Cast Steel Connectors for Circular Hollow Section Braces under Inelastic Cyclic Loading

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Abstract: Although concentrically braced frames are an efficient means of providing lateral support in steel structures, many such frames designed to resist seismic loading and featuring hollow structural section braces have performed poorly in recent earthquakes—primarily due to unexpected connection failures. To address this issue, the use of a cast steel connector that fits between a tubular brace and a gusset plate is presented as an alternative to the reinforced, fabricated connections that are commonly used in seismic load resisting braced frames. The resulting connector was shaped using solid modeling software, verified by finite-element analysis, and finally cast to ASTM A958 standards. Laboratory results from static and cyclic testing of concentrically loaded brace-connector assemblies showed that the use of a cast steel connector is a viable means of connecting to tubular brace members for seismic (or even static) applications. Further, as the connector was crafted to fit a range of tubular members and can be mass produced, the proposed solution is an economical alternative to conventional tube-to-gusset connections under seismic loading.

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Introduction

Concentrically braced frames are a particularly popular choice for the lateral force resisting systems of steel structures because of their design simplicity, their low cost, the ease with which they are constructed, and the increased stiffness they provide over other lateral systems. The diagonal brace members of braced frames are subject to predominately axial forces, and in the event of an earthquake, seismic energy is dissipated through the cyclic yielding in tension and inelastic buckling in compression of the brace members. Typical bracing members include angles, channels, W-sections, rectangular hollow sections (RHS), and circular hollow sections (CHS). Hollow structural sections (HSSs) are a common selection for lateral bracing members because of their efficiency in carrying compressive loads, their improved aesthetic appearance, and because of the wide range of section sizes that are readily available. Further, although all brace members exhibit degrading compression strength during inelastic cyclic loading due to the Baushinger effect, residual out-of-plane deformations, and local buckling (Tremblay 2001), research has shown that HSS brace members suffer less degradation of their compressive strength and energy dissipation than other structural sections (Lee and Bruneau 2005).

Initially, CHS members were considered best suited for seismic bracing applications. However, connection issues relating to shearlag and net section fracture at gusset connections led to the increased use of RHS braces because of the relative ease with which their connections can be reinforced. This has been an unfortunate industry response, because although the cold-forming manufacturing process of any HSS shape imparts residual stresses, the additional cold working required in forming RHS results in higher residual stresses in the section's corners. This leads to a local reduction in the material's ductility and increases the likelihood of premature fracture due to local buckling in RHS braces. In addition, North American RHSs have been shown to have inherently poor notch toughness (Kosteski et al. 2005), which severely limits their ability to resist impact loadings. While CHS braces have also been shown to exhibit degrading compression strength during cyclic inelastic loading (Elchalakani et al. 2003), because of the lesser degree of cold working required to form the shape and the section's symmetry about any axis, CHS members generally behave more favorably than RHS during cyclic inelastic loading.

Concentrically braced frames that are designed to resist seismic loading are subject to a variety of code requirements, as are the brace members that make up the lateral force resisting system (AISC 2005b,a, CSA 2001). These requirements, which are meant to ensure a stable building response in the event of an earthquake, include a limitation on the building's height and member requirements that focus on brace cross-section slenderness and overall member slenderness. AISC and CSA standards also stipulate stringent requirements for the brace connections themselves, as the tensile loads in the brace members are expected to exceed the nominal yield force of the brace, A_gF_y. Work done by Yang and Mahin (2005) and Haddad and Tremblay (2006) has shown that

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carefully detailed reinforced RHS-to-gusset connections can withstand the prerequisite amount of inelastic cyclic loading of the brace member. Unfortunately, in practice these connections must be individually designed for every brace by a certified engineer, with the design of each connection requiring careful detailing and relying heavily on an appropriate selection of the material's overstrength factor (R_y). At present there is a wide variation and much dispute over the overstrength value in current American and Canadian steel design standards. Further, fabrication of the reinforced RHS connections is costly, with the finished product being unsightly for exposed steel applications and providing a poorer overall building response than would arise from the use of CHS brace members.

Recently, research carried out by Martinez-Saucedo (2006) has shown that circular tube-to-gusset connections can be designed to carry cyclic loads exceeding the tube's nominal yield strength by using a connection detail in which both the tube and the gusset plate are slotted and fitted together. While this detail, termed a "modified-hidden-gap" connection, represents a promising step in the right direction, its proportioning still requires an accurate estimate of the brace member's yield overstrength, precise fabrication, and finite-element analysis to verify that the connection does not exhibit excessive distortion in the slotted region of the tube. This is required to ensure that the brace's inelastic load life is governed by the brace member itself rather than that of the connection.

To provide a more practical solution for seismically loaded brace connections, the writers investigated the use of a cast steel connector. This paper presents the design rationale, the analytical design process, laboratory test methods, test results, and finite-element analysis validation for the development of a cast steel, CHS brace-to-gusset connector.

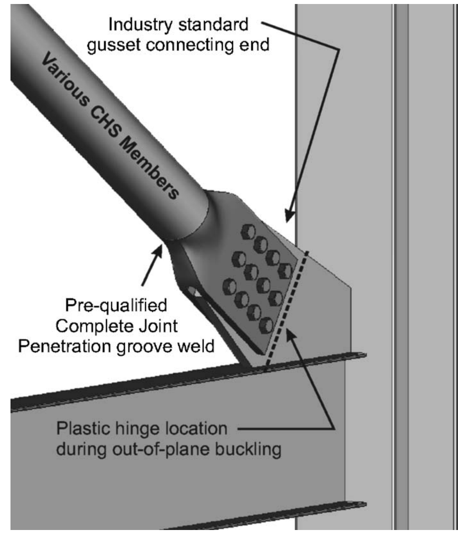

Fig. 1. Proposed cast steel connector shown in building frame

Cast Steel Seismic Brace Connector

The modern resurgence in the use of steel castings in structural engineering has been well documented (Armitage 1983; Marston 1990; Veselcic et al. 2003; Schober 2003; Fleischman et al. 2004; Poweleit and Monroe 2004; and de Oliveira et al. 2006). Casting manufacturing provides the geometric freedom to design a cast steel connector that fits between a CHS brace member and a single gusset plate (Fig. 1). In this configuration, two castings can be shop welded to the ends of a brace member allowing for bolted installation of the brace-connector assembly in the building frame. The connector itself can be shaped to accommodate any standard CHS section of a given outer diameter (i.e., variable wall thickness) with a prequalified complete joint penetration (CJP) groove weld to the tube. The benefits of this connection are numerous. First, provided that an appropriate weld electrode is selected and that a prequalified welding procedure specification is followed, the resulting CJP groove weld between the tube and the connector is guaranteed to be stronger than the brace member regardless of the tube's overstrength. Further, the tapered nose detail on the casting allows for the same welding protocol to be used for a square cut CHS brace of any thickness (Fig. 2). Since each connector can fit a range of section sizes, a small number of connectors—one for each standard tube outer diameter—would cover the entire range of brace options, thus one or two connector types can be used throughout an entire building structure with the appropriate story shear being achieved by varying the wall thickness of the brace. An additional benefit of the proposed cast steel connector design is that since casting manufacturing is predisposed to mass production, the cost per connector can be dramatically reduced with repetition, undercutting the cost of individually designing, detailing, and fabricating the reinforced, fabricated HSS-to-gusset connections. The improved aesthetics of the compact and streamlined connector also promotes its use in architecturally exposed steel applications.

Attachment of the other end of the connector to a single gusset plate accommodates simple fabrication, construction, and site erection. Leaving a gap of at least twice the gusset plate thickness between the end of the connector and any structural element is sufficient to ensure that the plastic hinge that forms at the brace end during compressive buckling will occur in the gusset plate rather than in the brace (Astaneh-Asl 1998; CISC 2006). Fig. 3 shows how forcing the plastic hinge to form in the gusset plate rather than the tube greatly reduces the bending moment demand put on the cast connector at the location of the weld between the connector and the tube.

Connector Prototype Design

The initial casting design was carried out in two-dimensions, accounting for the boundary conditions at each end of the connector. Attachment to CHS having an outer diameter of 168 mm was selected for the connector prototype for a number of reasons. CHS with an outer diameter of 168 mm, and with a wide range of wall thicknesses, are readily available from most steel tube manufacturers in North America as this is a common pipe size. Further, the nominal radius of gyration for most of the available 168 mm tubes provides slenderness ratios that are below 200 at typical brace member lengths (a requirement for tension-compression

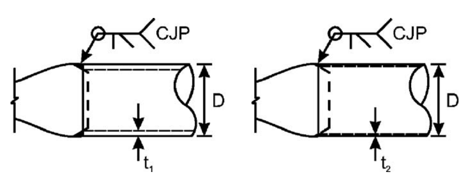

Fig. 2. Connector accommodates various tube sizes with same CJP groove weld detail

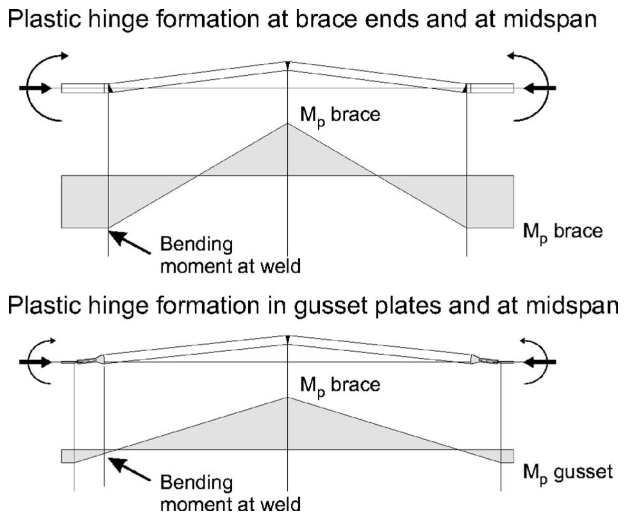

Fig. 3. Bending moment distributions resulting from formation of plastic hinges during inelastic brace buckling (plotted on tension side)

braces). Finally, the yield capacity of the 168-mm-diameter tube ranges from approximately 550 to 3,000 kN (125–675 kip), depending on the wall thickness and steel grade. This gives the designer the ability to provide the appropriate level of lateral strength to each story of a medium-rise structure while specifying the same cast connector.

The bolted end of the casting was designed to resist the highest probable yield strength of the thickest walled 168 mm CHS brace members that are typically available in North America: HSS 168×13 CAN/CSA-G40.20/G40.21 (CSA 2004) Grade 350W and HSS 6.625×0.500 ASTM A500 (ASTM 2003a) Grade C. This was achieved using 12 1-in.-diameter ASTM A490 (ASTM 2006b) bolts for connection to a 30 mm gusset plate. The 12 pretensioned high-strength bolts provide sufficient slip resistance (assuming a blast cleaned faying surface) to carry the probable yield strength of the largest available 168 mm CHS. While providing a slip-critical connection is not specifically required in the United States or Canadian codes (yet the use of pretensioned bolts is), slip-critical connections perform better under cyclic loading regimes and are preferred in seismic applications. Further, increasing the number of bolts beyond the number that would be required to satisfy the bolt shearing design requirement ensures that the connector will remain virtually fully elastic in the bolt region. This could allow for reuse of the connector after a seismic event. In practice, the designer can specify the number of bolts to use based on the strength of the connected tube. When fewer bolts are required, the designer has the option of cutting away any extra length of the connector tab to reduce the required gusset length, if so desired.

The design of the casting geometry was carried out by using three-dimensional finite-element solid modeling software, by carefully optimizing the flow of force though the connector, and with consideration of the limitations of casting manufacturing. For sand casting, the steel casting process most commonly used for structural engineering-sized components and the process used for the manufacture of the connector prototypes, transitional geometry must be kept as smooth as possible to ensure quality casting. Further, the casting's geometry should be conducive to directional solidification, thereby reducing the need for risering and other special and costly casting considerations. Risering is the

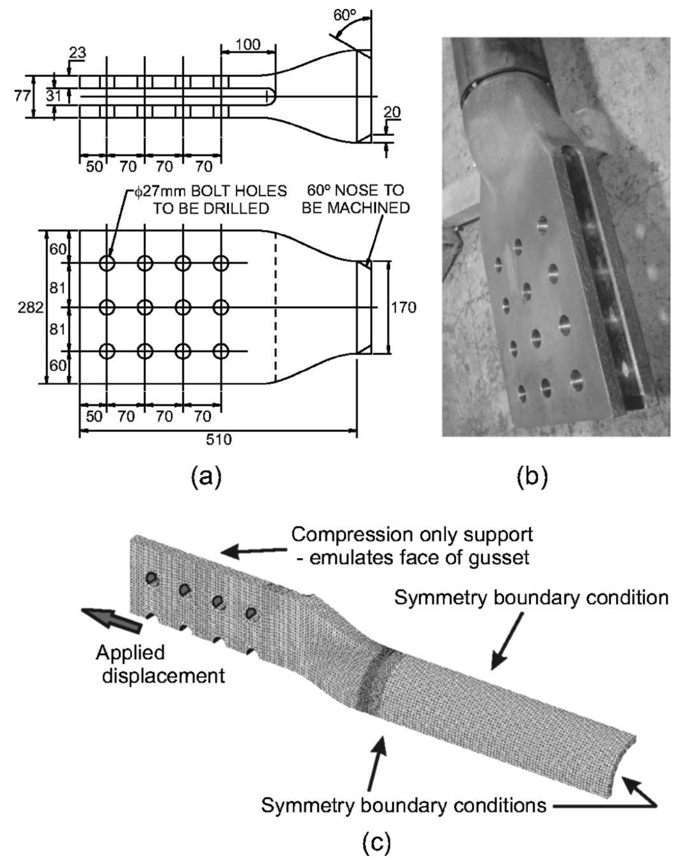

Fig. 4. Cast connector: (a) dimensional drawing; (b) prototype connector tack welded to tube; and (c) finite-element model of prototype connector and CHS tube

attachment of an exothermic sleeve to the surface of a casting to provide a liquid metal reservoir that feeds a section of the component which would otherwise solidify last, thereby avoiding the formation of a shrinkage void in the finished product. Risers are cut and ground from the component after the casting has solidified, with their number, size, and distribution having been determined by the foundry through their use of sophisticated finite-element solidification analysis software. Components cast using the sand casting process are also commonly subject to geometric requirements, for example section tapering, commonly referred to as draft. In the sand casting process, a positive replica of the component, called a pattern, is used to form a negative impression in chemically treated sand that hardens to form a mold into which the molten steel is poured. Tapering the component that is to be cast allows for the removal of the component's pattern from the chemically treated sand mold without damaging the mold or the pattern. Depending on the material from which the pattern is produced, patterns can be subsequently reused to form tens to hundreds of sand molds.

As designing a component that can be cast soundly is paramount, it is common to go through several iterations of stress design by the engineer followed by foundry analysis of the part, with recommendations being made by the foundry to improve solidification of the finished product. For the prototype cast connector, foundry suggestions resulted in an 18% increase in the connector's mass. The final dimensions of the connector are shown in Fig. 4(a). Fig. 5 shows the results of a solidification analysis carried out by the foundry retained by the writers for the manufacture of the prototype connectors. In this case, directional

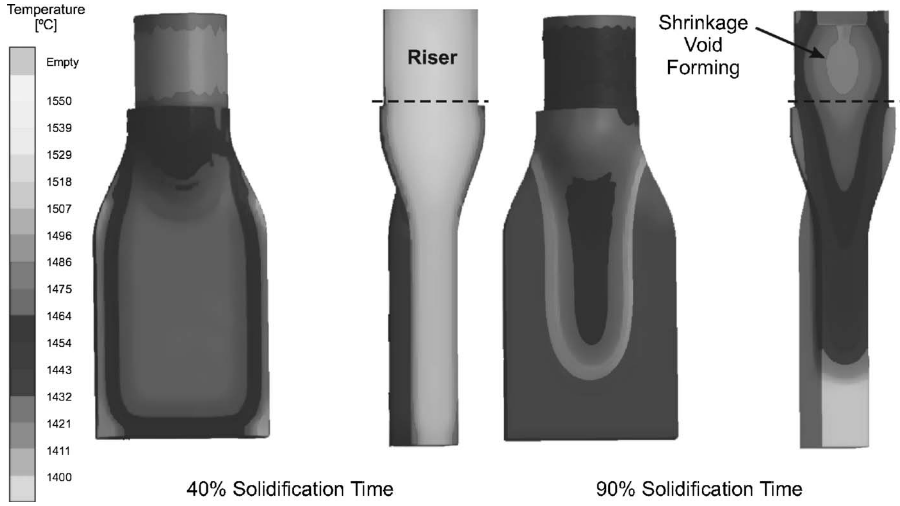

Fig. 5. Time step from solidification analysis carried out by steel casting foundry

solidification was ensured with the use of a riser connected to the top of the cast element, as indicated in the figure. After solidification was complete, the riser, which contained a shrinkage void, was cut away leaving the connector shrinkage void free.

As previously discussed, in typical concentrically braced frames the brace member itself is the energy-absorbing element. Therefore, according to the principles of capacity design, the cast connector must remain elastic during tensile yielding of the brace member, buckling of the brace, or plastic hinging of the brace at midspan and at the two brace ends due to overall or local inelastic buckling. The elastic behavior of the connector was established using finite-element stress analysis during the design process of the prototype connector.

A solid modeling software package, SolidWorks 2005, was used for the three-dimensional design of the connector. This model was forwarded to the foundry for their use in producing the pattern from which the sand molds for the prototypes were made. For the purposes of finite-element stress analysis, the connector model was modified to include a complete joint penetration groove weld between a 500 mm long (3 diameters) HSS 168×13 brace member of nominal diameter and thickness and the cast connector. Because of symmetry, finite-element modeling of only a quarter of the assembly was required. The geometry of the part was exported from SolidWorks directly into ANSYS Workbench v9.0 (SAS 2004). Solid bodies were meshed using higher order three-dimensional tetrahedral solid elements (SOLID187), with each element defined by ten nodes having three degrees of freedom at each node. These elements have quadratic displacement behavior and are best suited for modeling solid bodies that are curved or have irregular boundaries (Moaveni 1999). The mesh model of the connector-brace assembly is shown in Fig. 4(c). Symmetry boundary conditions were required on three faces of the model such that the finite-element model analyzed represented a full brace-connector assembly. As the gusset plate to which the connector bolts keeps the cast connector tabs from moving inward, a "compression only" boundary condition was applied to the inside face of the connector tab. Finally, displacements were applied to the internal faces of the 27-mm-diameter bolt holes over a width of 25.4 mm (1 in.) to reproduce the effects of bolt bearing. It is important to note that the aforementioned boundary conditions do not permit overall brace buckling. The boundary conditions do, however, permit symmetric local buckling of the circular brace member. As a result, the stresses produced during finite-element analysis for compressive loading represented an upper bound on those that would actually be present in the connector during overall brace buckling.

Nonlinear finite-element analysis was carried out with incremental displacements applied to the bearing faces of the bolt holes. In reality, the bolts were pretensioned resulting in load transfer through distributed frictional forces between the cast tabs and the gusset plate; however, application of displacement in this manner adequately emulated static displacement-control loading of the connection assembly. This also produced conservatively large stress concentrations at the bolt holes. Nonlinear material properties were considered and geometrical nonlinearities were taken into account by allowing large deformations, which also permitted shape change during loading. Reduced integration was used for the formulation of the local stiffness matrix of each element. Finite-element analysis confirmed that when the brace assembly was loaded, inelastic deformations were localized in the brace member up to the probable yield capacity of the largest brace size. Further, finite-element stress analysis showed that when a tensile or compressive deformation was applied to the connector that caused a brace force corresponding to the design yield force, stresses in the casting were below the specified minimum yield stress of the cast material selected for the component. The stress distribution in the tubular brace was uniform a short distance from the welded connection, showing none of the shear-lag effects associated with the typical fabricated slotted tube-to-gusset connections. Validation of the finite-element model used for the design of the connector was carried out and is presented at the end of this paper.

Prototype Manufacturing and Welding Considerations

The prototype cast steel connectors that were designed and subsequently tested at the University of Toronto were manufactured with steel produced to ASTM A958 Grade SC8620 Class 80/50 (ASTM 2000). This cast material has a chemical composition similar to that of a standard wrought steel grade and is considered a weldable base metal according to CSA W59 (CSA 2003) pro-

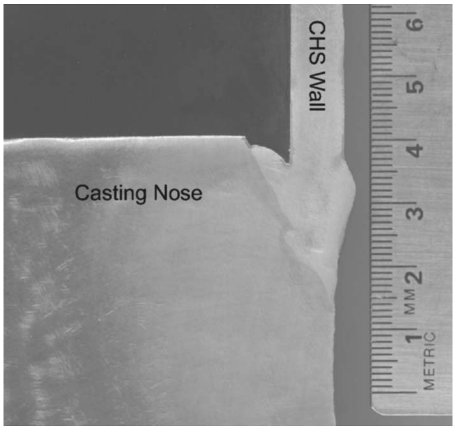

Fig. 6. Section through CJP groove weld between cast connector and CHS member

vided that the silicon content of the casting does not exceed 0.55% by weight. Material produced to this specification has a minimum yield stress of 345 MPa, a minimum ultimate tensile strength of 550 MPa, a minimum elongation of 22%, and a reduction in area of 35% in 50 mm. An additional Charpy V-Notch (CVN) impact test value requirement of 27 J (20 ft·lb) at −20°C (−4°F) was specified to ensure that the connection had a suitable toughness at the weld region between the connector and the brace. This exceeds the toughness requirement specified in CSA (2001) for energy-dissipating elements or welded parts, but more closely corresponds to the CVN requirement for the weld filler material required for dynamically loaded connections.

To ensure that the castings were sound they were subjected to visual examination per ASTM A802 (ASTM 2006a) Level I, nondestructive examination per ASTM A903 (ASTM 2003b) Level III, and ultrasonic examination per ASTM A609 (ASTM 2002) Level 3. Coupons cut directly from a cast connector in two directions revealed a yield stress of 565 MPa, an ultimate tensile strength of 695 MPa, and an isotropic material response.

A professional welding engineer determined an appropriate welding procedure specification for the tube-to-casting CJP groove weld. A prequalification for this CJP welding procedure was authorized by the Canadian Welding Bureau after trial welds performed to the welding procedure by a certified welder were destructively examined by the Bureau. Fig. 6 shows a chemically etched (10% Nital) section cut through the CJP groove weld between a cast connector and a HSS 168×9.5 member. This section clearly shows that the weld fully engages the entire cross section of the CHS. The laboratory tests described below demonstrated that the notch-like condition of the weld had no effect on the strength of the connection.

Experimental Program

Proof-of-concept laboratory testing consisted of inelastic cyclic testing and static tensile testing of connector-brace assemblies as well as destructive examination of a cast connector.

Cyclic testing was carried out in a 2,750 kN (620 kip) capacity MTS universal testing machine on two brace specimens of different brace sizes: HSS 168×6.4 and HSS 168×9.5 (Specimens PSD-1 and PSD-2, respectively) with tubes dual certified to ASTM A500 Grades C and B. Load capacity, testing frame size, and stroke limitations of the machine required that both test speci-

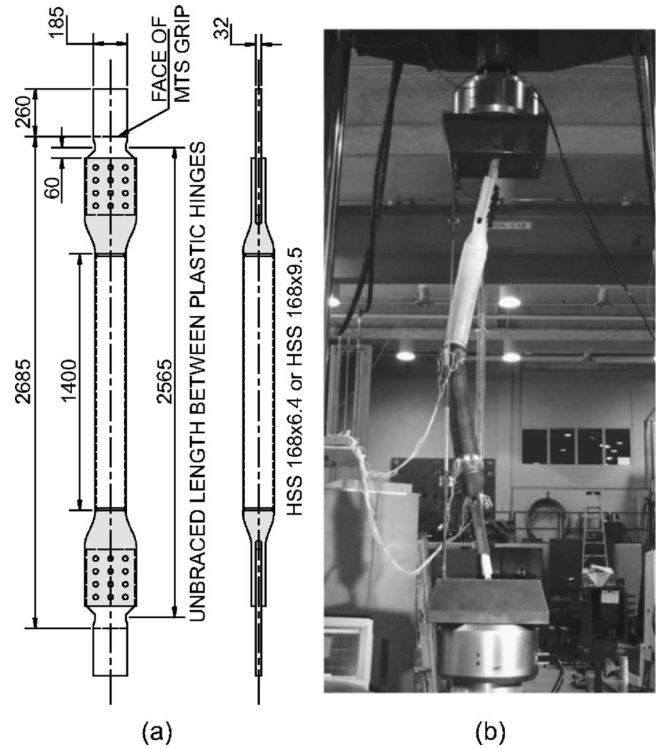

Fig. 7. (a) Specimen details for cyclic testing; (b) specimen during compression loading cycle

mens consist of two cast steel connectors welded to the ends of a 1.4 m long tube. The welding was carried out as per the previously described prequalified welding procedure for achieving complete joint penetration. The connectors were bolted with 12 1-in.-diameter ASTM A490 high strength pretensioned bolts to 32 mm (1-1/4 in.) gusset plates that were shaped to fit the testing frame's hydraulic grips. These gusset plates were dog-boned a distance of 60 mm (approximately two times the gusset plate thickness) away from the ends of the connectors. This detail ensured that formation of plastic hinges in the gusset plates occurred in the same region of the plates as would be expected in the field during overall brace buckling. Fig. 7 shows the specimen details and a specimen experiencing inelastic buckling during a compressive cycle of the cyclic testing procedure.

As the brace members tested were rather short in comparison to typical brace lengths for concentrically braced frames, a cyclic loading protocol was used which ensured that the braces would develop loads in excess of their yield force in tension while still ensuring that several inelastic cycles could be applied to the brace-connector assemblies. The loading protocol, shown schematically in Fig. 8, was based on the elongation associated with the test brace-connector assembly yielding, Δ_y. The protocol itself, carried out in displacement control, began with two cycles to Δ_y in tension and compression. After reaching Δ_y in compression for the second cycle, the brace was tensioned to an elongation associated with 4% story drift in a typical braced building. At this point, the tensile force in the assembly had exceeded the yield force of the brace. Following the attainment of the displacement corresponding to 4% story drift, the applied displacement was then decreased until the zero load point was reached. A compressive displacement was then applied to the assembly to the point at which the short brace began to exhibit a decrease in compressive capacity. A tensile displacement of Δ_y beyond the previously at-

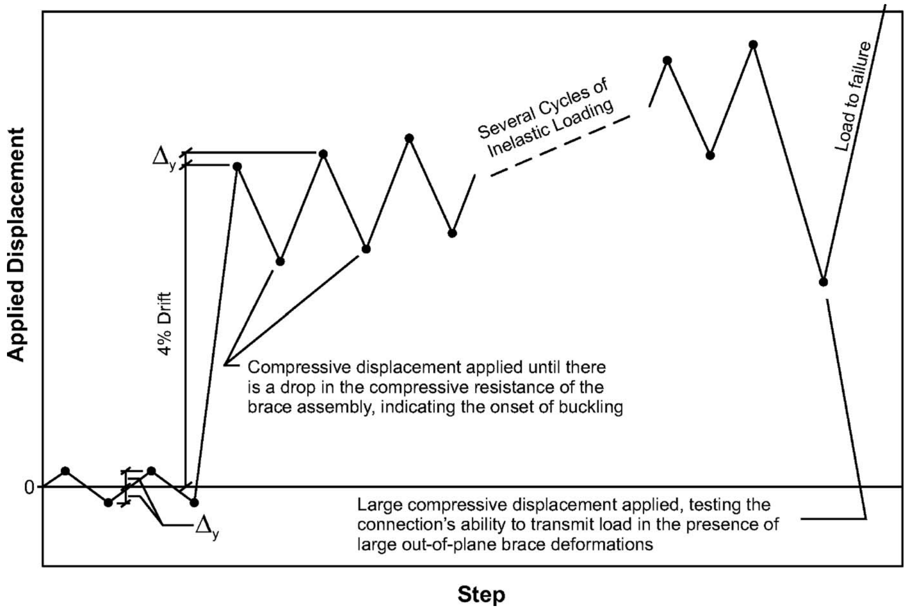

Fig. 8. Loading protocol for cyclic testing of brace-connector assemblies

tained tensile displacement was then applied, with the pattern repeated over several cycles of inelastic loading. Next, a large compressive displacement was applied to the assembly. This axial deformation caused a very large out-of-plane kink to form in the brace, proving that the brace connection was capable of transmitting large compressive forces when the brace was in its buckled configuration. The protocol was then completed by applying increasing tensile deformation until the brace member fractured. Because the brace member was rather short, the compressive forces attained in the brace-connector assembly are much higher than those that would be expected in brace members of typical length, thus the protocol that was applied was conservative with regard to the required compressive resistance of the connection.

After cyclic testing, two cast connectors were cut from the ends of one of the two tested brace-connector assemblies. After machining the connector noses back to their original shape, the two connectors were welded to a 1.4-m long HSS 168 × 13 tubular section, also dual certified to ASTM A500 Grades C and B, for static tensile testing in a 5,350 kN (1,200 kip) capacity testing machine. The cast connectors were in turn bolted to gusset plates that were designed to fit the wedge anchorages of the testing machine. Fig. 9 shows the specimen details (Specimen ST-1) and the fractured test specimen after completion of the static test.

Test Results

Cyclic Testing of Connector-Brace Assemblies

Figs. 10 and 11 show the load-displacement plots produced during the two cyclic tests of brace-connector assemblies PSD-1 (HSS 168 × 6.4) and PSD-2 (HSS 168 × 9.5), respectively. As is evident from the load-displacement plots, the connectors were able to transmit tensile loads in excess of the real (measured) tube yield capacity in both of the tested specimens. Further, the tested connectors were capable of transmitting compressive loads in the presence of large out-of-plane brace deformations. Bolt slip was observed only once for each connector during the testing of Specimen PSD-2 (indicated in Fig. 11) as the slip load was never exceeded in compression or in the testing of Specimen PSD-1. After a few high amplitude tension-compression cycles, the tubes exhibited plastic hinging in the gusset plates and local buckling at their midspan. The final failure mode in both tests was fracture at the brace's midspan after necking of the tube's cross section

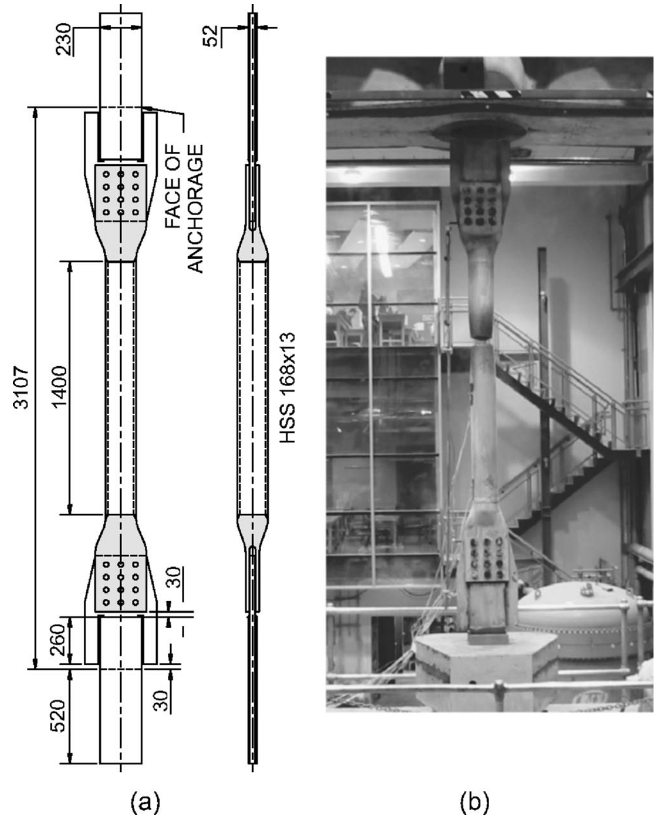

Fig. 9. (a) Specimen details for static tensile testing; (b) specimen after fracture

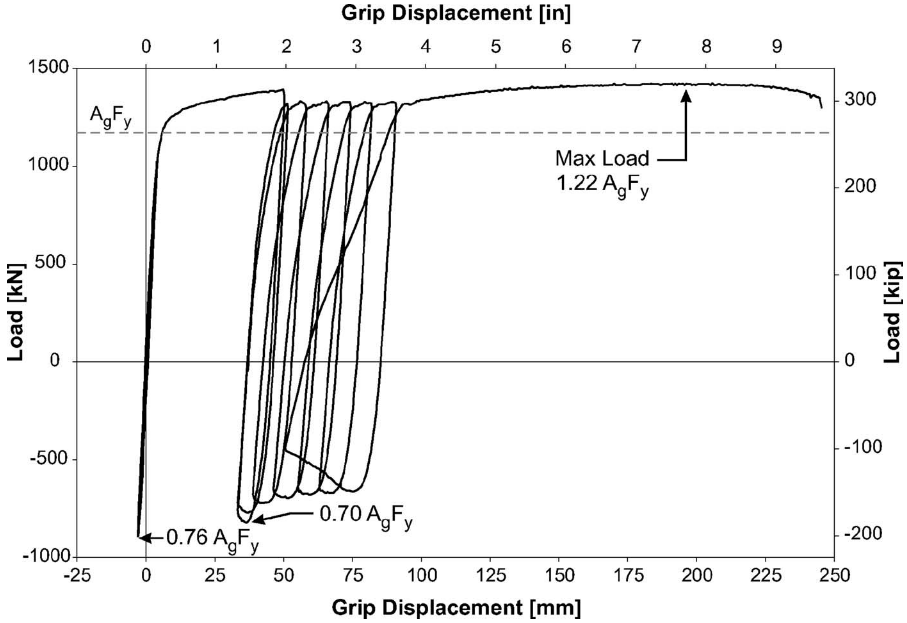

Fig. 10. Cyclic load-displacement plot for PSD-1 (HSS 168×6.4) (F_y=382 MPa as determined by average of several tensile coupon tests and A_g=3,063 mm^2 determined by measurement of tube geometric properties)

had become clearly visible. Strain gauge readings showed that the connectors remained elastic in the gauged regions during both tests and whitewash applied to the connectors did not flake during any stage of the testing (whereas whitewash applied to the CHS section had clearly flaked). No residual bolt hole deformations were visible on any of the connectors after the cyclic testing.

Bolt slippage during testing likely caused some degree of load eccentricity in the connection, as a slip can result in bearing of bolts only to one side of the brace centerline. It is expected that, if a significant eccentric loading was applied, it would cause local bearing deformations, which in turn would result in a redistribution of forces back to the centerline of the brace. As there was no indication of any significant bearing deformations in the bolt holes after testing, it is hypothesized that bolt slippage did not result in significant eccentricity in the connection tested. In practice, a lower coefficient of friction can be assumed in the calculation of the slip-critical connection capacity to eliminate all slippage.

Static Tensile Testing of Connector-Brace Assembly

Fig. 12 shows the load-displacement plot produced during static tensile testing of brace-connector assembly ST-1 (HSS 168×13).

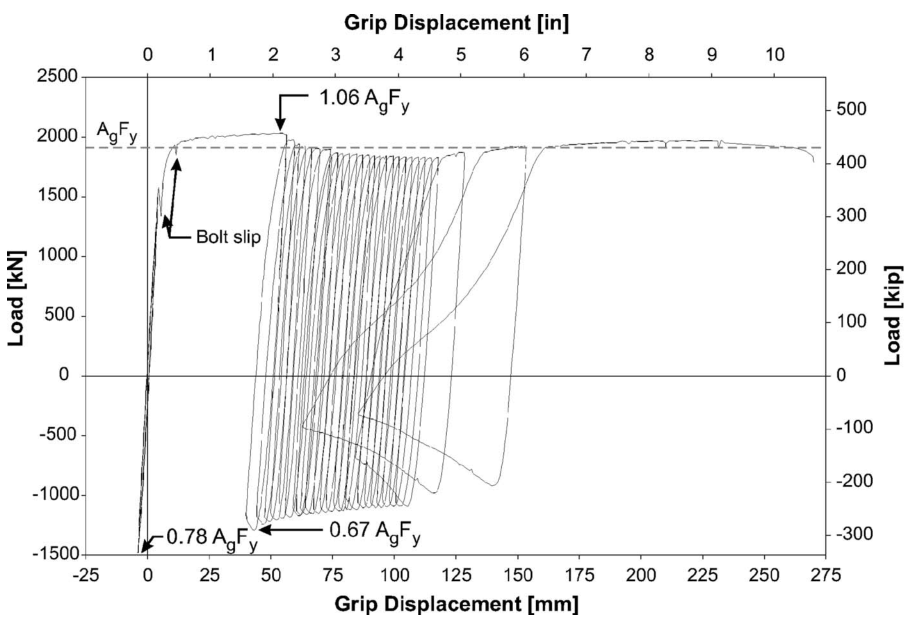

Fig. 11. Cyclic load-displacement plot for PSD-2 (HSS 168×9.5) (F_y=434 MPa as determined by average of several tensile coupon tests and A_g=4,404 mm^2 determined by measurement of tube geometric properties)

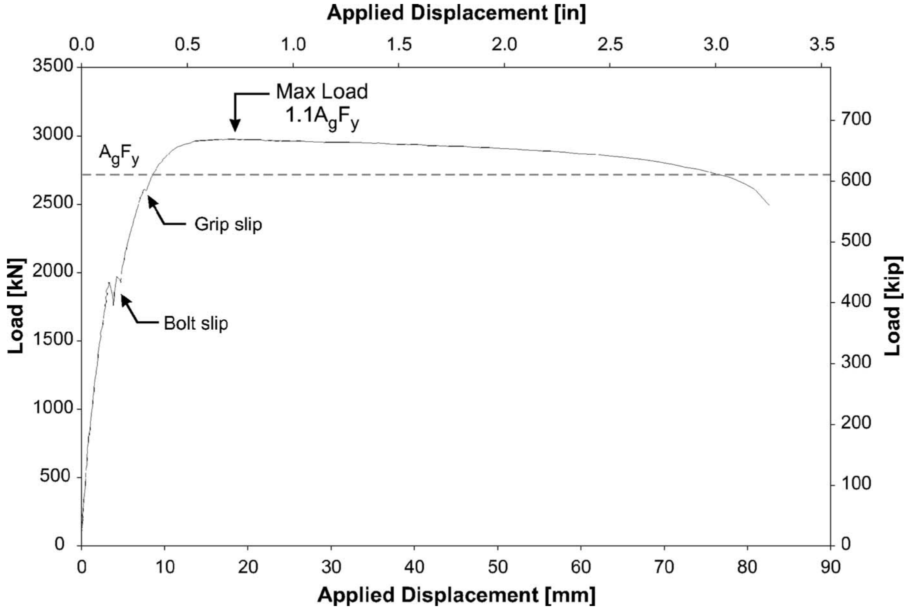

Fig. 12. Static tensile load-displacement plot for ST-1 (HSS 168×13) (F_y=475 MPa as determined by average of several tensile coupon tests and A_g=5,717 mm^2 determined by measurement of tube geometric properties)

During this test, tensile displacement was monotonically increased beyond the peak load of 2,970 kN (670 kip) up to rupture of the specimen, which occurred in the tube a short distance above the brace midpoint after significant tube necking was observed [see Fig. 9(b)]. Readings from strain gauges (located in the same positions as indicated in Fig. 13) showed that the connectors remained elastic in the gauged regions during the test. Whitewash applied to the connectors did not flake during any stage of the testing, whereas whitewash applied to the HSS section flaked, particularly in the necked region of the tube. No residual bolt hole deformations were visible on either connector after static testing.

Finite-Element Validation

Strains and deformations measured during laboratory testing were compared to the results of finite-element analysis for the purpose of validating the numerical models that were used for the design of the connector. Unlike the initial analysis which used assumed material properties and nominal tube dimensions, finite-element analysis performed for the purpose of validation was carried out using material properties determined from tensile testing of coupons cut from the brace members as well as the measured tube dimensions. The cast steel material could be assumed elastic

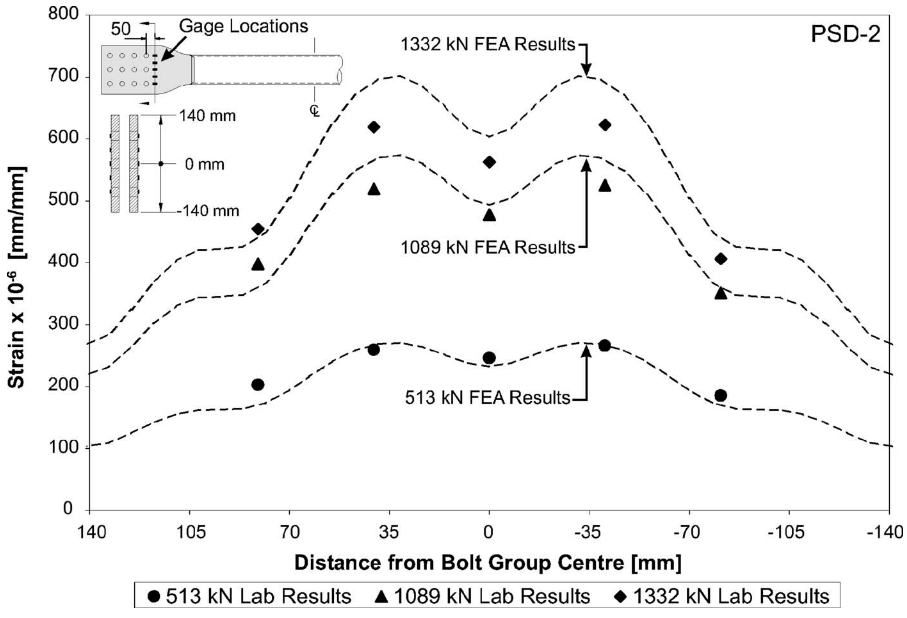

Fig. 13. Measured surface strains across cast node and finite-element predictions, at given brace loads

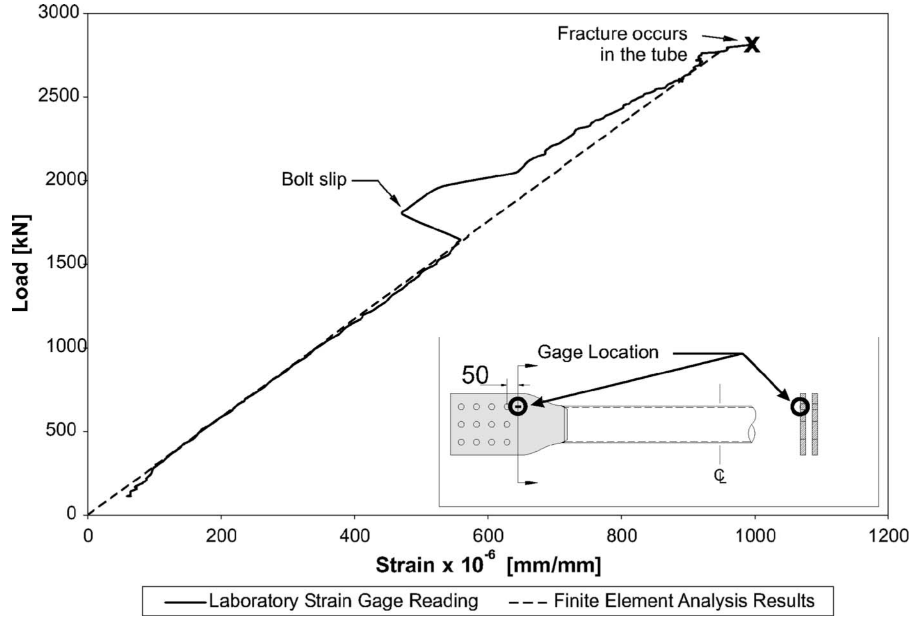

Fig. 14. Measured strain and finite-element prediction for point on cast connector (ST-1)

due to the high yield strength. Otherwise, all other finite-element modeling assumptions were the same as those previously described. Fig. 13 shows experimental surface strains in the connector compared to strains predicted by the finite-element model at several load levels measured during testing of Specimen PSD-2. The strain distribution predicted by finite-element analysis across the width of the cast connector is consistent with the measured distribution. Strains and deformations measured during static tensile testing were also compared to the results of finite-element analysis. As seen in Fig. 14, the cast connector remained elastic throughout the duration of the test. The correlation between laboratory measurements and the finite-element results serves as validation of the finite-element modeling conducted for the purpose of designing the connector.

Conclusions

This paper presents the use of a cast steel, CHS brace-to-gusset connector as an alternative to the reinforced, fabricated HSS connections currently used in seismic load resisting braced frames. Laboratory testing of prototypes has shown that the use of cast connectors is an effective and viable solution to many of the connection issues that are associated with seismically loaded tube-to-gusset connections. Further validation is necessary to confirm that this excellent performance is also achieved in braces of typical lengths, under a variety of loading protocols, and with testing criteria that more accurately reproduce the boundary conditions expected in the field, particularly with respect to member end rotations.

As casting manufacturing allows for the mass production of connectors that can accommodate complete joint penetration groove weld connections to a range of tube sizes, these connectors represent a viable alternative for engineers to connect steel bracing members in buildings designed to resist seismic loading. The standardization and prequalification of such connectors have the potential to reduce the uncertainty that currently exists with the one-off design of engineered and fabricated connections and to address the concerns of the engineering community about the seismic performance of fabricated brace HSS connections.

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Notation

The following symbols are used in this paper:

A_g = gross cross-sectional area of hollow section;
D = outside diameter of circular hollow section;
F_y = yield tensile stress;
M_p = plastic moment of section experiencing plastic hinging;
R_y = ratio of expected yield stress to specified minimum yield stress;
t = wall thickness of CHS; and
Δ_y = specimen yield deformation in tension.

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