduced.

It is well known that the effective slenderness ratio plays a key role in the buckling response of a brace. Fig. 4 shows that thinking has evolved regarding the maximum permitted value for this ratio, which is expressed in either of the following forms:

$$\left(\frac{KL}{r}\right)_{\max} = \frac{C_1}{\sqrt{F_y}} \tag{9a}$$

$$\left(\frac{KL}{r}\right)_{\max} = \frac{C_2}{\sqrt{E/F_y}} \tag{9b}$$

Because no special requirement was specified for seismic steel design prior to the 1988 UBC, the maximum effective slenderness ratio defaulted to the requirement specified (=200) in the AISC Specification for structural steel buildings (i.e., for nonseismic applications). In the 1988 UBC, stockier braces with $C_1 = 720$ were favored because available research showed that they exhibited greater energy dissipation capacities (Popov and Black 1981). The promotion of stockier braces was also compounded by a strength reduction factor, β, to compute the design compressive strength of the brace

$$\beta = \frac{1}{1 + (\frac{KL}{r})/(2C_c)} \tag{10}$$

This strength reduction factor was not intended for capacity design. It was simplified to 0.80 in the 1990 and 1997 AISC Seismic Provisions, and this design approach (using a reduced compressive design strength) was eventually abandoned.

Braces are designed to experience flexural buckling (either in plane or out of plane) with a plastic hinge forming at the midlength of the brace. Under axial compression, braces, especially those with rectangular HSS sections, at the plastic hinge location experience local buckling and initiate cracks under load reversal. Because of the large inelastic strains that develop in the plastic hinge, this local buckling becomes more severe when stocky braces are used. Tang and Goel (1989), Goel and Lee (1992), and Tremblay (2002) showed that the postbuckling cyclic fracture life of bracing members generally increases with an increase in the slenderness ratio. It was also found that frames with slender braces behave better because of the overstrength inherent in their tension capacity when design is governed by the strength of the compression brace (Tremblay 2003). For these reasons, after more than 15 years of research and deliberations, the limiting slenderness ratio in the 2005 AISC Seismic Provisions was restored to what it was in the preseismic steel design era.

Local buckling also plays an important role in the low-cycle fatigue life of braces, particularly HSS braces. Tang and Goel (1989) proposed an empirical model to predict the fracture of braces; it was believed that under cyclic loading straightening and stretching of the brace had a greater impact on fracture life than compressive deformation excursions. More testing led to several refined models (Fell et al. 2009; Uriz and Mahin 2008; Huang and Mahin 2010; Hsiao et al. 2012b). For code implementation, the limiting width-thickness ratio for rectangular HSS sections was set at $110/\sqrt{F_y}$ ($=0.64\sqrt{E/F_y}$) for SCBF design; this limiting value was based on test data from Tang and Goel (1987) and Uang and Bertero (1986). To enhance low-cycle fatigue life (Fell et al. 2009; Uriz and Mahin 2008), the limiting value was tightened to $0.55\sqrt{E/F_y}$ in the 2010 AISC Seismic Provisions. In the 2016 provisions, it became $0.65\sqrt{E/R_yF_y}$ for highly ductile and moderately ductile sections—the same as for specifying $R_y = 1.4$ for HSS members. Seismic codes treat global buckling and local buckling as separate limit states. Because the local buckling and low-cycle fatigue life of braces also correlate to member slenderness, further research is anticipated to evaluate the interaction between these failure modes.

Eccentrically Braced Frames

In an eccentrically braced frame, significant inelastic action is expected in the links in the form of shear yielding, flexural yielding, or shear-flexural yielding. With proper stiffening of the link web, shear yielding is preferred, as both energy dissipation and plastic rotation capacity of the link in this mode are at their greatest. Since this SFRS was first introduced in the 1988 UBC, stringent compactness requirements have been specified for both web and flanges. When a rolled I-shaped section was used, the stringent highly ductile compactness requirement (i.e., λ_{hd}) for the flanges quite often forced the designer to select a heavier section, which unnecessarily increased the web area and resulted in a much larger seismic force demand in the capacity design of beams outside the link, braces, and columns. When short links (defined as links of length no greater than $1.6M_p/V_p$, where M_p and V_p = plastic flexural strength and shear strength, respectively) are used for shear yielding, the web, not the flanges, experiences significant yielding. Based on research on local buckling of links (Richards and Uang 2006a; Okazaki and Engelhardt 2007), the 2005 AISC Seismic Provisions relaxed the flange compactness requirement to be moderately (λ_{md}) instead of highly ductile. In contrast, the seismic compactness requirement for the link web became much more stringent starting with the 2010 provisions. Because this requirement was the same for both the EBF link web and the SMF column web, the artificial shift of the anchoring point mentioned previously for SMF columns, which derived from the study of RBS beams, now also influences the link web compactness requirement. It is not clear if this unintended conservatism is needed.

In evaluating the flange slenderness requirement for links of A992 steel, what might be called an unexpected fracture mode in the web of links not observed in prior tests was reported by Okazaki and Engelhardt (2007). The EBF link cyclic loading protocol, first introduced in the 2005 AISC Seismic Provisions, was used for this test program. A parallel study (Richards and Uang 2006b) found that the loading protocol used was excessively conservative, leading to a revised protocol adopted in the 2010 provisions. Testing with this new loading protocol showed satisfactory response from the A992 links. This study further highlighted how the ductility capacity and failure mode of structural members (links in this case) can be sensitive to the loading protocol used in testing—a topic that has often been discussed in the earthquake engineering community but has yet to be thoroughly researched.

Buckling-Restrained Braced Frames

The buckling-restrained braces (BRBs) that are commonly used nowadays are composed of a structural steel core and a buckling-restraining system that prevents global buckling. To ensure that BRBs are the main source of energy dissipation through axial yielding in both tension and compression, the AISC Seismic Provisions require that BRBs in a BRBF "shall be designed, tested and detailed to accommodate expected deformations." Because BRBs used in actual construction are mainly a proprietary product in the United States, basic research on their development is most often conducted by BRB manufacturers and their detailed designs are confidential. However, a significant amount of BRB research, both experimental and analytical, has been conducted worldwide, especially in Asia in the last two decades.

The AISC Seismic Provisions leave it to BRB manufacturers to proportion the BRB steel core and the buckling-restraining mechanism in ductility design, but demonstration, through a specific cyclic testing protocol, is required to ensure that the BRB has a minimum cumulative cyclic ductility capacity of 200 times the brace axial yield deformation (Sabelli et al. 2003), and the ability to perform as intended up to twice the design displacements (Fahnestock et al. 2003). Notably, this is the only SFRS for which the provisions specify a minimum ductility capacity in terms of a cumulative response parameter (Table 1).

Special Plate Shear Walls

In a special plate shear wall, inelastic action is expected to develop in the form of web (i.e., infill) plate yielding and plastic hinge formation at the ends of the beams (i.e., horizontal boundary elements). In the closing corners of a laterally drifting frame, particularly when thicker infill plates are used, some localized compression and yielding of the infill plates has been reported (Shishkin et al. 2009; Clayton et al. 2015; Dowden and Bruneau 2014). However, this effect is considered of negligible significance for design and is not addressed by the AISC Seismic Provisions.

Table 1. Ductility Capacity Requirements in the AISC Seismic Provisions

SFRS	Deformation-controlled element	Ductility capacity requirement
SMF	Beams, panel zone, column base	Story drift angle ≥0.04 rad without significant strength degradation
EBF	Link	Inelastic rotation capacity ≥0.08 rad for short link ($e \leq 1.6M_p/V_p$); smaller for longer links
BRBF	BRB	(1) Axial deformation capacity at two times design story drift but not <2% story drift, and (2) Cumulative axial deformation capacity $\geq 200\Delta_y$
SCBF	Diagonal braces	Not specified
SPSW	Web plate, horizontal boundary elements	Not specified

Plastic hinging may also occur at the base of columns in developing the complete plastic mechanism of the structure; yielding is otherwise undesirable in columns—particularly shear yielding, which can be accidentally overlooked (it is not intuitive for designers to think of shear yielding governing the design of steel columns because it rarely does so in other steel structural systems).

It is desirable for both web plates and beams to contribute to the total energy dissipation of the system, particularly given that web plates, because of their slenderness, yield only in diagonal tension and thus require progressively larger drifts to maintain their contribution. When plates above and below a beam are of nearly the same thickness, the demands on the beam obtained from capacity design can be insignificant and result in undesirable beam sizes. Changes in the 2016 AISC Seismic Provisions require that SPSW moment frames alone (i.e., without web plates) resist 25% of the specified lateral loads to ensure a desirable minimum contribution of the frame to total system hysteretic behavior. Twenty-five percent was somewhat arbitrary, but is consistent with the strength required by ASCE 7 for back-up moment frames in dual systems. This minimum moment frame strength is in addition to the requirement that web plates be designed to resist 100% of the lateral load by themselves because recent studies have demonstrated this to be necessary to achieve a satisfactory margin against collapse for the R factors provided in ASCE 7 (Purba and Bruneau 2014a, b). Incidentally, the latter requirement significantly simplified design of SPSWs by directly using the simple equation provided in the AISC Seismic Provisions to select web plate thicknesses.

Ordinary moment-resisting connections have been used in many of the SPSWs tested and have performed well, even in SPSWs with lateral drifts of 3% or larger. Not needing special moment-resisting connections to achieve ductile plastic rotation is counterintuitive in the post-Northridge context, but research suggests that the satisfactory performance of ordinary moment connections in SPSWs is attributable to a smaller absolute range of plastic rotations that develops because of a strong directionality in the plastic rotations of plastic hinges when they are constrained to develop at the end of beams (Purba and Bruneau 2012).

Detailing to achieve the described ductile behavior is minimal. Multiple details have been proven adequate to transfer forces from the unstiffened yielding plates to the surrounding members, and beams must be appropriately sized to develop stable plastic hinging (Bruneau et al. 2011). For conventional hot-rolled steel, web plates are not expected to develop significant strain hardening.

Deformation Demand and Capacities for Seismic Design

The expected story drift specified in seismic provisions has evolved over the years. The ATC 3-06 document (ATC 1978) first introduced the deflection amplification factor, C_d, to compute expected story drift, Δ_u (Fig. 1)

$$\Delta_u = C_d \Delta_s \tag{11}$$

The 1988 UBC model code was the first to use a deflection amplification factor, although expressed as $3R_w/8$, where R_w = response modification factor for working stress design. For ductile SFRSs, the elastic story drift ratio, Δ_w (Fig. 1), at the C_w seismic force level was limited to $0.04/R_w$ times the story height for buildings less than 19.8 m (65 ft) in total height. Therefore, the expected story drift ratio was

$$\left(\frac{0.04}{R_w}\right)\left(\frac{3R_w}{8}\right) = 0.015 \tag{12}$$

For buildings taller than 19.8 m, the allowable story drift ratio was $0.03/R_w$ and the corresponding expected story drift ratio was even less (=0.0113). One reason that the pre-Northridge welded flange-bolted web connection was prequalified for SMF applications in the 1988 UBC was the belief that buildings do not drift beyond 2% of their story height (Popov et al. 1989). After Northridge, it was realized that buildings can drift significantly more (Gupta and Krawinkler 1999; Krawinkler et al. 2000). To qualify connections for SMFs, for example, later editions of the AISC Seismic Provisions require connections to be tested to a story drift ratio of 0.04. When nonlinear response history analysis is performed in design, 0.04 is the expected drift ratio at the maximum considered earthquake (MCE_R) level according to the 2016 edition of ASCE 7.

As mentioned earlier, the AISC Seismic Provisions generally do not require designers to explicitly check required inelastic deformations against available deformation capacity. For SMFs, significant inelastic deformation capacity through flexural yielding of the beams and limited yielding of the column panel zones is expected. The provisions require that beam-to-column connections be prequalified through cyclic testing for a story drift angle of 0.04 rad while the beam strength at the column face remains at 80% or more of the beam nominal flexural strength. AISC 358 (AISC 2016a) provides connections that have been prequalified through this process. No explicit check for beam and panel zone deformation capacities is required in design. Similarly, BRBs in a BRBF must be prequalified through cyclic testing for a cumulative ductility capacity of 200 times the brace axial yield deformation. Again, no check of BRB deformation capacity is required in design because it is already prequalified. Links in EBF design are a notable exception to the just mentioned approaches, with checks for link deformation explicitly required. The inelastic angle between the link and the beam outside it is calculated in design and then compared with the allowable deformation capacity, which ranges from 0.08 rad for shorter links to 0.02 rad for long links (Table 1). No specific ductility capacity requirement is provided for SCBFs and SPSWs.

Capacity Design

General

Once the designated primary energy-dissipating elements are designed for the R factor-reduced seismic forces and detailed for ductility, other structural elements are designed, per capacity design principles, with sufficient strength to ensure that the target energy-dissipating mechanism can be achieved in a significant seismic event. The capacity design concept was first developed and implemented in New Zealand in the 1970s for reinforced-concrete structures (Park and Paulay 1975). In the United States, the R and C_d factors, but not the system overstrength factor for capacity design, were introduced in ATC 3-06 in 1978 (ATC 1978). The 1987 edition of SEAOC's "Recommended Lateral Force Requirements and Commentary" (SEAOC 1987) first introduced the $3R_w/8$ for seismic force amplification to amplify the prescribed R_w factor-reduced seismic forces as the required seismic forces for structural components expected to remain elastic. This approach was subsequently adopted by the 1988 UBC. For example, to ensure that steel columns in a frame do not buckle, the required compression force was calculated from the following load combination:

$$1.0P_D + 0.7P_L + (3R_w/8)P_E \tag{13}$$

where P_D, P_L, and P_E = axial forcces produced by the dead, live, and earthquake loads, respectively.

Although a big step forward, $3R_w/8$ as a seismic force amplification factor was flawed because it effectively implied that the ultimate strength of every SFRS was kept at a level equal to three-eighths of the unreduced elastic seismic force (Uang 1993). The 1992 AISC Seismic Provisions, which were based on strength design, rounded the three-eighths to 0.4 and used $0.4R$ as the seismic force amplification factor for capacity design. When the UBC changed its seismic design format from working stress to strength in its 1997 edition (ICBO 1997), the seismic force amplification factor, $3R_w/8$, which was also used as the deflection amplification factor, was replaced by a system overstrength factor, Ω_o. This new factor has been used in ASCE 7 since 1997.

For strength design of structural components expected to experience inelastic action, ASCE 7 specifies what it calls basic seismic load combinations. One of these, included here for illustration, is the following:

$$(1.2 + 0.2S_{DS})D + L + 0.2S + 1.0\rho Q_E \tag{14}$$

where D, L, and S = dead, live, and snow loads, respectively; ρ = redundancy factor; and Q_E = seismic design force level at Point S in Fig. 1.

For capacity design of all other components (to ensure that they remain essentially elastic), seismic load combinations with the overstrength factor are used; the combination corresponding to the load combination is

$$(1.2 + 0.2S_{DS})D + L + 0.2S + 1.0E_{mh} \tag{15}$$

where, conceptually, E_{mh} = seismic force level at Point M in Fig. 1 when the target mechanism is developed.

Because Point M falls in the inelastic response range, the 2010 edition of ASCE 7 provides two approaches to facilitate design. The first, termed the system approach here, considers the SFRS at the system level and simply amplifies the seismic force level at Point S by a system overstrength factor, Ω_o. The second approach, effectively an exception to the first, states that "The value of E_{mh} needs not exceed the maximum force that can develop in the element as determined by a rational, plastic mechanism analysis or nonlinear response analysis utilizing realistic expected values of material strengths." It is the second approach, referred to as the local approach here, that should be used as much as possible; the designer should resort to the system approach only when the local approach is difficult to apply.

One example of the local approach is gusset connection design in an SCBF where diagonal braces serve as the energy dissipation element. Once the diagonal braces are sized, the gusset plate and its connections are designed for the probable maximum forces that the braces can develop. This requires the use of an expected yield stress, R_yF_y, to compute the expected brace tensile and compressive strengths. The use of R_y to compute the expected yield stress for estimating the required seismic force in capacity design was first introduced in the 1997 AISC Seismic Provisions. Another example is SMF moment connection design. Assuming that the inflection point is located at the midspan of the beam, the seismic beam moment (i.e., the effect of E_{mh}) can be established without nonlinear analysis as long as the probable moment at the assumed plastic hinge location is known. The probable beam moment, M_{pr}, is then computed as the product of the plastic section modulus, the expected yield stress, and a cyclic strain-hardening factor [=1.1 per the AISC Seismic Provisions or C_{pr} per AISC 358 (AISC 2016a)]. After including the effect of gravity components, the combined internal forces are used to design panel zones, continuity plates, and joints between beams and columns, and to check strong column-weak beam conditions.

When the local approach is difficult to implement, the system approach is used to amplify the design seismic forces corresponding to Point S in Fig. 1 by an empirical factor, Ω_o. One example is estimating the axial load in an SMF column for the effect of E_{mh}. This is reasonable for estimating column axial forces because these forces, produced by the seismic overturning moment of the SFRS, are more or less proportional to the base shear. However, the local approach should not be applied blindly. A good example of its misuse is determining the effect of E_{mh} on beams in an SCBF with inverted-V bracing. The tensile brace is expected to reach its expected tensile yield strength, and the compressive strength is expected to buckle and then significantly degrade in the postbuckling region. It is the postbuckling strength of the brace, not its prebuckling strength, that governs the design of the beam. The resulting unbalanced vertical force that the pair of braces imposes on the beam midspan cannot be captured by the system approach because the unbalanced vertical load from an elastic analysis cannot reflect the postbuckling scenario no matter how large the Ω_o value is used.

Use of either approach for capacity design was often up to the judgment of the designer. To avoid any potential misuse of the system approach as mentioned previously, the 2016 editions of ASCE 7 and the AISC Seismic Provisions use what is termed the capacity-limited horizontal seismic load effect, E_{cl}, to spell out explicitly when the local approach is required for seismic steel design. The Ω_o approach is not permitted when E_{cl} is specified in the 2016 AISC Seismic Provisions.

The 2005 edition of the AISC Seismic Provisions also introduced the R_t factor for calculating the expected tensile strength of steel. For capacity design, this factor is mainly used to compute the design strength, not the required strength, for limit states within the same member from which E_{cl} is determined.

Moment Frames

In some ways, the Northridge problem associated with the brittle fracture of moment connections was an issue of capacity design, not ductility design. On one hand, the expected beam flexural strength was underestimated because the concept of expected yield strength did not exist before the Northridge earthquake. Also, the significance of the beam-to-column welded joints, a critical component of capacity design, was overlooked in terms of design, fabrication, welding, and inspection. Triggered by Northridge, the SAC Joint Venture was probably the only coordinated and directed research that dealt with one particular SFRS (i.e., steel moment frames) as a national effort. Significant findings were documented in a series of recommended design criteria (FEMA 2000b, c, d, e), state-of-the art reports (FEMA 2000f, g, h, I, j, k), and in several background documents.

Notably, many steel moment frame buildings with pre-Northridge moment connections still exist in high seismic regions. Recommendations on seismic retrofit are available (FEMA 2000d), but studies on connection retrofit are still limited (Gross et al. 1999; Malley et al. 2006). Pre-Northridge SMFs designed per the 1988 UBC can have undesirably weak panel zones, and their retrofit can be challenging (Kim et al. 2015). The weak column-strong beam design condition was also more prevalent per the 1988 UBC (Roeder et al. 1993; Schneider et al. 1993). Moreover, challenges exist in retrofitting column splices with low-notch-toughness partial-joint-penetration (PJP) groove welds. Many older steel buildings were designed with little or no consideration of seismic behavior and used (partially restrained) riveted steel connections encased in massive but lightly reinforced concrete for fire protection. Research on seismic modeling and retrofit of buildings with riveted connections is, again, limited (e.g., Leon et al. 1994; Roeder et al. 1996; Bruneau and Sarraf 1996).

Concentrically Braced Frames

The definitions of expected brace strength in tension, expected brace strength in compression, and corresponding postbuckling strength for capacity design have varied somewhat over the years (Table 2). As far as determining force demands for structural components that need to be capacity-protected, the most significant changes for concentrically braced frame systems occurred in the 1997 Seismic Provisions, where the use of a postbuckling strength equal to 30% of the brace strength in compression was introduced. At first, this concept was used to design beams in an SCBF with V or inverted-V bracing configurations. Only in the 2010 edition of the AISC Seismic Provisions was the 30% rule generalized to also compute the required strength of SCBF columns, beams, struts, and connections. Other than singling out inverted-V bracing, the provisions now require designers to consider two scenarios independently:

- An analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension; and
- An analysis in which all braces in tension are assumed to resist forces corresponding to their expected strength and all braces in compression are assumed to resist their expected postbuckling strength.

Whether these two scenarios cover the worst case remains to be investigated. Taking the two-story X-braced configuration with braces of the same size, for example, the beam at the midspan is not subjected to any unbalanced vertical load unless the first scenario in the upper story and the second scenario in the lower story are combined, or unless a scenario is considered in one story together with a percentage of those values in the other story (Bruneau et al. 2011).

Table 2. AISC 341 Expected Brace Strengths for SCBF Capacity Design

Edition	Expected tensile strength	Expected compressive strength: Initial buckling	Expected compressive strength: Postbuckling coefficient
1990, 1992	$F_y A_g$	$0.8(\phi_c F_{cr} A_g)$	NA
1997	$F_y A_g$	$\phi_c F_{cr} A_g$	0.3
2002	$R_y F_y A_g$	$\phi_c F_{cr} A_g$	0.3
2005	$R_y F_y A_g$	$F_{cr} A_g$	0.3
2010, 2016	$R_y F_y A_g$	$\left(\frac{1}{0.877}\right) F_{cre} A_g \leq R_y F_y A_g$	0.3

Brace connections can be designed to withstand the flexural forces or rotations imposed by brace buckling, although design to withstand rotations is more common. No matter whether the brace is designed to buckle out of plane (Astaneh-Asl et al. 1986) or in plane (Tsai et al. 2013), accommodation of inelastic rotation at the brace ends is typically accomplished by means of a single gusset plate with the brace terminating a short distance before a line of restraint to form a linear hinge zone. A more compact gusset design with an elliptical hinge zone was proposed by Roeder et al. (2011). The gusset connection must be capacity-protected and is usually designed based on the uniform force method (AISC 2017). However, the welds of the gusset plate are prone to fracture because this method does not consider the brace-buckling deformation demands on the connection and the frame action (see the section "Buckling-Restrained Braced Frames" for discussion of frame action in the gusset plate.) To avoid weld fracture and to improve system ductility, Roeder et al. (2011) also proposed a design procedure to balance brace yielding and gusset yielding. Carter et al. (2016) proposed a procedure for sizing welds between the gusset plate and the beam or column when the brace is designed to buckle out-of-plane.

Based on research by Imanpour et al. (2016a, b), Stoakes and Fahnestock (2016), and Imanpour and Tremblay (2017), the 2016 AISC Seismic Provisions also introduced special combinations for the capacity design of multitier braced frames (defined as braced frames having two or more levels of bracing between diaphragm levels or locations of out-of-plane bracing). These combinations consider multiple combinations of the two previously mentioned scenarios in brace tiers to determine the worst-case scenario, torsional moments about the brace-buckling axis, and provide special requirements for in-plane column flexural stiffness to prevent premature brace fracture. Similar requirements are prescribed for multitier braced frames having BRBs.

Eccentrically Braced Frames

Because links in an EBF serve as energy dissipation components, diagonal braces and their connections, beams outside the links, and columns need to be capacity-protected. The expected shear strength of the link considered for this purpose is

$$V_l = \alpha(R_y V_n) \tag{16}$$

where α = strain-hardening factor. Experiments have shown that yielding links can exhibit greater strain hardening than flexural plastic hinges in beams (Kasai and Popov 1986b). Okazaki et al. (2005) showed that rolled wide-flange links constructed of ASTM A992 steel have a strain-hardening factor, α, ranging from 1.2 to 1.45. Tests on smaller rolled wide-flange links constructed of ASTM A36 steel sometimes showed a strain-hardening factor in excess of 1.5 (Hjelmstad and Popov 1983; Engelhardt and Popov 1989). Recent tests on very large welded built-up wide-flange links for use in major bridge structures have shown strain-hardening factors close to 2.0 (McDaniel et al. 2003; Dusicka and Itani 2002). The AISC Seismic Provisions refer to adjusted instead of expected link shear strength for capacity design to include the effect of material overstrength and cyclic hardening. Instead of the expected link shear strength with a high strain-hardening factor for capacity design, the provisions use the adjusted link shear, the main motivation being to make EBF design and construction feasible and not to penalize it by the high strain-hardening factor.

Intended for rolled wide-flange links, the AISC Seismic Provisions start with a hardening factor of 1.5 and, borrowing from the inherent safety margin provided by $R_y = 1.1$ and $1/\phi_c = 1.1$ in brace design, adjust the α value in Eq. (16) downward to 1.25 ($\approx$1.5/1.21). This relaxation is not used elsewhere in the provisions.

Designing the beams outside the link is challenging because these beam segments, which are usually an extension of and the same size as the link, are subjected to high axial compression and bending moment. The AISC Seismic Provisions adjust the α value further from 1.25 to 1.1, reasoning that (1) allowing the beams to experience limited yielding near the link ends is not detrimental, (2) composite action due to the presence of a slab not explicitly accounted for increases beam flexural strength (Ricles and Popov 1989), and (3) reinforcing the beam outside the link, say with welded cover plates, may increase the possibility of fracture.

When an EBF is configured for link-to-column connections, those connections are subjected to both high shear and high moment. Prior to the 1994 Northridge earthquake, link-to-column connections were typically constructed like SMF moment connections. Testing conducted after Northridge confirmed that these connections were vulnerable to brittle fracture (Okazaki et al. 2009). As a result, the AISC Seismic Provisions now require the connections to be either prequalified or qualified by cyclic testing. The provisions have yet to provide any prequalified connections. At least two connection details, one for shop-welded and one for field-welded, have been proposed (Okazaki et al. 2015; Hong et al. 2015). Further research on this subject is needed.

Buckling-Restrained Braced Frames

Capacity design of buckling-restrained braced frames (BRBFs) is similar to that of SCBFs, with the major difference being that only one brace strength scenario need be considered. Also, whereas initial compression strength is generally lower than expected tensile strength for an SCBF brace, the opposite is true for a BRB. Taking the inverted-V braced configuration as an example, this implies that the unbalanced force acting at the midspan of the beam is downward for an SCBF and upward for a BRBF; the unbalanced force for the latter is also much smaller.

At the component level, BRB capacity design is needed to avoid undesirable failure modes. In particular, the restraining casing, which receives little (ideally no) axial force, should be designed to be stiff enough that its elastic buckling strength is at least equal to the BRB's maximum strength (Watanabe et al. 1988). In addition, the steel core is expected to experience higher-mode buckling and, so deformed, exerts out-of-plane forces to the restraining casing, possibly causing it to bulge if not properly designed (Takeuchi et al. 2010; Wu et al. 2014; Lin et al. 2012, 2016).

Although the technology for producing reliable BRBs with excellent cyclic performance at the component level is mature, frame-level tests that include beams, columns, BRBs, and gusset connections have highlighted the importance of other factors that may hinder performance if not explicitly addressed.

First, buckling of the BRB-to-gusset joints, if they are not properly designed and detailed, can cause out-of-plane buckling of the BRBs themselves (Tsai et al. 2002; Mahin et al. 2004; Tsai et al. 2008; Chou et al. 2012; Palmer et al. 2014). It has been shown that Thornton's column strip method (Thornton 1991), which uses an effective length factor of 0.6 (AISC 2012), is not conservative and so a much larger value (2.0) has been proposed (Tsai and Hsiao 2008; Chou et al. 2012; Wei and Bruneau 2017a, b; Westeneng et al. 2017). It has also been demonstrated that adding free-edge stiffeners is an effective way to avoid gusset buckling (Tsai and Hsiao 2008). Takeuchi et al. (2014, 2016) proposed a gusset stability criterion based on bending moment transfer capacity at the restrainer's ends.

Second, gusset plates at the beam-to-column intersection produce shorter clear column length, which not only can cause shear yielding in the columns (Mahin et al. 2004) but also, and more significantly, can cause the gusset plate to buckle when the brace is in tension because of the frame action pinching of the gusset between the beam and the column. Two solutions have been proposed to address this issue. The first is a mechanism to minimize the frame action—for example, beam splicing outside the gusset region (Fahnestock et al. 2007). Printz et al. (2008) showed through finite-element simulation that maximum gusset stresses are significantly reduced and that beam splices have negligible impact on drifts. The second approach is to quantify the force demand on the gusset plates due to frame action and design the gusset plate to meet it. Palmer et al. (2014) showed that, when a detail like beam splicing is not used, significant inelastic deformation demands in the beams and columns at the gusset connection region cause buckling of beam webs and flanges, resulting in out-of-plane BRB rotation. Designing gusset connections based on the uniform force method alone does not prevent damage either to the gusset plate or to the welded joints to the beam and column. Procedures for calculating the stresses produced by frame action have been proposed (Chou et al. 2012; Lin et al. 2014, 2016). Zhao et al. (2012, 2016) investigated the effect of frame action on BRB in-plane stability, finding that the frame action may cause the protrusion ends of the core to yield prematurely and buckle in the end zones.

More contemporary research is focusing on the adequacy of BRB end connections and gusset details to accommodate their out-of-plane demands under bidirectional earthquake excitations. This topic is relevant to other types of similarly connected braces in braced frames (Khoo et al. 2016; Wei and Bruneau 2017a, b).

Special Plate Shear Walls

Capacity design issues in special plate shear walls involve better determining demands on boundary elements. Research that has clarified demands on beams (Purba and Bruneau 2014c) and columns (Berman and Bruneau 2008b, c) has led to changes in the 2010 and 2016 AISC Seismic Provisions. In addition, a minimum required stiffness has typically been specified for the boundary elements, which, initially introduced to prevent undesirable column behavior (Montgomery et al. 2001), was eventually extended to be applicable to top beams (Dastfan and Driver 2008). Subsequent research showed that this stiffness requirement serves only indirectly to ensure that yielding of the entire web plate occurs at the expected seismic drifts (Qu and Bruneau 2010a) given that adequate column behavior can be achieved even when violating the stiffness requirement (Lee and Tsai 2008).

Forces applied by the infill plates, being tension only, have been specified by the AISC Seismic Provisions to be equal to the expected yield strength of the steel, but recent research has indicated that compression forces may develop in some circumstances and therefore deserve consideration (Shishkin et al. 2009; Clayton et al 2015; Dowden and Bruneau 2014). Intertwined with this issue is the angle of the diagonal strips used in simplified analysis models to obtain realistic or conservative demands on beams and columns for the design of these boundary elements. This is still the subject of deliberations (Webster et al. 2014; Fu et al. 2017).

Although neither the original intent of nor required by the AISC Seismic Provisions, RBS connections have been used in some instances at the ends of beams to achieve smaller beam sizes. Qu and Bruneau (2010b) showed that plastic hinging does not develop exactly in the middle of the RBS in SPSW applications; they provided an equation to account for the plastic hinge demand that can be used for capacity design purposes.

Column Issues

Required Force for Capacity Design

Columns play a critical role in preventing building collapse in a seismic event. Capacity design of steel columns in high seismic regions was first introduced for SFRS (both moment and braced) in the 1988 UBC, which used the response modification factor, R_w, to compute the base shear for working stress design. The force amplification factor, $(3/8)R_w$, was used to amplify the column axial force produced by the prescribed seismic forces, which effectively meant that the seismic axial load corresponded to that produced by 37.5% of the unreduced seismic forces (i.e., $R_w = 1$) and was independent of SRFS type (Uang 1993). The intent was to prevent global column failure. For LRFD strength design, the 1992 AISC Seismic Provisions rounded 37.5% to 40% and specified the following load combination to check for flexural buckling:

$$1.2P_D + 0.5P_L + 0.2P_S + 0.4R(P_E) \leq \phi_c P_n \quad (17)$$

For SMFs ($R_w = 12$), the value of 4.5 for $3R_w/8$ was a very large force amplification factor when it was first introduced. Most likely its impact on design practice at the time was the reason that the 1988 UBC waived the requirement to account for moments concurrently acting on columns in capacity design. With minor modifications, this approach has continued in the AISC Seismic Provisions, although the force amplification factor has been replaced by the SFRS-dependent system overstrength factor, Ω_0. Eq. (17), per 2016 ASCE 7, can thus be rewritten as follows:

$$(1.2 + 0.2S_{DS})P_D + 0.5P_L + 0.2P_S + \Omega_o(P_E) \leq \phi_c P_n \quad (18)$$

This requirement applies to all SFRS columns.

Columns in moment frames are commonly subjected to modest axial loads but high moments because a story drift up to 4% of story height is expected. For braced frames (e.g., SCBFs, EBFs, BRBFs), columns are subjected to high axial loads; the seismic moment is small in the elastic range, but can increase significantly once nonuniform inelastic drifts occur along the building height. Nonlinear time history analysis shows that moments in braced frame columns can be significant in a seismic event. Research is needed to evaluate (1) if the capacity design practice of ignoring moment in checking column strength to prevent global column failure is appropriate, and (2) to what extent inelastic action is permitted if columns do not have to remain elastic.

Research has shown the significant role played by continuous seismic and gravity column stiffness to better distribute yielding along the building height and decrease the possibility of large drift concentrations (Tremblay and Stiemer 1994; MacRae et al. 2004). Canadian Standard CSA-S16-14 (CSA 2014), unlike the AISC Seismic Provisions, requires columns in a braced frame to be continuous over at least two stories. Further research is needed to establish simple rules for all SFRSs. At the extreme, reliance on rigid walls pinned at base has been proposed and shown to be effective in controlling the deformation pattern of the frame and hence in avoiding weak story failure (Mar 2006; Qu et al. 2012). A steel truss configuration using the same concept has also been proposed to provide a so-called strongback in braced frames (Tremblay and Poncet 2007; Lai and Mahin 2015).

Ductility Capacity and Deep Columns

Cyclic testing of steel columns prior to the 1994 Northridge earthquake, mostly at reduced scale, was limited (Popov et al. 1975; Mitani et al. 1977; MacRae et al. 1990; Nakashima et al. 1990; Schneider et al. 1993). After Northridge, a significant number of full-scale tests of beam-column subassemblies were conducted to evaluate moment connection performance. However, although cyclic performance of wide-flange beams was intensively researched, this was not the case for columns. Except for panel zones, columns almost always remained elastic in these moment connection tests. Most of the tests did not apply axial compression to the columns, partly because doing so was challenging.

Shallow columns (e.g., W12, W14) are common in braced frames. Because they can be subjected to high axial loads, Newell and Uang (2008) tested nine full-scale W14 columns with axial forces up to $0.75P_y$ and reported that they achieved story drift capacities of 0.07–0.09 rad. These large deformation capacities were, in part, the result of delay in flange local buckling caused by the stabilizing effect of the stocky column web ($h/t_w = 6.9$–17.7). However, numerical simulation of deep (W27) columns showed that local buckling can significantly affect cyclic ductility capacity (Newell 2008).

To achieve an economical design, deep columns are often preferred to meet code-specified story drift limits in SMF construction. Concerns about deep columns were first reported by Chi and Uang (2002). They tested beam-column subassemblies with RBS moment connections that used W27 columns and concluded that this type of column is prone to twisting; once lateral-torsional buckling of the beam occurs, the compressive beam flange force together with its out-of-plane movement applies torsion to the column. Chi and Uang also showed that vulnerability for twisting is highly related to the h/t_f^3 ratio, where h is the centerline distance between flanges and t_f is the flange thickness of the column.

Because of concerns that column bases in an SMF are expected to form plastic hinges, and that the width-thickness ratios of the deep sections are significantly larger, a research program (NIST 2011) was initiated that focused on the behavior and design of deep columns for SMF design. As part of this program, deep columns (W24 sections with h/t_w ratios ranging from 28.7 to 54.6) subjected to axial load and cyclic lateral drift confirmed that plastic rotation capacities are significantly lower (Ozkula et al. 2017). Most columns experienced in-plane plastic hinging with significant local buckling and axial shortening. Specimens with the most compact sections experienced not only in-plane hinging but also out-of-plane lateral-torsional buckling at an axial load as low as 18% of the yield strength of the column. This phenomenon was not previously known.

For plastic design, AISC 360 (AISC 2016c) limits the axial column compressive force to $0.75P_y$ for columns with plastic hinges. For seismic applications, the AISC Seismic Provisions limit the axial compressive force to $\phi_c P_n$ for capacity design; the axial force is also limited to meet the web compactness requirement (see the Appendix). Based on Popov et al. (1975), ASCE 41 (ASCE 2013) limits the column axial force to $0.5P_y$, beyond which the ductility capacity cannot be counted on. Significantly, the axial force in ductile moment frames is restricted to $0.3P_y$ and $0.5P_y$ in Canadian (CSA 2014) and New Zealand (NZS 2007) standards, respectively. Also, the AISC Seismic Provisions provide a limiting L/r_y ratio for SMF beams but no similar limit is provided for columns. As active research continues on deep columns (Zargar et al. 2014; Fogarty and El-Tawil 2016; Elkady and Lignos 2015, 2016), it is expected that its findings will impact the design and modeling of steel columns in the next editions of the seismic codes.

Column Splices

PJP groove welds were commonly used for column splicing before 1994. Limited full-scale testing of large column splices showed that

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their ductility capacity is insignificant although their design strength can be developed (Popov and Stephen 1977; Bruneau and Mahin 1991). Nominal design strength usually could be exceeded in testing, mainly because of the lateral restraint effect of the PJP welds (Gagnon and Kennedy 1989). The lack of ductility in partial penetration joints is mainly attributable to the notch-like condition created by the unwelded portion of the joint.

Computation of the required strength for column splice design pre-Northridge was also based on elastic analysis, where the inflection point generally occurred close to the splice location. Research conducted after the Northridge earthquake showed, however, that the actual force demand at the splice location in an SMF can be significantly higher because of a shift in inflection point once the structure deforms into the nonlinear range. (Shen et al. 2010). The 1997 AISC Seismic Provisions continued to permit PJP welds, but specified that the required force must be doubled when a tensile force was predicted based on the amplified seismic load combination. Furthermore, a minimum required splice strength equal to 50% of the expected column flange yield force was specified to account for design uncertainty in predicting the inflection point location. In 2005 the provisions began to require expensive CJP welds for SMF column splices. Via a fracture mechanics approach, together with experimental verification of full-scale column splices using modern, toughness-rated weld filler materials and welding practice (Shaw et al. 2015; Galasso et al. 2015), this stringent requirement was relaxed in the 2016 provisions, which permit PJP welds but only when meeting very specific design requirements.

Column Bases

Although column bases play a key role in transferring seismic forces from a structure as a whole to its foundation, research on their seismic performance and design has been limited compared with other connection types. According to a synthesis of relevant seismic research (Grauvilardell et al. 2005), column base-plate connections can be classified as exposed and embedded; the latter can be further divided into deep and shallow. Research to date has mainly focused on exposed connections (Astaneh et al. 1992; Burda and Itani 1999; Fahmy et al. 1999; Myers et al. 2009; Gomez et al. 2011; Kanvinde et al. 2012), although Cui and Nakashima (2011) and Barnwell (2015) studied shallow-embedded connections and Rodas et al. (2017), Grilli and Kanvinde (2017), and Grilli et al. (2017) studied deep-embedded connections. Much of this research is ongoing, and findings are likely to affect future seismic design provisions.

Built-Up Box Columns

Space SMFs with cold-formed HSS columns and moment connections in both directions at all columns are common in multistory construction in Japan (Nakashima et al. 2000), whereas built-up box columns are used for high-rise construction. In the United States, the last few decades have seen a series of planar SMFs with wide-flange columns used to promote strong-axis bending and prevent biaxial bending. When biaxial bending is unavoidable, design engineers use boxed wide-flange columns, flanged cruciform columns, or built-up box columns. Built-up columns often require internal continuity plates (or diaphragm plates), and it is common to weld at least one side of each plate using electroslag welding (ESW). Because the inside of the built-up box column is not accessible after welding, significant research has been conducted in Asia, especially in Japan and Taiwan, on vulnerability to brittle fracture resulting from the notchlike condition created by ESW (Song et al. 2011; Tsai et al. 2015). Testing conducted by Anderson and Linderman (1991), Kim et al. (2008), and Uang et al. (2014b) also revealed the same problem. Research on this topic in the United States is very limited.

Performance-Based Design

As indicated previously, the use of R factors to design structural systems bypasses the need to determine ductility demands; the use of capacity design bypasses the need to assess ductility capacities. Although it is convenient to design SFRS according to the AISC Seismic Provisions, in some situations (e.g., exceptional applications, innovative structural systems, seismic evaluations of archaic systems), nonlinear time-history analysis and other tools may be needed. Performance-based design has often been used not only for this purpose but also for qualifying a broader range of earthquake engineering activities. A review of research, developments, and codification in this area is beyond the scope of this paper, but a few words must be said about the approach specified in ASCE 41 (ASCE 2013), which most significantly uses FEMA-356 (FEMA 2000a) and is primarily employed in the seismic evaluation of buildings.

The performance-based design presented in ASCE 41 and FEMA-356 expresses the nonlinear behavior of various structural systems in terms of an idealized nonlinear force-deformation relationship (or moment-rotation or other relationship relevant to the ductile structural element considered), as shown in Fig. 5. Such nonlinear relationships are typically the backbone curves traced on top of experimentally obtained hysteretic curves; these backbone curves are combined with empirical judgment to establish the performance limits of immediate occupancy (IO), life safety (LS), and collapse prevention (CP). Several empirical correction factors account for a number of effects (such as pinching in the actual hysteretic curves) on the demands and capacities to be compared.

Although nonlinear analysis is becoming popular, its complexity sometimes creates the impression of high reliability (in strain demand prediction for example) that in fact may be beyond its capabilities. Earthquake engineering research on structural elements has typically focused on the evolution of hysteretic curve shapes and behavior in terms of drifts or rotation (or, more rarely, in terms of strains), but research to correlate strains, drifts, or rotation limits for steel structures can be more challenging. For example, a tension-only braced frame, with flat-bar braces having a slenderness ratio, KL/r, of approximately 800 (effectively buckling immediately in compression), exhibits the most highly pinched hysteretic behavior. However, its backbone hysteretic curve is a nearly ideal bilinear relationship up to strain likely exceeding 10%. Frame ductility capacity alone does not tell one how to set IO, LS, and CP limits. Failure modes that involve fracture introduce

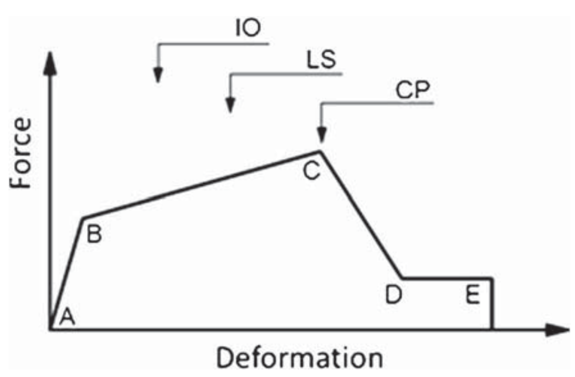

Fig. 5. Generic nonlinear force-displacement relationship used in ASCE-41 analyses

additional complexity and uncertainty because the macro-models typically used in earthquake engineering do not account for behaviors driven by fracture mechanics or dislocations in crystal structures (Kanvinde 2017).

Finally, performance-based design requirements established for the retrofit of existing structures are sometimes less stringent than requirements for new buildings in order to make retrofit economical. These less stringent requirements must be identified and assessed based on the application at hand because they may not be appropriate for new construction. Furthermore, because nonlinear time histories are required instead of an R factor-based design, attention must be paid to modeling issues. These issues are beyond the scope here, but useful guidance is available elsewhere (NIST 2010, 2017; PEER 2017; LATBSDC 2017).

Proprietary Components

A significant change in recent years has been the explicit inclusion of proprietary connections and elements in the AISC Seismic Provisions and in seismic specifications in general. The emergence of proprietary components can be attributed in part to the Northridge earthquake. The rapid adoption of proprietary buckling-restrained braces, beginning in the late 1990s in the United States, was followed by the introduction of design requirements in the 2005 AISC Seismic Provisions. However, as far as moment-resisting connections are concerned, many more proprietary moment connections have since found acceptance.

FEMA-350, "Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings," published a few years after the Northridge earthquake (FEMA 2000c), summarizes the findings of extensive studies conducted following the earthquake and provides comprehensive design and detailing provisions for seven prequalified moment-resisting connections. Six of its 224 pages are devoted to proprietary systems (not prequalified), referring the reader to manufacturers for information on patented connections used in actual projects. The main characteristics of only four such systems are described: (1) the SidePlate connection, (2) the slotted web connection, (3) the cast-steel Kaiser bolted bracket connection, and (4) the reduced web connection.

No prequalified connections were included in the 2005 edition of AISC 358 (which is referenced by standard specifications and readily accepted by building officials), but they were progressively introduced over the next decade. The latest edition, AISC 358-16, includes the Kaiser Bolted Bracket connection, the SidePlate connection, the Simpson Strong-Tie Strong Frame connection, and the ConXtech ConXL connection with concrete-filled square HSS or built-up box columns. The significant integration of proprietary systems into seismic design specifications for steel structures is unprecedented and is likely to continue as manufacturers vie for recognition.

Innovative Systems

This paper has so far focused on shortcomings in knowledge related to the conventional steel structural systems used in nearly all steel buildings designed and built in seismic regions (if only by virtue of being referenced in codes and specifications). However, an entirely different state-of-the-art paper could be written on innovative steel systems for seismic applications based on much recent research. A thorough overview cannot be provided here, but selected developments are briefly summarized, arbitrarily divided into variations based on existing systems and variations based on alternative designs. The line between the two is at times blurred.

Variations on Existing Systems

Many variations on components of existing structural systems have been proposed to improve seismic performance. One variation per type of structural system, drawn from North American research, is described.

Concentrically Braced Frames

To enhance ductility, and to distill the unbalanced force to be considered in capacity design down to the difference between the tension and compression strengths of braces in CBFs, a specially detailed fuse was created by locally reducing the brace cross-sectional area. The fuse area is confined by cold-formed C shapes and flange cover plates to prevent cross-sectional distortions during brace buckling (Vincent 2008; Egloff and Tremblay 2012).

Eccentrically Braced Frames

To enhance EBF resilience, a bolted link was developed that can be unbolted and replaced following earthquake damage, thereby facilitating return to service (Mansour et al. 2011). This innovation makes possible the use of link sizes different from beam sizes, which facilitates capacity design of the link beam.

Buckling-Restrained Braced Frames

To provide an improved sequence of plastification in BRBs, a hybrid BRB was developed that replaces the regular-steel core of conventional BRB designs with a so-called multicore built using both low-yield-point and high-performance steel. When subjected to earthquake excitations, a BRB frame benefiting from the two steel cores acting in parallel develops lower residual drifts for greater reliability against collapse (Atlayan and Charney 2014).

Special Plate Shear Walls

Building on the fact that a ring deforms into an oval when subjected to tension forces in its plane, with the shortening in the compression direction equal to the elongation in the tension direction, ring-shaped steel-plate shear walls have been developed where the infill plate of the wall is cut to create a pattern of interconnected rings. Analytical and experimental studies have demonstrated that this innovation allows SPSWs to exhibit fuller hysteretic loops even in the absence of rigid beam connections (Egorova et al. 2014).

Alternative Designs

A number of structural devices have been developed that rely on triangular plates or tapered shapes to dissipate hysteretic energy. Bruneau et al. (2011) provide examples. Triangular plates are favored because their width when bending out of plane matches the profile of the bending diagram This results in simultaneous yielding over the entire length of the plates that requires less inelastic strains at large relative displacements. The outcome is particularly good hysteretic energy dissipation (Steimer et al. 1981; Tyler 1978). Two device configurations are noteworthy because they have been implemented in a number of buildings internationally. A added damping and stiffness (TADAS) device made of multiple parallel triangular plates, developed in the 1990s, is used (mostly in Taiwan) as a flexural-beam damper (Tsai et al. 1993). An alternative is the scorpion brace, in which triangular plates act as specially designed fingers in flexure at the end of a cast-steel claw used as a brace connection (Gray et al. 2014). The scorpion brace promotes symmetric hysteretic behavior (similarly to that exhibited by BRBs but relying on an altogether different yielding mechanism).

Considerable literature exists, going back to Housner (1963), on the rocking response of various types of rigid or flexible structures and structural elements during earthquake excitations. Steel frames have been considered in many of the more contemporary analytical and experimental studies of rocking systems, such as Pollino and Bruneau (2007, 2010a, b) on braced towers with energy-dissipating devices at their base, Eatherton et al. (2014a, b) on tested pairs of braced frames linked by energy-dissipating elements and having posttensioned strands tying down the top of the frame, Pollino et al. (2017) on retrofitting concentrically braced frames to prevent soft story formation, and Gledhill et al. (2008) on tying down rocking frames relying on Ringfeder friction springs installed at the column bases to control frame leg uplift amplitude. In all cases, rocking about the base of the structure is allowed to limit the maximum force transmitted to the structure itself, akin to seismic base isolation.

As a variant on rocking systems, researchers have investigated the seismic response of steel moment frames with beams posttensioned between columns and relying on the rocking action of the beams onto the column faces. Like the previously mentioned rocking systems, structural systems using posttensioned connections have the benefit of self-centering after an earthquake excitation. Implementation in steel frames includes various tendon configuration layouts and reliance on friction, viscous, or yielding steel devices to provide energy dissipation (Christopoulos et al. 2002a, b; Garlock et al. 2005, 2007; Kim and Christopoulos 2008; Pekcan et al. 2000; Ricles et al. 2001, 2002; Rojas et al. 2005). This concept has also been used to develop self-centering steel plate shear walls (Dowden et al. 2016; Dowden and Bruneau 2016).

Finally, although not per se a ductile behavior attributable to the plasticity of steel, friction has been used to provide energy dissipation in various structural steel systems. There are many challenges in achieving relatively stable repeatability and smoothness of hysteretic curves generated by friction, as well as the amount of energy dissipated for a given displacement amplitude, because these naturally depend on many factors (Bruneau et al. 2011). Nonetheless, a sliding hinge joint (SHJ) relying on friction was developed by Clifton (1996, 2005) and has been implemented in buildings in New Zealand (Gledhill et al. 2008). A proprietary friction-braced system was tested in the 1980s (Filiatrault and Cherry 1987, 1988; Kelly et al. 1988). Other examples include a patented so-called pin-fuse connection proposed for use in steel moment-resisting frames (Cordova and Hamburger 2011; Sarkisian et al. 2011) and a self-centering energy-dissipating steel brace (Christopoulos et al. 2008; Chou and Chen 2015).

Structures Other Than Buildings

Although the focus of this paper has been on steel buildings, the philosophy outlined in it is similarly (but to different degrees) embedded in seismic design provisions for other engineered steel structures, with some adjustments dictated by special characteristics and requirements. For example, for bridge bents plastic hinging has traditionally and deliberately been allowed to develop in columns rather than beams because their loads and geometry unavoidably require beams to be much stiffer and stronger flexurally than columns.

Seismic design provisions already address bridge substructures consisting of steel moment-resisting or braced bents. Even though not included in most bridge design specifications, buckling-restrained braces have been used in special bridge applications and are being studied for potential implementation in bridge bents (Wei and Bruneau 2017a). Eccentrically braced towers have been used in the temporary support structure of the new San Francisco-Oakland Bay Bridge (Dowdell et al. 2015). Ductile steel applications germane to bridges (such as ductile end diaphragms and cross frames) have been developed and implemented. An overview of these design requirements and some implementations is presented in Uang et al. (2014a).

Future Directions—Impact of the Christchurch Earthquakes

As much as the Northridge earthquake had a defining influence on the seismic design of steel structures, it is foreseen that the Christchurch 2010–2011 earthquake series will have a major impact on future seismic design practice although it may take more time for this impact to be realized. The seismic performance of the few steel buildings in Christchurch at the time of the earthquakes generally exceeded that of many other structural systems (Bruneau et al. 2011; Clifton et al. 2011; MacRae et al. 2015). Although not all of these buildings were undamaged, their repairs were expedient. To some extent, and partly because of public perception, this has affected the choice of structural systems during the reconstruction of the city. The fact that two reinforced-concrete buildings collapsed during the earthquake, the years of arguments on whether repairs of concrete buildings could bring them to an as-new condition (or do so at less cost than rebuilding), and the subsequent demolition of hundreds of nonsteel buildings—all of these contributed to the favoring of steel structures during reconstruction to an extent never before seen in New Zealand or anywhere else following major earthquakes.

The trend just mentioned can be qualitatively observed by anyone perusing websites on Christchurch's reconstruction or walking the streets of the city's Central Business District. It has also been quantitatively documented that, whereas before the earthquakes almost all buildings in the Christchurch Central Business District and the Addington area had reinforced-concrete frames or walls as their lateral force-resisting systems, the floor areas of rebuilt buildings with steel, concrete, and timber lateral force-resisting systems have been in the ratio of approximately 79:20:1 because steel floors tend to be found in the larger structures (Bruneau and MacRae 2017). Furthermore, in rebuilt concrete buildings three-quarters of the internal gravity frames are of structural steel.

It is not the striking shift from reinforced-concrete construction to steel construction in Christchurch that is most significant; rather, it is that a major driver of this shift was the desire to build resilient or low-damage structures. For this reason, reconstruction has not only emphasized steel structures but has led to the use of existing and innovative/emerging structural systems intended to make new buildings (and indirectly Christchurch as a whole) more seismically resilient. The performance of the structural systems in these new buildings during future earthquakes will certainly be critically assessed. In any case, tolerance to damage and delayed repairs is expected to be low. It will also be interesting to see if the observed trends persist as the earthquakes fade from collective community memory in the years and decades to come.

The Christchurch experience is perhaps unique today, but it is likely to repeat itself in other similarly developed cities worldwide after future devastating earthquakes. Christchurch's rebuilding is thus most significant particularly because one of the city's declared goals is to emerge stronger, smarter and more resilient to the physical, social, and economic challenges (Christchurch City Council 2016). Bruneau and MacRae (2017) provide insight into some of the factors that dictate structural engineering decisions during post-earthquake reconstruction of a modern city. Christchurch as a model will have a huge impact on the seismic design of steel structures.

Conclusions

Seismic steel structure design has evolved to a point where it embodies comprehensive requirements both for the detailing of designated yielding components and for capacity design protection of a structure as a whole. However, the paths taken to arrive to this point leave unresolved a number of inconsistencies that have been highlighted in the text:

- Slenderness limits for flange local buckling, web local buckling, and lateral-torsional buckling are specified independently of each other, although they interact in influencing the global ultimate behavior of steel members;
- Specification of slenderness limits independent of application type may have introduced conservatism in some structural systems; and
- Prequalification testing to expected cyclic inelastic demands is required for some structural systems (SMF/IMF and BRBF) and not for others; the EBF is the only system for which inelastic deformation demands must be explicitly calculated and compared with a specified deformation capacity, and the SCBF and SPSW are the only two systems for which no specific ductility capacity requirement is provided (either in design or by prequalification testing).

The work here has highlighted design issues that remain to be investigated, such as ductility capacity of deep, slender columns where plastic hinging is expected at the column base, axial load demands, and column base design to name a few. It has also touched on innovative systems being developed to broaden the range of solutions in the engineer's toolbox and, in some cases, enhance expected seismic performance to provide more resilient designs.

It is hoped that the review presented here has (1) provided readers with an appreciation of contemporary seismic design requirements for steel structures as they have been shaped by evolving earthquake engineering philosophy, impacts of past earthquakes, new research findings, and, to some degree, serendipity; and (2) inspired research and development to bridge the gaps in existing knowledge and iron out some of the wrinkles identified so as to better unify design provisions across seismic force-resisting systems.

In the meantime, steel structures are playing a dominant role in the reconstruction of Christchurch, New Zealand, where there is a marked interest in low-damage and resilient systems. This interest may drive future research to investigate rapidly reparable steel structural systems that minimize business interruption and expedite rapid return to service.

Appendix. Limiting Width-Thickness Ratios for SMF Rolled I-Shaped Beams and Columns

Design code	Limiting $b_f/2t_f$	Limiting h_c/t_w
1985 UBC	8.5, 7.0, 6.0 for $F_y = 36$, 50, 65 ksi, respectively (from plastic design)	$C_a \leq 0.30$: $\frac{412}{\sqrt{F_y}}(1 - 1.26C_a)$ $C_a > 0.30$: $\frac{257}{\sqrt{F_y}}$
1988 UBC; 1990, 1992, 1997[a] AISC Seismic Provisions (λ_p)	$52/\sqrt{F_y}$	$C_a \leq 0.125$: $\frac{520}{\sqrt{F_y}}(1 - 1.54C_a)$ $C_a > 0.125$: $\frac{191}{\sqrt{F_y}}(2.33 - C_a) \geq \frac{253}{\sqrt{F_y}}$
2002, 2005 AISC Seismic Provisions (λ_{ps})	$0.30\sqrt{E/F_y}$	Beam web: $2.45\sqrt{E/F_y}$ Column web: $C_a \leq 0.125$: $3.14\sqrt{E/F_y}(1 - 1.54C_a)$ $C_a > 0.125$[b]: $1.12\sqrt{E/F_y}(2.33 - C_a) \geq 1.49\sqrt{E/F_y}$
2010 Seismic Provisions (λ_{hd})	$0.30\sqrt{E/F_y}$	Beam and column webs: $C_a \leq 0.125$: $2.45\sqrt{E/F_y}(1 - 0.93C_a)$ $C_a > 0.125$: $0.77\sqrt{E/F_y}(2.93 - C_a) \geq 1.49\sqrt{E/F_y}$
2016 Seismic Provisions (λ_{hd})	$0.32\sqrt{E/(R_yF_y)}$	$C_a \leq 0.114$: $2.57\sqrt{E/(R_yF_y)}(1 - 1.04C_a)$ $C_a > 0.114$: $0.88\sqrt{E/(R_yF_y)}(2.68 - C_a) \geq 1.57\sqrt{E/(R_yF_y)}$

[a]The limiting h_c/t_w ratio is relaxed to λ_p in LRFD Specification Table B5.1 if the SCWB ratio is larger than 1.25; the SCWB ratio is increased to 2.0 in Supplement No. 2 (AISC 2000) and subsequent editions of the AISC Seismic Provisions.
[b]The lower limit ($1.49\sqrt{E/F_y}$) was dropped from the 2002 edition.

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References

AISC. (1969). "Specification for the design, fabrication and erection of structural steel for buildings." Chicago.

AISC. (1986). "Specification for structural steel buildings: Load and resistance factor design." Chicago.

AISC. (1990). "Seismic provisions for structural steel buildings: Load and resistance factor design." Chicago.

AISC. (1992). "Seismic provisions for structural steel buildings." Chicago.

AISC. (1997). "Seismic provisions for structural steel buildings." Chicago.

AISC. (2000). "Seismic provisions for structural steel buildings (1997) supplement No. 2." Chicago.

AISC. (2002). "Seismic provisions for structural steel buildings." *ANSI/AISC 341-02*, Chicago.
AISC. (2005). "Seismic provisions for structural steel buildings." *ANSI/AISC 341-05*, Chicago.
AISC. (2010). "Seismic provisions for structural steel buildings." *ANSI/AISC 341-10*, Chicago.
AISC. (2012). *Seismic design manual*, 2nd Ed., Chicago.
AISC. (2016a). "Prequalified connections for special and intermediate steel moment frames for seismic applications." *AISC 358*, Chicago.
AISC. (2016b). "Seismic provisions for structural steel buildings." *ANSI/AISC 341-16*, Chicago.
AISC. (2016c). "Specification for structural steel buildings." *AISC 360*, Chicago.
AISC. (2017). *Steel construction manual*, 15th Ed., Chicago.
Anderson, J. C., and Linderman, R. R. (1991). "Steel beam to box column connections." *Rep. No. CE 91-03*, Univ. of Southern California, Los Angeles.
ASCE. (2013). "Seismic rehabilitation of existing buildings." *ASCE/SEI 41-13*, Reston, VA.
ASCE. (2016). "Minimum design loads for buildings and other structures." *ASCE/SEI 7*, Reston, VA.
ASCE-WRC (Welding Research Council). (1971). "Plastic design in steel: A guide and commentary." *Manual of Practice No. 41*, New York.
Aslani, F., and Goel, S. C. (1991). "Stitch spacing and local buckling in seismic resistant double angle bracing members." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1991)117:8(2442), 2442–2463.
Astaneh-Asl, A., Bergsma, G., and Shen, J. H. (1992). "Behavior and design of base plates for gravity, wind, and seismic loads." *Proc., National Steel Construct. Conf.*, AISC, Chicago.
Astaneh-Asl, A., and Goel, S. C. (1984). "Cyclic in-plane buckling of double-angle bracing." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1984)110:9(2036), 2036–2055.
Astaneh-Asl, A., Goel, S. C., and Hanson, R. D. (1985). "Cyclic out-of-plane buckling of double-angle bracing." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1985)111%3A5(1135), 1135–1153.
Astaneh-Asl, A., Goel, S. C., and Hanson, R. D. (1986). "Earthquake-resistant design of double angle bracing." *Eng. J.*, 23(4), 133–147.
ATC (Applied Technology Council). (1978). "Tentative provisions for the development of seismic regulations for buildings." *Rep. No. ATC 3-06*, Redwood City, CA.
Atlayan, O., and Charney, F. A. (2014). "Hybrid buckling-restrained braced frames." *J. Constr. Steel Res.*, 96, 95–105.
AWS (American Welding Society). (2005). "Structural welding code: Seismic supplement." *AWS D1.8/D1.8M*, Miami.
AWS (American Welding Society). (2015). "Structural welding code: Steel." *AWS D1.1/D1.1M*, Miami.
Barnwell, N. (2015). "Experimental testing of shallow embedded connections between steel columns and concrete footings." Master's thesis, Brigham Young Univ., Provo, UT.
Barsom, J. M., and Rolfe, S. T. (1999). *Fracture and fatigue control in structures*, 3rd Ed., Butterworth-Heinemann, Waltham, MA.
Behbahanifard, M., Grondin, G., and Elwi, A. (2003). "Experimental and numerical investigation of steel plate shear walls." *Rep. No. 254*, Univ. of Alberta, Edmonton, AB.
Berman, J. W., and Bruneau, M. (2003). "Plastic analysis and design of steel plate shear walls." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2003)129%3A11(1448), 1448–1456.
Berman, J. W., and Bruneau, M. (2008a). "Capacity design of vertical boundary elements in steel plate shear walls." *Eng. J.*, 45(1), 57–71.
Berman, J. W., and Bruneau, M. (2008b). "Tubular links for eccentrically braced frames. I: Finite element parametric study." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2008)134%3A5(692), 692–701.
Berman, J. W., and Bruneau, M. (2008c). "Tubular links for eccentrically braced frames. II: Experimental verification." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2008)134%3A5(702), 702–712.
Bertero, V. V., Uang, C.-M., Llopiz, C. R., and Igarashi, K. (1989). "Earthquake simulator testing of concentric braced dual system." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1989)115%3A8(1877), 1877–1894.
Bradley, C. R., Fahnestock, L. A., Hines, E. M., and Sizemore, J. G. (2017). "Full-scale cyclic testing of low-ductility concentrically braced frames." *J. Struct. Eng.*, 10.1061/%28ASCE%29ST.1943-541X.0001760, 04017029.
Bruneau, M., and MacRae, G. (2017). "Reconstruction of Christchurch: A seismic shift to steel buildings." Univ. of Canterbury, Canterbury, New Zealand.
Bruneau, M., and Mahin, S. A. (1991). "Full scale tests of butt welded splices in heavy rolled steel sections subjected to primary tensile stresses." *Eng. J.*, 28(1), 1–17.
Bruneau, M., and Qu, B. (2011). "Plastic moment of intermediate horizontal boundary elements of steel plate shear walls." *Eng. J.*, 48(1), 49–64.
Bruneau, M., and Sarraf, M. H. K. (1996). "Cyclic testing of existing and retrofitted riveted stiffened-seat angle connections." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1996)122%3A7(762), 762–775.
Bruneau, M., Uang, C.-M., and Sabelli, R. (2011). *Ductile design of steel structures*, 2nd Ed., McGraw-Hill, New York.
Burda, J. J., and Itani, A. M. (1999). "Studies of seismic behavior of steel base plates." *Rep. No. CCEER 99-7*, Univ. of Nevada, Reno, NV.
Caccese, V., Elgaaly, M., and Chen, R. (1993). "Experimental study of thin steel-plate shear walls under cyclic load." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1993)119:2(573), 573–587.
Carter, C. J., Muir, L., and Dowswell, B. (2016). "Establishing and developing the weak-axis strength of plates subjected to axial loads." *Eng. J.*, 53(3), 147–158.
Celik, O. C., Berman, J. W., and Bruneau, M. (2005). "Cyclic testing of braces laterally restrained by steel studs." *J. Struct. Eng*, 10.1061/(ASCE)0733-9445(2005)131%3A7(1114), 1114–1124.
Chi, B., and Uang, C.-M. (2002). "Cyclic response and design recommendations of RBS moment connections with deep columns." *J. Struct. Eng*, 10.1061/(ASCE)0733-9445(2002)128:4(464), 464–473.
Chou, C. C., and Chen, Y. C. (2015). "Development of steel dual-core self-centering braces: Quasi-static cyclic tests and finite element analyses." *Earthquake Spectra*, 31(1), 247–272.
Chou, C. C., Liu, J. H., and Pham, D. H. (2012). "Steel buckling-restrained braced frames with single and dual corner gusset connections: Seismic tests and analyses." *Earthquake Eng. Struct. Dyn.*, 41(7), 1137–1156.
Christchurch City Council. (2016). "Resilient Greater Christchurch." ⟨http://greaterchristchurch.org.nz/⟩ (Feb. 22, 2017).
Christopoulos, C., Filiatrault, A., and Folz, B. (2002a). "Seismic response of self-centering hysteretic SDOF systems." *Earthquake Eng. Struct. Dyn.*, 31(5), 1131–1150.
Christopoulos, C., Filiatrault, A., Uang, C.-M., and Folz, B. (2002b). "Posttensioned energy dissipating connections for moment-resisting steel frames." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2002)128:9(1111), 1111–1120.
Christopoulos, C., Tremblay, R., Kim, H.-J., and Lacerte, M. (2008). "Self-centering energy dissipative system for the seismic resistance of structures: Development and validation." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2008)134%3A1(96), 96–107.
Clayton, P., Tsai, C., Berman, J., and Lowes, L. (2015). "Comparison of web plate numerical models for self-centering steel plate shear walls." *Earthquake Eng. Struct. Dyn.*, 44(12), 2093–2110.
Clifton, C., Bruneau, M., MacRae, G., Leon, R., and Fussell, A. (2011). "Steel building damage from the Christchurch earthquake series of 2010/2011." *Bull. N. Z. Soc. Earthquake Eng.*, 44(4), 279–318.
Clifton, G. C. (1996). "Development of perimeter moment-resisting steel frames incorporating semirigid elastic joints." *Proc., NZSEE Conf.*, New Zealand National Society for Earthquake Engineering, Wellington, New Zealand, 177–184.
Clifton, G. C. (2005). "Semirigid joints for moment resting steel framed seismic resisting systems." Ph.D. thesis, Univ. of Auckland, Auckland, New Zealand.
Cordova, P. P., and Hamburger, R. O. (2011). "Steel connections: Proprietary or public domain." *Mod. Steel Constr.*, 10, 24–31.
CSA (Canadian Standards Association). (1994). "Limit states design of steel structures." *CAN/CSA S16-94*, Toronto.
CSA (Canadian Standards Association). (2014). "Design of steel structures." *CSA-S16-14*, Toronto.
Cui, Y., and Nakashima, M. (2011). "Hysteretic behavior and strength capacity of shallowly embedded steel column bases with SFRCC slab." *Earthquake Eng. Struct. Dyn.*, 40(13), 1495–1513.

Dastfan, M., and Driver, R. G. (2008). “Flexural stiffness limits for frame members of steel plate shear wall systems.” *Proc., Annual Stability Conf.*, Structural Stability Research Council, Rolla, MO.

Dowdell, D. J., Hamersley, B., Bruneau, M., and Tudor, S. (2015). “Tubular eccentrically braced frames for the San Francisco-Oakland Bay Bridge temporary works.” *Proc., 11th Canadian Conf. on Earthquake Engineering*, Canadian Association for Earthquake Engineering, Vancouver, BC, Canada.

Dowden, D. M., et al. (2016). “Full-scale pseudo-dynamic testing of self-centering steel plate shear walls.” *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001367, 04015100.

Dowden, D. M., and Bruneau, M. (2014). “Resilient self-centering steel plate shear walls.” *Technical Rep. MCEER-14-0010*, Univ. at Buffalo, Buffalo, NY.

Dowden, D. M., and Bruneau, M. (2016). “Dynamic shake-table testing and analytical investigation of self-centering steel plate shear walls.” *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001547, 04016082.

Driver, R. G., Kulak, G. L., Elwi, A. E., and Kennedy, D. J. L. (1998). “Cyclic tests of four-story steel plate shear wall.” *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1998)124:2(112), 112–120.

Dusicka, P., and Itani, A. M. (2002). “Behavior of built-up shear links under large cyclic deformations.” *Proc., Annual Stability Conf.*, Structural Stability Research Council, Rolla, MO.

Dusicka, P., and Lewis, G. R. (2010). “Investigation of replaceable sacrificial steel links.” *Proc., 9th U.S. National and 10th Canadian Conf. on Earthquake Engineering*, Earthquake Engineering Research Institute, El Cerrito, CA.

Eatherton, M., et al. (2014a). “Design concepts for controlled rocking of self-centering steel-braced frames.” *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001047, 04014082.

Eatherton, M., Ma, X., Krawinkler, H., Deierlein, G., and Hajjar, J. (2014b). “Quasi-static cyclic behavior of controlled rocking steel frames.” *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001005, 04014083.

Egloff, O., and Tremblay, R. (2012). “Finite element analysis of ductile fuses for W-shape steel bracing members.” *Proc., 15th World Conf. on Earthquake Engineering*, International Association of Earthquake Engineering, Tokyo.

Egorova, N., Eatherton, M. R., and Maurya, A. (2014). “Experimental study of ring-shaped steel plate shear walls.” *J. Constr. Steel Res.*, 103, 179–189.

Elgaaly, M. (1998). “Thin steel plate shear walls behavior and analysis.” *Thin Walled Struct.*, 32(1–3), 151–180.

Elkady, A., and Lignos, D. G. (2015). “Analytical investigation of the cyclic behavior and plastic hinge formation in deep wide-flange steel beam-columns.” *Bull. Earthquake Eng.*, 13(4), 1097–1118.

Elkady, A., and Lignos, D. G. (2016). “Dynamic stability of deep and slender wide-flange steel columns: Full scale experiments.” *Proc., Annual Stability Conf.*, Structural Stability Research Council, Orlando, FL.

El-Tawil, S., Mikesell, T. D., Vidarsson, E., and Kunnath, S. K. (1999). “Inelastic behavior and design of steel panel zones.” *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1999)125%3A2(183), 183–193.

Engelhardt, M. D., and Husain, A. S. (1993). “Cyclic loading performance of welded flange-bolted web connections.” *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1993)119%3A12(3537), 3537–3550.

Engelhardt, M. D., and Popov, E. P. (1989). “On design of eccentrically braced frames.” *Earthquake Spectra*, 5(3), 495–511.

Engelhardt, M. D., and Popov, E. P. (1992). “Experimental performance of long links in eccentrically braced frames.” *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1992)118%3A11(3067), 3067–3088.

Engelhardt, M. D., Winneberger, T., Zekany, A. J., and Potyraj, T. (1998). “Experimental investigation of dogbone moment connections.” *Eng. J.*, 35(4), 128–139.

Fahmy, M., Stojadinovic, B., and Goel, S. C. (1999). “Analytical and experimental studies on the seismic response of steel column base.” *Proc., 8th Canadian Conf. on Earthquake Engineering*, Canadian Association for Earthquake Engineering, Vancouver, BC, Canada.

Fahnestock, L. A., Ricles, J. M., and Sause, R. (2003). “Ductility demands on buckling-restrained braced frames under earthquake loading.” *Earthquake Eng. Eng. Vib.*, 2(2), 255–268.

Fahnestock, L. A., Ricles, J. M., and Sause, R. (2007). “Experimental evaluation of a large-scale buckling-restrained braced frame.” *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2007)133%3A9(1205), 1205–1214.

Fell, B. V., Kanvinde, A. M., Deierlein, G. G., and Myers, A. T. (2009). “Experimental investigation of inelastic cyclic buckling and fracture of steel braces.” *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2009)135%3A1(19), 19–32.

FEMA. (2000a). “Prestandard and commentary for the seismic rehabilitation of buildings.” *FEMA-356*, Washington, DC.

FEMA. (2000b). “Recommended postearthquake evaluation and repair criteria for welded steel moment-frame buildings.” *FEMA-352*, Washington, DC.

FEMA. (2000c). “Recommended seismic design criteria for new steel moment-frame buildings.” *FEMA-350*, Washington, DC.

FEMA. (2000d). “Recommended seismic evaluation and upgrade criteria for existing welded steel moment-frame buildings.” *FEMA-351*, Washington, DC.

FEMA. (2000e). “Recommended specifications and quality assurance guidelines for steel moment-frame construction for seismic applications.” *FEMA-353*, Washington, DC.

FEMA. (2000f). “State of the art report on base metals and fracture.” *FEMA-355A*, Washington, DC.

FEMA. (2000g). “State of the art report on past performance of steel moment-frame buildings in earthquakes.” *FEMA-355E*, Washington, DC.

FEMA. (2000h). “State of the art report on performance prediction and evaluation of steel moment-frame buildings.” *FEMA-355F*, Washington, DC.

FEMA. (2000i). “State of the art report on systems performance of steel moment frames subject to earthquake ground shaking.” *FEMA-355C*, Washington, DC.

FEMA. (2000j). “State of the art report on welding and inspection.” *FEMA-355B*, Washington, DC.

FEMA. (2000k). “State-of-the-art report on connection performance.” *FEMA-355D*, Washington, DC.

Filiatrault, A., and Cherry, S. (1987). “Performance evaluation of friction damped braced frames under simulated earthquake loads.” *Earthquake Spectra*, 3(1), 57–78.

Filiatrault, A., and Cherry, S. (1988). “Comparative performance of friction damped systems and base isolation systems for earthquake retrofit and aseismic design.” *Earthquake Eng. Struct. Dyn.*, 16(3), 389–416.

Fogarty, J., and El-Tawil, S. (2016). “Collapse resistance of steel columns under combined axial and lateral loading.” *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001350, 04015091.

Foutch, D. A., Goel, S. C., and Roeder, C. W. (1987). “Seismic testing of full-scale steel building. Part I.” *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1987)113%3A11(2111), 2111–2129.

Freeman, S. A. (1990). “On the correlation of code forces to earthquake demands.” *Proc., 4th U.S. Japan Workshop on the Improvement of Building Structure Design and Construction Practices*, Applied Technology Council, Redwood City, CA, 245–268.

Fu, Y., Wang, F., and Bruneau, M. (2017). “Diagonal tension field inclination angle in steel plate shear walls.” *J. Struct. Eng.*, 10.1061/%28ASCE%29ST.1943-541X.0001779, 04017058.

Fujimoto, M., Aoyagi, T., Ukai, K., Wada, A., and Saito, K. (1972). “Structural characteristics of eccentric k-braced frames.” *Trans. AIJ*, 195, 39–49 (in Japanese).

Gagnon, D. P., and Kennedy, D. J. L. (1989). “Behaviour and ultimate tensile strength of partial joint penetration groove welds.” *Can. J. Civ. Eng.*, 16(3), 384–399.

Galasso, C., Stillmaker, K., Eltit, C., and Kanvinde, A. (2015). “Probabilistic demand and fragility assessment of welded column splices in steel moment frames.” *Earthquake Eng. Struct. Dyn.*, 44(11), 1823–1840.

Garlock, M. M., Ricles, J. M., and Sause, R. (2005). “Experimental studies of full-scale posttensioned steel connections.” *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2005)131:3(438), 438–448.

Garlock, M. M., Sause, R., and Ricles, J. M. (2007). "Behavior and design of posttensioned steel frame systems." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2007)133%3A3(389), 239–399.

Ghanaat, Y., and Clough, R. W. (1982). "Shaking table tests of a tubular steel frame model." *Rep. No. UCB/EERC-82/02*, Univ. of California, Berkeley, CA.

Gledhill, S. M., Sidwell, G. K., and Bell, D. K. (2008). "The damage avoidance design of tall steel frame buildings—Fairlie Terrace Student Accommodation Project, Victoria University of Wellington." *Proc., NZSEE Conf.*, New Zealand National Society for Earthquake Engineering, Wellington, New Zealand.

Goel, S. C. (1992). "Cyclic post buckling behavior of steel bracing members." *Stability and ductility of steel structures under cyclic loading*, CRC Press, Boca Raton, FL, 75–84.

Goel, S. C., and Itani, A. M. (1994). "Seismic-resistant special truss-moment frames." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1994)120%3A6(1781), 1781–1797.

Goel, S. C., and Lee, S. (1992). "A fracture criterion for concrete-filled tubular bracing members under cyclic loading." *Proc., 1992 ASCE Structures Congress*, ASCE, Reston, VA, 922–925.

Gomez, I. R., Kanvinde, A. M., and Deierlein, G. G. (2011). "Experimental investigation of shear transfer in exposed column base connections." *Eng. J.*, 48(4), 246–264.

Grauvilardell, J. E., Lee, D., Ajar, J. F., and Dexter, R. J. (2005). "Synthesis of design, testing and analysis research on steel column base plate connections in high seismic zone." *Structure Engineering Rep. No. ST-04-02*, Univ. of Minnesota, Minneapolis.

Gray, M. G., Christopoulos, C., and Packer, J. A. (2014). "Cast steel yielding brace system (YBS) for concentrically braced frames: Concept development and experimental validations." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0000910, 04013095.

Grilli, D. A., Jones, R., and Kanvinde, A. M. (2017). "Seismic performance of embedded column base connections subjected to axial and lateral loads." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001741, 04017010.

Grilli, D. A., and Kanvinde, A. M. (2017). "Embedded column base connections subjected to seismic loads: Strength model." *J. Constr. Steel Res.*, 129(2), 240–249.

Grondin, G. Y., and Behbahannidard, M. R. (2001). "Cyclic behaviour of three-storey unstiffened steel plate shear wall." *Proc., Canadian Society of Civil Engineers Conf.*, Victoria, BC, Canada.

Gross, J. L., Engelhardt, M. D., Uang, C.-M., Kasai, K., and Iwankiw, N. R. (1999). *Modification of existing steel welded moment frame connections for seismic resistance*, AISC, Chicago.

Gugerli, H., and Goel, S. (1980). "Large scale tests for the hysteresis behavior of inclined bracing members." *Proc., 7th World Conf. on Earthquake Engineering*, International Association of Earthquake Engineering, Tokyo, 87–94.

Gupta, A., and Krawinkler, H. (1999). "Prediction of seismic demands for SMRFs with ductile connections and elements." *Rep. No. SAC/BD-99/06*, SAC Joint Venture, Sacramento, CA.

Hamburger, R. O., and Malley, J. O. (2016). "Seismic design of steel special moment frames—A guide for practicing engineer." *NEHRP Seismic Design Technical Brief No. 2*, NIST, Gaithersburg, MD.

Hines, E., Appel, M., and Cheever, P. (2009). "Collapse performance of low-ductility chevron braced steel frames in moderate seismic regions." *Eng. J.*, 46(3), 149–180.

Hjelmstad, K. D., and Popov, E. P. (1983). "Cyclic behavior and design of link beams." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1983)109%3A10(2387), 2387–2403.

Hjelmstad, K. D., and Popov, E. P. (1984). "Characteristics of eccentrically braced frames." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1984)110%3A2(340), 340–353.

Hong, J. K., Uang, C.-M., Okazaki, T., and Engelhardt, M. D. (2015). "Link-to-column connection with supplemental web doublers in eccentrically braced frame." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001170, 04014200.

Housner, G. (1963). "The behavior of inverted pendulum structures during earthquakes." *Bull. Seismol. Soc. Am.*, 53(2), 403–417.

Hsiao, P. C., Lehman, D. E., Berman, J. W., Roeder, C. W., and Powell, J. (2012a). "Seismic vulnerability of older braced frames." *J. Perform. Constr. Facil.*, 10.1061/(ASCE)CF.1943-5509.0000394, 108–120.

Hsiao, P. C., Lehman, D. E., and Roeder, C. W. (2012b). "A model to simulate special concentrically braced frames beyond brace fracture." *Earthquake Eng. Struct. Dyn.*, 42(2), 183–200.

Huang, Y., and Mahin, S. A. (2010). "Simulating the inelastic seismic behavior of steel braced frames including the effects of low-cycle fatigue." *PEER Rep. No. 2010/104*, Univ. of California, Berkeley, CA.

ICBO (International Conference of Building Officials). (1985). "Uniform building code." Whittier, CA.

ICBO (International Conference of Building Officials). (1988). "Uniform building code." Whittier, CA.

ICBO (International Conference of Building Officials). (1997). "Uniform building code." Whittier, CA.

Imanpour, A., and Tremblay, R. (2017). "Analysis methods for the design of special concentrically braced frames with three or more tiers for in-plane seismic demand." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001696, 04016213.

Imanpour, A., Tremblay, R., Davaran, A., Stoakes, C., and Fahnestock, L. (2016a). "Seismic performance assessment of multitiered steel concentrically braced frames designed in accordance with the 2010 AISC seismic provisions." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001561, 04016135.

Imanpour, A., Tremblay, R., Fahnestock, L., and Stoakes, C. (2016b). "Analysis and design of two-tiered steel braced frames under in-plane seismic response." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001568, 04016115.

Itani, A. M., Elfass, S., and Douglas, B. M. (2003). "Behavior of built-up shear links under large cyclic displacement." *Eng. J.*, 40(4), 221–234.

Iwata, M., Kato, T., and Wada, A. (2000). "Buckling-restrained braces as hysteretic dampers." *Proc., 3rd Int. Conf. on Behavior of Steel Structures in Seismic Areas (STESSA)*, A.A. Balkema, Rotterdam, Netherlands, 33–38.

Kanvinde, A. M. (2017). "Predicting fracture in civil engineering steel structures: State of the art." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001704, 03116001.

Kanvinde, A. M., Grilli, D. A., and Zareian, F. (2012). "Rotational stiffness of exposed column base connections—Experiments and analytical models." *J. Struct. Eng.*, 10.1061/%28ASCE%29ST.1943-541X.0000495, 549–560.

Kasai, K., and Popov, E. P. (1986a). "Cyclic web buckling control for shear link beams." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1986)112:3(505), 505–523.

Kasai, K., and Popov, E. P. (1986b). "General behavior of WF steel shear link beams." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1986)112:2(362), 362–382.

Kato, B. (1990). "Deformation capacity of steel structures." *J. Constr. Steel Res.*, 17(1–2), 33–94.

Kelly, J. M., Aiken, I. D., and Pall, A. S. (1988). "Shake table tests on a nine-storey friction-damped frame." *Proc., 9th World Conf. on Earthquake Engineering*, International Association of Earthquake Engineering, Tokyo.

Kemp, A. R. (1996). "Inelastic local and lateral buckling in design codes." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1996)122%3A4(374), 374–382.

Kersting, R. A., Fahnestock, L. A., and López, W. A. (2015). "Seismic design of steel buckling-restrained braced frames—A guide for practicing engineers." *NEHRP Seismic Design Technical Brief No. 11*, NIST, Gaithersburg, MD.

Khoo, H. H., Tsai, K. C., Tsai, C. Y., Tsai, C. Y., and Wang, K. J. (2016). "Bidirectional substructure pseudo-dynamic tests and analysis of a full-scale two-story buckling-restrained braced frame." *Earthquake Eng. Struct. Dyn.*, 45(7), 1085–1107.

Kiggins, S., and Uang, C.-M. (2006). "Reducing residual drift of buckling-restrained braced frames as a duel system." *Eng. Struct.*, 28(11), 1525–1532.

Kim, D. W., Blaney, C., and Uang, C.-M. (2015). "Panel zone deformation capacity as affected by weld fracture at column kinking location." *Eng. J.*, 53(1), 27–46.

Kim, M. J., and Christopoulos, C. (2008). "Friction damped posttensioned self-centering steel moment-resisting frames." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2008)134%3A11(1768), 1768–1779.

Kim, T., Stojadinovic, B., and Whittaker, A. S. (2008). "Seismic performance of pre-Northridge welded steel moment connections to built-up box columns." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2008)134:2(289), 289–299.

Krawinkler, H. (1978). "Shear in beam-column joints in seismic design of steel frames." *Eng. J.*, 15(3), 82–91.

Krawinkler, H., Bertero, V. V., and Popov, E. P. (1971). "Inelastic behavior of steel beam-to-column subassemblages." *Rep. No. UCB/EERC-71/7*, Univ. of California, Berkeley, CA.

Krawinkler, H., Gupta, A., Medina, R., and Luco, N. (2000). "Loading histories for seismic performance testing of SMRF components and assemblies." *Rep. No. SAC/BD-00/10*, SAC Joint Venture, Sacramento, CA.

Lai, J. W., and Mahin, S. A. (2014). "Steel concentrically braced frames using tubular structural sections as bracing members: Design, full-scale testing and numerical simulation." *Int. J. Steel Struct.*, 14(1), 43–58.

Lai, J. W., and Mahin, S. A. (2015). "Strongback system: A way to reduce damage concentration in steel-braced frames." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001198, 04014223.

LATBSDC. (2017). "An alternative procedure for seismic analysis and design of tall buildings located in the Los Angeles region." Los Angeles Tall Buildings Structural Design Council, Los Angeles.

Lee, C. S., and Tsai, K. C. (2008). "Experimental response of four 2-story narrow steel plate shear walls." *Proc., Structures Congress*, ASCE, Reston, VA.

Lehman, D. E., Roeder, C. W., Herman, D., Johnson, S., and Kotulka, B. (2008). "Improved seismic performance of gusset plate connections." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2008)134%3A6(890), 890–901.

Leon, R. T., Forcier, G., Roeder, C. W., and Preece, F. R. (1994). "Cyclic performance of riveted connections." *Proc., 12th Structures Congress*, Vol. 2, ASCE, Reston, VA, 1490–1495.

Lin, C. H., Tsai, K. C., Qu, B., and Bruneau, M. (2010). "Sub-structural pseudo-dynamic performance of two full-scale two-story steel plate shear walls." *J. Constr. Steel Res.*, 66(12), 1467–1482.

Lin, P. C., et al. (2012). "Seismic design and hybrid tests of a full-scale three-story buckling-restrained braced frame using welded end connections and thin profile." *Earthquake Eng. Struct. Dyn.*, 41(5), 1001–1020.

Lin, P. C., Tsai, K. C., Chang, C. A., Hsiao, Y. Y., and Wu, A. C. (2016). "Seismic design and testing of buckling-restrained braces with a thin profile." *Earthquake Eng. Struct. Dyn.*, 45(3), 339–358.

Lin, P. C., Tsai, K. C., Wu, A. C., and Chuang, M. C. (2014). "Seismic design and test of gusset connections for buckling-restrained braced frames." *Earthquake Eng. Struct. Dyn.*, 43(4), 565–587.

López, W. A., and Sabelli, R. (2004). "Seismic design of buckling-restrained braced frames." *Steel TIPS*, Structural Steel Education Council, Chicago.

Lubell, A. S., Prion, H. G. L., Ventura, C. E., and Rezai, M. (2000). "Unstiffened steel plate shear wall performance under cyclic loading." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2000)126%3A4(453), 453–460.

MacRae, G. A., Carr, A. J., and Walpole, W. R. (1990). "The seismic response of steel frames." *Research Rep. 90-6*, Univ. of Canterbury, Christchurch, New Zealand.

MacRae, G. A., Clifton, G. C., Bruneau, M., Kanvinde, A., and Gardiner, S. (2015). "Lessons from steel structures in Christchurch earthquakes." *Proc., 8th Int. Conf. on Behavior of Steel Structures in Seismic Areas (STESSA)*, Shanghai, China.

MacRae, G. A., Kimura, Y., and Roeder, C. W. (2004). "Effect of column stiffness on braced frame seismic behavior." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2004)130%3A3(381), 381–391.

Mahin, S. A., Uriz, P., Aiken, I. D., Field, C., and Ko, E. (2004). "Seismic performance of buckling restrained braced frame systems." *Proc., 13th World Conf. on Earthquake Engineering*, International Association of Earthquake Engineering, Tokyo.

Malley, J. O., et al. (2006). "Seismic upgrade of a 15-story steel moment frame building—Satisfying performance criteria with application of experimental and analytical procedures." *Proc., 75th Annual Convention*, Structural Engineers Association of California, Sacramento, CA, 69–85.

Malley, J. O. (2010). "The AISC seismic provisions: Past, present and future." *Proc., NASCC: The Steel Conf.*, AISC, Chicago.

Malley, J. O., and Popov, E. P. (1984). "Shear links in eccentrically braced frames." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1984)110%3A9(2275), 2275–2295.

Manheim, D. N., and Popov, E. P. (1983). "Plastic shear hinges in steel frames." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1983)109:10(2404), 2404–2419.

Mansour, N., Christopoulos, C., and Tremblay, R. (2011). "Experimental validation of replaceable shear links for eccentrically braced steel frames." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0000350, 1141–1152.

Mar, D. (2006). "Design examples of rocking and restoring frames and walls." *Proc., 75th Annual Convention*, Structural Engineers Association of California, Sacramento, CA, 21–32.

McDaniel, C. C., Uang, C.-M., and Seible, F. (2003). "Cyclic testing of built-up steel shear links for the new Bay Bridge." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2003)129%3A6(801), 801–809.

Mitani, I., Makino, M., and Matsui, C. (1977). "Influence of local buckling on cyclic behavior of steel beam-columns." *Proc., 6th World Conf. on Earthquake Engineering*, International Association of Earthquake Engineering, Tokyo, 3175–3180.

Montgomery, C. J., et al. (2001). Unstiffened steel plate shear wall performance under cyclic loading." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2001)127:8(973), 973–975.

Myers, A. T., Kanvinde, A. M., Deierlein, G. G., and Fell, B. V. (2009). "Effect of weld details on the ductility of steel column baseplate connection." *J. Constr. Steel Res.*, 65(6), 1366–1373.

Naeim, F., ed. (2001). *The seismic design handbook*, 2nd Ed., Springer, New York.

Nakashima, M., Kanao, I., and Liu, D. (2002). "Lateral instability and lateral bracing of steel beams subjected to cyclic loading." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2002)128%3A10(1308), 1308–1316.

Nakashima, M., Roeder, C. W., and Maruoka, Y. (2000). "Steel moment frames for earthquakes in United States and Japan." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2000)126:8(861), 861–868.

Nakashima, M., Takanashi, K., and Kato, H. (1990). "Test of steel beam-column subject to sidesway." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1990)116%3A9(2516), 2516–2531.

Newell, J. D. (2008). "Cyclic behavior and design of steel columns subjected to large drift." Ph.D. dissertation, Univ. of California, San Diego.

Newell, J. D., and Uang, C.-M. (2008). "Cyclic behavior of steel wide-flange columns subjected to large drift." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2008)134:8(1334), 1334–1342.

NIST. (2010). "Nonlinear structural analysis for seismic design: A guide for practicing engineers." *NEHRP Seismic Design Technical Brief No. 4*, Gaithersburg, MD.

NIST. (2011). "Research plan for the study of seismic behavior and design of deep, slender, wide-flange structural steel-beam-column members." *NIST-GCR-11-917-13*, Gaithersburg, MD.

NIST. (2012). "Tentative framework for development of advanced seismic design criteria for new buildings." *NIST-GCR-12-917-20*, Gaithersburg, MD.

NIST. (2017). "Guidelines for nonlinear structural analysis for design of buildings. II: Steel moment frames." *NIST GCR 17-917-46*, Gaithersburg, MD.

NZS (New Zealand Standard). (2007). "Steel structures standard." *NZS 3404*, Wellington, New Zealand.

Okazaki, T., Arce, G., Ryu, H.-C., and Engelhardt, M. D. (2005). "Experimental study of local buckling, overstrength, and fracture of links in eccentrically braced frames." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2005)131%3A10(1526), 1526–1535.

Okazaki, T., and Engelhardt, M. D. (2007). "Cyclic loading behavior of EBF links constructed of ASTM A992 steel." *J. Constr. Steel Res.*, 63(6), 751–765.

Okazaki, T., Engelhardt, M. D., Drolias, A., Schell, E., Hong, J. K., and Uang, C.-M. (2009). "Experimental investigation of link-to-column connections in eccentrically braced frames." *J. Constr. Steel Res.*, 65(7), 1401–1412.

Okazaki, T., Engelhardt, M. D., Hong, J. K., Uang, C.-M., and Drolias, A. (2015). "Improved link-to-column connections for steel eccentrically braced frames." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001041, 04014201.

Ozkula, G., Harris, J., and Uang, C.-M. (2017). "Observations from cyclic tests on deep, wide-flange beam-columns." *Eng. J.*, 54(1), 45–59.

Palmer, K. D., Christopulos, A. S., Lehman, D. E., and Roeder, C. W. (2014). "Experimental evaluation of cyclically loaded, large-scale, planar and 3-D buckling-restrained braced frames." *J. Constr. Steel Res.*, 101, 415–425.

Park, R., and Paulay, T. (1975). *Reinforced concrete structures*, Wiley, New York.

PEER (Pacific Earthquake Engineering Research Center). (2017). "Guidelines for performance-based seismic design of tall buildings." *PEER Rep. No. 2017/06*, Univ. of California, Berkeley, CA.

Pekcan, G., Mander, J. B., and Chen, S. S. (2000). "Experiments on steel MRF building with supplemental tendon system." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2000)126%3A4(437), 437–444.

Plumier, A. (1997). "The dogbone: Back to the future." *Eng. J.*, 34(2), 61–67.

Pollino, M., and Bruneau, M. (2007). "Seismic retrofit of bridge steel truss piers using a controlled rocking approach." *J. Bridge. Eng.*, 10.1061/(ASCE)1084-0702(2007)12%3A5(600), 600–610.

Pollino, M., and Bruneau, M. (2010a). "Bi-directional behavior and design of controlled rocking 4-legged bridge steel truss piers." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0000262, 1512–1522.

Pollino, M., and Bruneau, M. (2010b). "Seismic testing of a bridge truss pier designed for controlled rocking." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0000261, 1523–1532.

Pollino, M., Slovenec, D., Qu, B., and Mosqueda, G. (2017). "Seismic rehabilitation of concentrically braced frames using stiff rocking cores." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001810, 04017080.

Popov, E. P. (1987). "Panel zone flexibility in seismic moment joints." *J. Constr. Steel Res.*, 8(1), 91–118.

Popov, E. P., Amin, N. R., Louie, J. J., and Stephen, R. M. (1985). "Cyclic behavior of large beam-column assemblies." *Earthquake Spectra*, 1(2), 203–238.

Popov, E. P., Bertero, V. V., and Chandramouli, S. (1975). "Hysteretic behavior of steel columns." *Rep. No. UCB/EERC-75-11*, Univ. of California, Berkeley, CA.

Popov, E. P., and Black, R. G. (1981). "Steel struts under severe cyclic loadings." *J. Struct. Div.*, 107(9), 1857–1881.

Popov, E. P., and Stephen, R. M. (1970). "Cyclic loading of full-size steel connections." *Rep. No. UCB/EERC-70-3*, Univ. of California, Berkeley, CA.

Popov, E. P., and Stephen, R. M. (1977). "Tensile capacity of partial penetration welds." *J. Struct. Div.*, 103(9), 1721–1729.

Popov, E. P., and Tsai, K. C. (1989). "Performance of large seismic steel moment connections under cyclic loads." *Eng. J.*, 26(2), 51–60.

Popov, E. P., Tsai, K. C., and Engelhardt, M. D. (1989). "On seismic steel joints and connections." *Eng. Struct.*, 11(4), 193–209.

Prinz, G. S., Richards, P. W., and Fremming, S. (2008). "Seismic response of buckling-restrained braced frames with beam splices." *Proc., 14th World Conf. on Earthquake Engineering*, International Association of Earthquake Engineering, Tokyo.

Purba, R., and Bruneau, M. (2009). "Finite element investigation and design recommendations for perforated steel plate shear walls." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0000061, 1367–1376.

Purba, R., and Bruneau, M. (2010). "Impact of horizontal boundary elements design on seismic behavior of steel plate shear walls." *Technical Rep. No. MCEER-10-0007*, Univ. at Buffalo, Buffalo, NY.

Purba, R., and Bruneau, M. (2012). "Case study on the impact of horizontal boundary elements design on seismic behavior of steel plate shear walls." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0000490, 647–657.

Purba, R., and Bruneau, M. (2014a). "Seismic performance of steel plate shear walls considering two different design philosophies of infill plates. I: Deterioration model development." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001098, 04014160.

Purba, R., and Bruneau, M. (2014b). "Seismic performance of steel plate shear walls considering two different design philosophies of infill plates. II: Assessment of collapse potential." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001097, 04014161.

Purba, R., and Bruneau, M. (2014c). "Seismic performance of steel plate shear walls considering various design approaches." *Technical Rep. No. MCEER-14-0005*, Univ. at Buffalo, Buffalo, NY.

Qu, B., and Bruneau, M. (2009). "Design of steel plate shear walls considering boundary frame moment resisting action." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0000069, 1511–1521.

Qu, B., and Bruneau, M. (2010a). "Behavior of vertical boundary elements in steel plate shear walls." *Eng. J.*, 47(2), 109–122.

Qu, B., and Bruneau, M., (2010b). "Capacity design of intermediate horizontal boundary elements of steel plate shear walls." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0000167, 665–675.

Qu, B., Bruneau, M., Lin, C. H., and Tsai, K. C., (2008). "Testing of full-scale two-story steel plate shear wall with reduced beam section connections and composite floors." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2008)134%3A3(364), 364–373.

Qu, Z., Wada, A., Motoyui, S., Sakata, H., and Kishiki, S. (2012). "Pin-supported walls for enhancing the seismic performance of building structures." *Earthquake Eng. Struct. Dyn.*, 41(14), 2075–2091.

Ramadan, T., and Ghobarah, A. (1995). "Behaviour of bolted link-column joints in eccentrically braced frame." *Can. J. Civ. Eng.*, 22(4), 745–754.

Reina, P., and Normile, D. (1997). "Fully braced for seismic survival." *Eng. News Rec.*, 239(3), 34–36.

Rezai, M. (1999). "Seismic behaviour of steel plate shear walls by shake table testing." Ph.D. dissertation, Univ. of British Columbia, Vancouver, BC, Canada.

Richards, P. W., and Uang, C.-M. (2006a). "Effect of flange width-thickness ratio on eccentrically braced frames link cyclic rotation capacity." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2005)131%3A10(1546), 1546–1552.

Richards, P. W., and Uang, C.-M. (2006b). "Testing protocol for short links in eccentrically braced frames." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2006)132%3A8(1183), 1183–1191.

Ricles, J. M., and Popov, E. P. (1989). "Composite action in eccentrically braced frames." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1989)115:8(2046), 2046–2066.

Ricles, J. M., Sause, R., Garlock, M., and Zhao, C. (2001). "Posttensioned seismic-resistant connections for steel frames." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2001)127:2(113), 113–121.

Ricles, J. M., Sause, R., Peng, S. W., and Lu, L. W. (2002). "Experimental evaluation of earthquake resistant posttensioned steel connections." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2002)128:7(850), 850–859.

Roberts, T. M., and Sabouri-Ghomi, S. (1991). "Hysteretic characteristics of unstiffened plate shear panel." *Thin Walled Struct.*, 12(2), 145–162.

Roberts, T. M., and Sabouri-Ghomi, S. (1992). "Hysteretic characteristics of unstiffened perforated steel plate shear walls." *Thin Walled Struct.*, 14(2), 139–151.

Rodas, P. T., Zareian, F., and Kanvinde, A. M. (2017). "Rotational stiffness of deeply embedded column-base connections." *J. Struct. Eng.*, 10.1061/%28ASCE%29ST.1943-541X.0001789, 04017064.

Roeder, C. W., Foutch, D. A., and Goel, S. C. (1987). "Seismic testing of full-scale steel building. Part: II." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1987)113%3A11(2130), 2130–2145.

Roeder, C. W., Knechtel, B., Thomas, E., Vaneaton, A., Leon, R. T., and Preece, F. R., (1996). "Seismic behavior of older steel structures." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1996)122%3A4(365), 365–373.

Roeder, C. W., Lumpki, E. J., and Lehman, D. E. (2011). "A balanced design procedure for special concentrically braced frame connections." *J. Constr. Steel Res.*, 67(11), 1760–1772.

Roeder, C. W., and Popov, E. P. (1978). "Eccentrically braced steel frames for earthquakes." *J. Struct. Div.*, 104(3), 391–411.

Roeder, C. W., Schneider, S. P., and Carpenter, J. E. (1993). "Seismic behavior of moment-resisting steel frames: Analytical study." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1993)119%3A6(1866), 1866–1884.

Rojas, P., Ricles, J. M., and Sause, R. (2005). "Seismic performance of post-tensioned steel moment resisting frames with friction devices." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2005)131:4(529), 529–540.

Sabelli, R., and Bruneau, M. (2007). *Steel plate shear walls*, AISC, Chicago.

Sabelli, R., Mahin, S., and Chang, C. (2003). "Seismic demands on steel braced frame buildings with buckling-restrained braces." *Eng. Struct.*, 25(5), 655–666.

Sabelli, R., Roeder, C. W., and Hajjar, J. F. (2013). "Seismic design of steel special concentrically braced frame systems—A guide for practicing engineers." *NEHRP Seismic Design Technical Brief No. 8*, NIST, Gaithersburg, MD.

Sahoo, D. R., and Chao, S.-H. (2015). "Stiffness-based design for mitigation of residual displacements of buckling-restrained braced frames." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001204, 04014229.

Sarkisian, M., Mathias, N., Garai, R., Uang, C.-M., and Kim, D. W. (2011). "Pin-fuse frame: Full scale tests." *Proc., 80th Annual Convention*, Structural Engineers Association of California, Sacramento, CA.

Sarraf, M., and Bruneau, M. (2004). "Performance tests of innovative ductile steel retrofitted deck-truss bridges." *Proc., 13th World Conf. on Earthquake Engineering*, International Association of Earthquake Engineering, Tokyo.

Schneider, S. P., Roeder, C. W., and Carpenter, J. E. (1993). "Seismic behavior of moment-resisting steel frames: Experimental study." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1993)119:6(1885), 1885–1902.

SEAOC (Structural Engineers Association of California). (1980). "Recommended lateral force requirements and commentary." Sacramento, CA.

SEAOC (Structural Engineers Association of California). (1987). "Recommended lateral force requirements and commentary." Sacramento, CA.

Shaback, B., and Brown, T. (2003). "Behavior of square hollow structural steel braces with end connections under reversed cyclic axial loading." *Can. J. Civ. Eng.*, 30(4), 745–753.

Shaw, S. M., Stillmaker, K., and Kanvinde, A. M. (2015). "Seismic response of partial-joint-penetration welded column splices in moment-resisting frames." *Eng. J.*, 52(2), 87–108.

Shen, J., Sabol, T. A., Akbas, B., and Sutchiewcharn, N. (2010). "Seismic demand on column splices in steel moment frames." *Eng. J.*, 47(4), 223–240.

Shishkin, J., Driver, R., and Grondin, G. (2009). "Analysis of steel plate shear walls using the modified strip model." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0000066, 1357–1366.

Song, Y. H., Ishii, T., Harad, Y., and Morita, K. (2011). "Study on fracture behavior of electro-slag welded joints in beam-to-built-up box column connection." *Proc., 6th Int. Symp. on Steel Structures*, Korean Society of Steel Construction, Seoul.

Steimer, S. F., Godden, W. G., and Kelly, J. M. (1981). "Experimental behavior of a spatial piping system with steel energy absorbers subjected to a simulated differential seismic input." *Rep. No. UCB/EERC-81/09*, Univ. of California, Berkeley, CA.

Stoakes, C. D., and Fahnestock, L. A. (2011). "Cyclic flexural testing of concentrically braced frame beam-column connections." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0000326, 739–747.

Stoakes, C. D., and Fahnestock, L. A. (2016). "Strong-axis stability of wide flange steel columns in the presence of weak-axis flexural yielding." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0001448, 04016004.

Takeuchi, T., Hajjar, J. F., Matsui, R., Nishimoto, K., and Aiken, I. D. (2010). "Local buckling restraint condition for core plates in buckling restrained braces." *J. Constr. Steel Res.*, 66(2), 139–149.

Takeuchi, T., Matsui, R., and Mihara, S. (2016). "Out-of-plane stability assessment of buckling-restrained braces including connections with chevron configuration." *Earthquake Eng. Struct. Dyn.*, 45(12), 1895–1917.

Takeuchi, T., Ozaki, H., Matsui, R., and Sutcu, F. (2014). "Out-of-plane stability of buckling-restrained braces including moment transfer capacity." *Earthquake Eng. Struct. Dyn.*, 43(6), 851–869.

Tanabashi, R., Naneta, K., and Ishida, T. (1974). "On the rigidity and ductility of steel bracing assemblage." *Proc., 5th World Conf. on Earthquake Engineering*, Vol. 1, International Association of Earthquake Engineering, Tokyo, 834–840.

Tang, X., and Goel, S. C. (1987). "Seismic analysis and design considerations of braced steel structures." *Rep. No. UMCE 87-4*, Univ. of Michigan, Ann Arbor, MI.

Tang, X., and Goel, S. C. (1989). "Brace fractures and analysis of phase I structure." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1989)115%3A8(1960), 1960–1976.

Thorburn, L. J., Kulak, G. L., and Montgomery, C. J. (1983). "Analysis of steel plate shear walls." *Structure Engineering Rep. No. 107*, Univ. of Alberta, Edmonton, AB, Canada.

Thornton, W. A. (1991). "On the analysis and design of bracing connections." *Proc., National Steel Construction Conf.*, AISC, Chicago, 26.1–26.33.

Timler, P. A., and Kulak, G. L. (1983). "Experimental study of steel plate shear walls." *Structure Engineering Rep. No. 114*, Univ. of Alberta, Edmonton, AB, Canada.

Timler, P. A., Ventura, C. E., Prion, H., and Anjam, R. (1998). "Experimental and analytical studies of steel plate shear walls as applied to the design of tall buildings." *Struct. Des. Tall Build.*, 7(3), 233–249.

Tremblay, R. (2002). "Inelastic seismic response of bracing members." *J. Constr. Steel Res.*, 58(5–8), 665–701.

Tremblay, R. (2003). "Achieving a stable inelastic seismic response for multi-story concentrically braced steel frames." *Eng. J.*, 40(2), 111–129.

Tremblay, R., Archambault, M. H., and Filiatrault, A. (2003). "Seismic performance of concentrically braced steel frames made with rectangular hollow bracing members." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2003)129%3A12(1626), 1626–1636.

Tremblay, R., and Filiatrault, A. (1996). "Seismic impact loading in inelastic tension-only concentrically braced steel frames: Myth or reality." *Earthquake Eng. Struct. Dyn.*, 25(12), 1373–1389.

Tremblay, R., and Poncet, L. (2007). "Improving the seismic stability of concentrically braced steel frames." *Eng. J.*, 44(2), 103–116.

Tremblay, R., and Stiemer, S. F. (1994). "Back-up stiffness for improving the stability of multi-storey braced frames under seismic loading." *Proc., Annual Task Group Technical Session*, Structural Stability Research Council, Rolla, MO, 311–325.

Tromposch, E. W., and Kulak, G. L. (1987). "Cyclic and static behaviour of thin panel steel plate shear walls." *Structure Engineering Rep. No. 145*, Univ. of Alberta, Edmonton, AB, Canada.

Tsai, C.-Y., et al. (2013). "Seismic design and hybrid tests of a full-scale three-story concentrically braced frame using in-plane buckling braces." *Earthquake Spectra*, 29(3), 1043–1067.

Tsai, K. C., et al. (2008). "Pseudo-dynamic tests of a full-scale CFT/BRB frame. Part I: Specimen design, experiment and analysis." *Earthquake Eng. Struct. Dyn.*, 37(7), 1081–1098.

Tsai, K. C., Chen, H. W., Hong, C. P., and Su, Y. F. (1993). "Design of steel triangular plate energy absorbers for seismic-resistant construction." *Earthquake Spectra*, 9(3), 505–528.

Tsai, K. C., and Hsiao, P. C. (2008). "Pseudo-dynamic test of a full-scale CFT/BRB frame. Part II: Seismic performance of buckling restrained braces and connections." *Earthquake Eng. Struct. Dyn.*, 37(7), 1099–1115.

Tsai, K. C., Hwang, Y. C., Weng, C. S., Shirai, T., and Nakamura, H. (2002). "Experimental tests of large scale buckling restrained braces and frames." *Proc., Passive Control Symp.*, Tokyo Institute of Technology, Tokyo, 1–11.

Tsai, K. C., Lin, K. C., Juang, S. J., Li, C. H., and Lin, C. H. (2015). "Research and practice on seismic design of welded steel beam-to-box column moment connections in Taiwan." *Symp. on Future Development of Seismic Design*, Hong Kong Institute of Steel Construction, Hong Kong.

Tyler, R. G. (1978). "Tapered steel energy dissipators for earthquake-resistant structures." *Bull. N. Z. Nat. Soc. Earthquake Eng.*, 11(4), 282–294.

Uang, C.-M. (1991). "Establishing R (or R_w) and C_d factors for building seismic codes." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1991)117:1(19), 19–28.

Uang, C.-M. (1993). "An evaluation of two-level seismic design procedure." *Earthquake Spectra*, 9(1), 121–135.

Uang, C.-M., and Bertero, V. V. (1986). "Earthquake simulation tests and associated studies of a 0.3-scale model of a six-story concentrically braced steel structure." *Rep. No. UCB/EERC-86/10*, Univ. of California, Berkeley, CA.

Uang, C.-M., Bruneau, M., and Tsai, K.-C. (2014a). "Seismic design of steel bridges." Chapter 9, *Bridge engineering handbook*, W.-F. Chen and L. Duan, eds., CRC Press, Boca Raton, FL, 301–336.

Uang, C.-M., and Fan, C. C. (2001). "Cyclic stability criteria for steel moment connections with reduced beam section." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2001)127%3A9(1021), 1021–1027.

Uang, C.-M., Lee, P., Ozkula, G., Garai, R., and Sarkisian, M. (2014b). "Issues on using welded built-up box columns in steel special moment frames." *Proc., 10th U.S. National Conf. on Earthquake Engineering*, EERI, Oakland, CA.

Uang, C.-M., Nakashima, M., and Tsai, K. C. (2004). "Research and application of buckling-restrained braced frames." *Int. J. Steel Struct.*, 4(4), 301–313.

Uriz, P., and Mahin, S. A. (2008). "Toward earthquake-resistant design of concentrically braced steel-frame structures." *PEER Rep. No. 2008/08*, Univ. of California, Berkeley, CA.

Vian, D., Bruneau, M., and Purba, R. (2009a). "Special perforated steel plate shear walls with reduced beam section anchor beams. II: Analysis and design recommendations." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2009)135%3A3(221), 221–228.

Vian, D., Bruneau, M., Tsai, K. C., and Lin, Y. C. (2009b). "Special perforated steel plate shear walls with reduced beam section anchor beams. I: Experimental investigation." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2009)135%3A3(211), 211–220.

Vincent, R. B. (2008). "Minimizing the strength of bracing connection." *Proc., 6th Int. Workshop on Connections in Steel Structures*, AISC, Chicago, 117–141.

Watanabe, A., Hitomi, Y., Saeki, E., Wada, A., and Fujimoto, M. (1988). "Properties of brace encased in buckling-restraining concrete and steel tube." *Proc., 9th World Conf. on Earthquake Engineering*, Vol. IV, International Association of Earthquake Engineering, Tokyo, 719–724.

Webster, D. J., Berman, J. W., and Lowes, L. N. (2014). "Experimental investigation of SPSW web plate stress field development and vertical boundary element demand." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0000989, 04014011.

Wei, X., and Bruneau, M. (2017a). "Buckling restrained braces applications for superstructure and substructure protection in bridges." *Rep. No. MCEER-16-0009*, Univ. at Buffalo, Buffalo, NY.

Wei, X., and Bruneau, M. (2017b). "Experimental performance of buckling restrained braces subjected to bidirectional displacement histories." *Proc., 16th World Conf. on Earthquake Engineering*, International Association of Earthquake Engineering, Tokyo.

Westeneng, B., Lee, C.-L., MacRae, G., and Jones, A. (2017). "Out-of-plane buckling behaviour of BRB gusset plate connections." *Proc., 16th World Conf. on Earthquake Engineering*, International Association of Earthquake Engineering, Tokyo.

Whittaker, A. S., Uang, C.-M., and Bertero, V. V. (1989). "Seismic testing of eccentrically braced dual steel frames." *Earthquake Spectra*, 5(2), 429–449.

Wu, A. C., Lin, C. H., and Tsai, K. C. (2014). "High-mode buckling responses of buckling-restrained brace core plates." *Earthquake Eng. Struct. Dyn.*, 43(3), 375–393.

Xie, Q. (2005). "State of the art buckling-restrained braces in Asia." *J. Constr. Steel Res.*, 61(6), 727–748.

Yang, F., and Mahin, S. A. (2005). "Limiting net section fracture in slotted tube braces." *Steel tips, structure*, Structural Steel Educational Council, Moraga, CA.

Yang, M.-S. (1985). "Shaking table studies of an eccentrically braced steel structure." *Proc., 8th World Conf. Earthquake Engineering*, Vol. 4, IAEE, San Francisco, 257–264.

Yu, Q. S., and Uang, C.-M. (2001). "Effects of lateral bracing and system restraint on the behavior of reduced beam section moment frame connections." *Steel Compos. Struct.*, 1(1), 145–158.

Zahrai, S. M., and Bruneau, M. (1999). "Cyclic testing of ductile end diaphragms for slab-on-girder steel bridges." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1999)125%3A9(987), 987–996.

Zargar, S., Medina, R. A., and Miranda, E. (2014). "Cyclic behavior of deep steel columns subjected to large deformation demands and high axial loads." *Proc., 10th U.S. National Conf. on Earthquake Engineering*, Oakland, CA.

Zayas, A. Z., Shing, P. B., Mahin, S. A., and Popov, E. P. (1980). "Cyclic inelastic buckling of tubular steel braces." *Rep. No. UCB/EERC-80/16*, Univ. of California, Berkeley, CA.

Zhao, J. X., Lin, F. X., and Wang, Z. (2016). "Effect of non-moment braced frame seismic deformations on buckling-restrained brace end connection behavior: Theoretical analysis and subassemblage tests." *Earthquake Eng. Struct. Dyn.*, 45(3), 359–381.

Zhao, J. X., Wu, B., and Ou, J. (2012). "Flexural demand on pin-connected buckling-restrained braces and design recommendations." *J. Struct. Eng.*, 10.1061/(ASCE)ST.1943-541X.0000549, 1398–1415.

Zhao, Q., and Astaneh-Asl, A. (2004). "Cyclic behavior of traditional and innovative composite shear walls." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2004)130%3A2(271), 271–284.

Ziemian, R. D., ed. (2010). *Guide to stability design criteria for metal structures*, 6th Ed., Wiley, Hoboken, NJ.