High-Strength Brace Connectors for use in SCBF and OCBF

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Abstract

Innovative cast steel high-strength connectors for round hollow structural section brace members have been developed at the University of Toronto for use in Special and Ordinary Concentrically Braced Frames (SCBF and OCBF). The standardized connectors, now available through Cast Connex Corporation under their High-Strength Connector line of products, have been applied in the construction of various seismic-resistant steel braced frame buildings in the United States and Canada. At one end, the connectors are designed with a circular shape and preparation to allow for complete joint penetration shop welding to a range of round tubular braces of a given outer diameter for the full development of their expected strength. At the other end, the connectors are shaped such that a bolted double shear connection or longitudinal fillet welds can be used for connecting the shop-welded brace-connector assembly to conventional gusset plates in the field. This paper describes the mechanical properties of the cast steel material comprising the connectors, provides an outline of the design rationale for the connectors meeting the AISC Seismic Provisions’ requirements for SCBF and OCBF brace connections, and provides a summary of the full-scale structural testing that has been conducted to validate the connector design philosophy.

Introduction

Special and Ordinary Concentrically Braced Frames (SCBF and OCBF) are an efficient and popular choice for the lateral force resisting systems of mid- to low-rise seismic-resistant steel building frames. When lateral forces are applied to these structures, the diagonal bracing members of the braced frame are subjected to predominately axial loading. As Hollow Structural Section (HSS) members are the most economical structural shapes for carrying compressive loading, and given the wide range of section sizes that are readily available in the United States, it is not surprising that HSS are the typical brace member of choice in these buildings.

Statically loaded HSS braces are typically connected to the beam, column, or beam-column intersection via slotted HSS-to-gusset connections, and the braces are typically welded in the field to the single gusset plate at each end of the brace. Design of these connections for static axial loading accounting for shear lag effects has been studied by a number of researchers (British Steel, 1992; Korol et al., 1994; Zhao and Hancock, 1995; Cheng et al., 1996; Zhao et al., 1999; Wilkinson et al., 2002; Ling, 2005; Willibald et al., 2006; Martinez-Saucedo et al., 2006; Martinez-Saucedo et al., 2008; Martinez-Saucedo and Packer, 2009); these statically loaded connections can be readily detailed according to prevailing design standards. However, during a design level seismic event, the brace members in SCBF and OCBF are intended to cyclically yield in tension and buckle in compression, and as such the connections at each end of the brace must be appropriately capacity designed and detailed to ensure this behavior. If brace end connections are inappropriately designed, detailed, or fabricated, the connections can be prone to premature failure, as witnessed during post-earthquake reconnaissance and in laboratory testing (AIJ, 1995; Bonneville and Bartoletti, 1996; Tremblay et al., 1996; Yang and Mahin, 2005; Fell et al. 2006).

Failures in these types of connections in earthquakes typically occur due to a concentration of inelastic strain at the reduced section of the HSS-to-gusset connection. As a result, AISC
Seismic Provisions (AISC, 2005a) now require the reinforcement of the net-section of the brace at the connection whenever slotted HSS-to-gusset connections are used. As reinforcement is more readily fabricated for flat surfaces, the industry has tended toward the specification of square or rectangular HSS rather than round HSS for seismic-resistant braces. Preference has also been given to rectangular HSS braces as the specified minimum yield strength of round HSS is somewhat lower than for rectangular sections in ASTM A500 grade tubulars (ASTM, 2010a). This inclination to rectangular HSS over round is unfortunate, as recent studies have again shown that round HSS braces provide an improved cyclic inelastic braced frame response over rectangular HSS (Fell et al., 2009). Besides the reasons associated with a curved cross-section’s enhanced ability to resist local buckling, round HSS braces outperform their rectangular counterpart as a result of the additional cold working required to form rectangular HSS. Specifically, the manufacturing process for rectangular HSS reduces the available ductility in the corner radii of the section, which in turn increases the likelihood of premature brace fracture during cyclic inelastic buckling. An additional reason to select round HSS over rectangular HSS for seismic-resistant braces is that North American rectangular HSS have been shown to have inherently poor notch toughness (Kosteski et al., 2005), which limits their ability to resist crack propagation.

To address some of the shortcomings of existing standards for HSS, the AISC HSS Committee has been working on developing a new, higher performance manufacturing specification for HSS. A draft has been submitted to ASTM as an alternate production standard, in addition to ASTM A500. This manufacturing standard will hopefully feature tighter control of geometric tolerances and improved mechanical properties, including a notch toughness requirement and a restricted yield stress range, thus making it ideal for seismic design. Nevertheless, the difficulties surrounding the adequate detailing of seismic-resistant HSS bracing connections will remain.

To offer a simple solution for connecting round HSS in SCBF and OCBF, researchers at the University of Toronto developed standardized, capacity-designed, cast steel connectors which accommodate bolted double-shear connection between round HSS braces and a typical corner gusset plate and which also eliminate shear lag in the connection (de Oliveira et al., 2008b). Thus, use of the connectors vastly simplifies connection design by eliminating the need for connection reinforcement and, as the connectors allow bolted connection to the gusset plates, the need for field welding the brace to the gusset is also eliminated when the connectors are employed. The standardized connectors, now available through Cast Connex Corporation under their High-
casings are far closer to being isotropic than those in rolled steel products, and, owing to the improved geometric freedom offered by casting manufacturing, steel material need only be placed where it is necessary as determined from engineering analysis. It was for these reasons that the University of Toronto researchers opted to produce the brace connectors they conceived using steel casting technologies.

There are a number of ASTM cast steel grades that are suitable for structural use – AISC 360 (2010a) currently lists ASTM A216 (2008) as a cast steel grade useful for steel structures. Another ASTM cast steel grade, ASTM A958 Grade SC8620 Class 80/50 (ASTM, 2010c), was evaluated and approved by the AISC Connection Prequalification Review Panel in the course of approving the Kaiser Bolted Bracket for use in seismic-resistant moment resisting steel frames (Adan and Gibb, 2009; AISC, 2005b). This same grade, including the additional Charpy V-Notch (CVN) requirements outlined for the material in Appendix A of Supplement 1 to AISC 358 (2009), is also used in the production of the standardized brace connectors. This casting material has a chemical composition similar to that of standard wrought steel grades and provides a minimum yield strength of 50 ksi, a minimum ultimate tensile strength of 80 ksi, a minimum elongation in 2-inches of 22% with a reduction of area of 35%, and a CVN toughness of 20 ft-lb at 70ºF. To ensure the connectors are sound, production components are subjected to the following acceptance criteria, which mirror or exceed those set for the Kaiser Bolted Bracket: visual examination per ASTM A802 (2010d) with a surface acceptance Level I, ultrasonic testing per ASTM A609 (2007a) with an acceptance Level 3, and magnetic particle testing with an acceptance Level III in accordance with ASTM A903 (2007b).

**Connector Design Rationale per AISC Seismic Provisions**

As presented above, when SCBF or OCBF are inelastically loaded, their brace members undergo tensile yielding and compressive inelastic buckling. The AISC Seismic Provisions contain prescriptive requirements for bracing connections which are intended to accommodate the forces and deformations associated with brace yielding and buckling. As the requirements for SCBF connections are more stringent than those for OCBF connections, the standardized cast steel connectors were designed to exceed the SCBF brace connection requirements, as described in the following sections.

**Required Tensile Strength**

The required tensile strength of an SCBF brace connection must be greater than the lesser of either the expected yield strength of the bracing member, determined as $R_{yFyAg}$ (LRFD) or $R_{yFyAg}/1.5$ (ASD), or the maximum load effect that can be transferred to the brace by the system as indicated by analysis. The standardized cast steel connectors were thus designed to have a tensile strength at all cross-sections and joints exceeding the expected yield strength of all brace members that the connector is intended to accommodate. Specifically, the connector includes a weld prep bevel on one end which is designed to provide a 60-degree included angle between the end of the wall of the round HSS member and the connector’s bevel, around the entire circumference of the tube. The geometry is such that a complete joint penetration (CJP) groove weld can be produced between the tubular member and the connector, regardless of the HSS member’s wall thickness (Figure 2). With an appropriately selected electrode, and with properly executed welding to a Welding Procedure Specification approved per AWS requirements (AWS, 2010), a CJP provides a resistance equal to the expected tensile strength of the brace.

![Figure 2: Brace connection configuration and illustration showing how a single connector size can accommodate a range of wall thickness sizes for HSS of a given outer diameter](image-url)
Beyond the beveled end, the connector is solid steel and transitions gradually and smoothly from a round shape into two parallel plates. The transitional geometry is gradual enough to eliminate shear lag in the connection to the tubular member – refer to Section K1.6 of AISC 360 and to page 83 of AISC Design Guide 24 (AISC, 2010b) for the cap-plate-to-round HSS case – and to the plates. A section through the plates is the clearly governing gross section of the connector (Figure 3); the plates are sized such that their tensile yield strength exceeds the expected yield strength of all of the brace members that each connector is intended to accommodate. Beyond the governing gross section of the connector, the bolted double shear connection to the gusset plate can be readily detailed by the Engineer of Record or connection designer to have the requisite tensile capacity at all net sections (in the connector and gusset plate) and at the gross section of the gusset. Because the connector’s two plates have $F_y$ and $F_u$ greater than or equal to that of the gusset plate and have a combined thickness well over that of the gusset (note that the gusset plate’s maximum thickness is limited by the connector’s standardized geometry, specifically the size of the gap provided between the connector’s two plates), the resistance of the bolted joint at the gusset always governs the resistance of the overall bolted double shear connection.

**Figure 3:** Top and side views of a typical cast steel connector showing the governing gross section of the connector for use in determining the tensile resistance of the connector

### Required Flexural Strength

Braces in SCBFs are expected to undergo cyclic inelastic buckling under severe ground motions, forming plastic hinges at their center and at each end. Therefore, and as described in the commentary of the AISC Seismic Provisions, bracing connections must either have sufficient strength to confine inelastic rotation to the bracing member or sufficient ductility to accommodate brace-end rotations. In the first case, where the rotation is to be confined to the bracing member, the required flexural strength of the brace connection must be greater than $1.1R_yM_p$ (LRFD) or $(1.1/1.5)R_yM_p$ (ASD) of the brace about its critical buckling axis. An alternative to this requirement is to detail the brace connection such that it has sufficient flexural stiffness and strength to force the formation of the plastic hinge within the single gusset plate that is employed between the brace member end and the beam-column intersection. As described in the commentary of the AISC Seismic Provisions, this is typically achieved in conventional SCBF connections by separating the brace end from the restraint provided by the beam, column, or other brace joints by a free length of gusset plate that is at least twice the thickness of the gusset. The same brace buckling response can be achieved when employing the cast steel connectors by providing the free length of gusset beyond the ends of the connectors. The connectors are designed to have a flexural stiffness at all cross-sections and joints exceeding the flexural stiffness of the gusset plate at the plastic hinge, and a flexural strength at all cross-sections and joints exceeding $1.1R_yM_p$ (LRFD) or $(1.1/1.5)R_yM_p$ (ASD) of the gusset plate at the plastic hinge. The latter flexural strength requirement is conservative, as the bending moment induced in the connector during inelastic buckling of the brace will be lower than the plastic moment capacity of the gusset, as illustrated in Figure 4.

**Figure 4:** Deformed shape and the associated bending moments induced through the brace and brace connection during out-of-plane inelastic buckling of the brace – plotted on tension side (adapted from de Oliveira et al., 2008b)

As noted above, the gusset plate’s maximum thickness is limited by the connector’s standardized geometry, specifically the size of the gap provided between the connector’s two plates. It can be readily shown from engineering first principles that the flexural stiffness and strength of the connector at its governing cross section would greatly exceed those of the single gusset plate at the plastic hinge for all reasonable gusset plate widths. Note that connection designers would always try to minimize the width of the gusset plate at the plastic hinge to reduce the overall size of the connection, particularly since the Whitmore width would limit the effectiveness of any plate material beyond a 30-degree projection from the first bolt line of the bolted joint at the gusset.
Required Compressive Strength

Pursuant to the AISC Seismic Provisions, the compressive strength of SCBF brace connections must be at least $1.1R_yP_n$ (LRFD) or $(1.1/1.5)R_yP_n$ (ASD) where $P_n$ is the nominal compressive strength of the brace. This design force is based on the limit state of overall buckling of the brace, amplified by the brace material’s overstrength. In addition to the typical gusset plate design procedures, the Engineer of Record or connection designer making use of the cast steel brace connectors need only confirm that the sum of the compressive strengths of the two plates of the connector, acting individually in compression, exceed the required compressive strength indicated above. The free length of the plates would be equal to the gap between the first bolt line of the bolted joint at the gusset plate and the location where the two plates merge together at the heavier portion of the connector, as illustrated in Figure 5. For most practical (and correctly capacity-designed) bolted joint details, the compressive resistance of the two plates is equal to or exceeds even the expected yield strength of the brace, $R_yA_g$ (LRFD) or $R_yA_g/1.5$ (ASD), so this design provision will rarely, if ever, govern the connection design for all practical brace lengths. Additionally, the connection designer can opt to increase the longitudinal spacing between the bolts to reduce the unbraced length of the connector’s plates.

Validation of Design Philosophy through Full-Scale Testing

To validate the overall connector design philosophy presented above, a variety of full-scale destructive tests have been conducted on various sizes of round HSS braces fitted with cast steel connectors designed as described above. The following subsections summarize each of the types of test that have been conducted and provide representative test results. The original publications can be referenced for the full description of each of the tests and for the full suite of test results.

Static Monotonic Tensile Testing

As described in de Oliveira et al. (2008b), a static monotonic tensile test was conducted at the University of Toronto’s Structural Testing Facilities on a full-scale brace-connector assembly consisting of an HSS 6.625 x 0.500 member produced to ASTM A500 Grade C with cast steel connectors welded on either end. The cast connectors were in turn bolted to gusset plates that were designed to fit the wedge anchorages of the testing machine. During this test, displacement was monotonically increased beyond the peak load of 670 kip up to rupture of the specimen, which occurred in the tube a short distance above the midpoint of the round HSS member and only after significant necking of the tube’s cross section was observed. Figure 6 shows the fractured test specimen after completion of the tensile test.

This test validated the effectiveness of the CJP weld under monotonic tensile loading and illustrated the significant stiffness that the connectors possess, which forced tensile yielding and necking toward the center of the HSS segment,
away from the welded joint. Also of note is that the connectors employed in this test had already been used in one of the short brace quasi-static cyclic tests described in the following section. After that testing, the spent tube segments were removed from the connectors, the connectors’ noses were machined back to their original shape, and the connectors were welded to the HSS 6.625 x 0.500 tube segment for the tensile test pictured above.

**Quasi-Static Cyclic Testing of Short Brace-Connector Assemblies**

As described in de Oliveira et al. (2008b), full-scale, short brace-connector assemblies were tested at the University of Toronto’s Structural Testing Facilities under a specially prepared quasi-static cyclic inelastic loading protocol that was designed to stress the brace end connections to a higher degree than would be expected in a typical braced frame. Specifically, the loading protocol was tension-biased — as large compressive displacements in short braces result in premature local buckling and failure in the brace assemblies — but as the braces were stocky (L/r in the range of 45), the compressive loads attained in each compressive cycle were significantly higher than would be expected in a brace of a more typical slenderness. In this way, the brace-connector assemblies (and thus their welded CJP joints) were subjected to repeated loading, cycling between tensile brace yielding and elevated compressive loading. In all tests, failure of the brace occurred at the midsection of the brace, away from the connections. Figure 7 shows one of the short brace assemblies during a compression loading cycle and Figure 8 shows the hysteretic response of the short brace-connector assembly tests. In the plot, the force transmitted axially through the brace is plotted against the imparted axial strain in the HSS segment.

The short brace-connector assembly tests provided further verification of the strength and robustness of the welded CJP joints between the connector and the round HSS elements.

**Quasi-Static Cyclic Testing of Mid-Slenderness Brace-Connector Assemblies**

As described in de Oliveira et al. (2008a), full-scale, mid-slenderness brace-connector assemblies (L/r’s ranging between 85 and 141) were tested under quasi-static cyclic inelastic loading at the Hydro-Québec Structural Engineering Laboratory at École Polytechnique de Montréal as part of a collaborative research effort between University of Toronto, École Polytechnique, and Cast Connex Corporation. The four brace-connector specimens tested were fabricated using the following round HSS sections produced to ASTM A500 Grade C: HSS 4.000 x 0.313, HSS 5.563 x 0.375, HSS 6.625 x 0.500, and HSS 8.625 x 0.625. These are the heaviest walled sections available in ASTM A500 grade material in North America for these outer diameters. The full-scale tests were conducted in a 2,700 kip capacity MTS load frame fitted with specially designed grips which provided realistic representation of the out-of-plane restraint of the gusset plate at the beam-column intersection, as illustrated in Figure 9.
Symmetric quasi-static cyclic loading was applied to the brace-connector assembly at increasing displacement amplitudes until failure occurred. All four of the brace-connector assemblies behaved as would be expected of ductile buckling braces during the cyclic quasi-static testing. Large compressive excursions resulted in out-of-plane inelastic buckling of the bracing element (Figure 10b), with significant tensile yielding occurring over the majority of the HSS member’s length during the higher amplitude tensile excursions. As expected, fan-shaped plastic hinges formed beyond the ends of the connectors within the free length of the gusset plates as a result of overall member buckling (Figure 10c). At higher amplitude compressive excursions, a discrete plastic hinge formed at the mid-length of the brace during inelastic buckling (Figure 10d) which eventually resulted in local buckling of the HSS member’s wall. In 3 out of the 4 tests, brace fracture occurred prior to reaching the displacement limit of the test frame, and in all of these cases failure occurred at the midpoint of the brace, away from the connections (Figure 10e). Figure 11 shows the hysteretic response generated during one of the brace-assembly tests. In the plot, the force transmitted axially through the brace is plotted against the imparted axial strain in the HSS segment.

Figure 9: (a) Brace connection detail at beam-column intersection; (b) laboratory setup producing similar boundary conditions as those in the field; (c) laboratory test setup

Figure 10: HSS 8.625 x 0.625 brace-connector assembly: (a) undeformed; (b) experiencing inelastic buckling; (c) plastic hinge formation in free length of gusset plate; (d) localization of plastic hinge in HSS member; (e) fracture of brace at its mid-length

Figure 11: Hysteretic response generated during a mid-slenderness brace-connector assembly test – HSS 6.625 x 0.500 produced to ASTM A500 Grade C
The mid-slenderness brace-connector assembly tests provided further validation of the design philosophy for the connectors, as the braces in these tests underwent all of the expected limit states of typical SCBF and OCBF.

**Quasi-Static Full-Scale Braced Frame Testing**

Recently, a full-scale concentric braced frame with a round HSS brace (HSS 5.563 x 0.375 produced to ASTM A500 Grade C) fitted with the cast steel connectors was tested under quasi-static cyclic loading at the Hydro-Québec Structural Engineering Laboratory at École Polytechnique de Montréal (Figure 12). The beams (W21x93) and columns (W14x233) of the frame were designed to remain fully elastic during the test (and during subsequent frame tests) and the frame was positioned vertically, loaded with twin 225 kip actuators connected on either side of the web of the frame’s upper beam. The supports at the base of the columns, located at the working point formed by the centerlines of the column, beam, and brace, were rotational pins and the beams were fastened to the columns using double angles. The center-to-center bay width and height of the frame were approximately 295-inches and 160-inches, respectively. Given the relatively shallow angle of the frame, the brace was connected to the beams only, rather than to both the beams and columns. As the beams were to be reused in subsequent frame tests, the braces were connected to the beams via shaped WT18x97 sections bolted to the beam flanges. To reduce the load transferred through the column base pin connections, axial force in the bottom beam was taken out of the frame via a horizontal reaction block that was pretensioned to the laboratory’s strong floor.

Symmetric quasi-static cyclic loading was applied to the frame at increasing story drift levels up to 4-percent drift. The frame exhibited the typical behavior expected of a ductile concentrically braced frame undergoing cyclic inelastic loading. The first low amplitude cycles showed only minor inelasticity in the system; elastic brace buckling was observed early in the protocol. Subsequent larger compressive excursions resulted in out-of-plane inelastic buckling of the bracing, with significant tensile yielding occurring over the majority of the HSS segment’s length during the higher amplitude tensile excursions, as evidenced by the loss of white-wash (Figure 13). As intended, plastic hinges formed beyond the ends of the cast connectors within the free length of the gusset plates during overall member buckling (Figure 14). At higher amplitude compressive excursions, a more discrete plastic hinge formed at the mid-length of the brace during inelastic buckling, however, the brace could not be fractured within the displacement capabilities of the test frame. White-wash applied to the cast connectors remained intact throughout testing and no sign of yielding could be

![Figure 12: Full-scale braced frame with brace equipped with cast connectors prior to testing](image12)

![Figure 13: Full-scale braced frame with brace equipped with cast connectors during final tensile cycle; a loss of white-wash along the length of the HSS segment is clearly observable, indicating significant yielding of the brace](image13)

![Figure 14: Formation of plastic hinge in the free length of the gusset plate, beyond the end of the connector, as induced during out-of-plane inelastic buckling of the brace](image14)
observed on the connectors during or after the test. Figure 15 shows the hysteretic response generated during the frame test.

Conclusions

High-strength cast steel connectors for round hollow structural section brace members have been developed at the University of Toronto for use in Special and Ordinary Concentrically Braced Frames (SCBF and OCBF). The connectors have been capacity designed pursuant to the AISC Seismic Provisions’ requirements for SCBF and OCBF brace connections and brace assemblies fitted with the connectors have been subjected to a number of structural tests to verify the connector design philosophy. The full suite of tests included monotonic tensile testing and quasi-static cyclic inelastic testing of connector-brace assemblies having a range of overall and cross-sectional slenderness and the testing of a full-scale braced frame fitted with the connectors. In all tests, the braces survived several inelastic cycles beyond the test protocols that were applied and ultimately failed during a tensile excursion at the center of the brace. The cast steel connectors showed no sign of yielding in all tests, validating the overall design rationale for, and performance of, the connectors in seismic-resistant braced frame applications. The connectors, now available from Cast Connex Corporation, have been used in the construction of a number of seismic-resistant braced frame structures in North America.

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