Cast steel connectors for seismic-resistant tubular bracing applications

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ABSTRACT: While concentrically braced frames are among the most popular lateral load resisting systems in North America for medium- to low-rise steel structures, many such frames featuring hollow structural section (HSS) brace members have been shown to be prone to unexpected connection failure in the event of an earthquake. This paper presents the use of a cast steel connector as an improvement over the reinforced, fabricated HSS bracing connections that are commonly used in seismic load resisting braced frames. The cast connector developed was shaped using solid modeling software, verified by finite element analysis, and cast to ASTM A958 standards. The connector concept was subsequently validated through cyclic inelastic and monotonic tensile testing of concentrically loaded HSS brace-connector assemblies. Since the completion of this research, this technology has been commercialized and may herald a new era of connection design for seismic-resistant HSS bracing.

1 INTRODUCTION

In North America, concentrically braced frames are amongst the most common lateral load resisting systems for medium- to low-rise steel structures. This is due to design and erection simplicity and the increased stiffness the system provides in comparison to other lateral systems. Hollow structural sections (HSS) are frequently specified as the bracing elements in braced frames due to their efficiency in carrying compressive loads, their improved aesthetic appearance, and because of the wide range of section sizes that are readily available.

In concentrically braced frames, the diagonal bracing element is subjected to predominantly axial loading. Statically loaded HSS braces are typically connected to the beam, column, or beam-column intersection via slotted HSS-to-gusset connections. Design of these connections for static 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 2006); these connections can be readily detailed according to prevailing design standards. However, in the event of an earthquake, the diagonal brace members of concentrically braced frames cyclically yield in tension and buckle in compression, thereby dissipating seismic energy. For this reason, brace connections in braced frames designed to be seismically resistant must be stronger than the probable tensile yield capacity of the connected brace member including any expected material overstrength.

Both in the laboratory and in the field as witnessed during post-earthquake reconnaissance (Tremblay et al. 1996, AIJ 1995, Bonneville and Bartoletti 1996), conventional slotted HSS-to-gusset connections have been shown to fail when subjected to cyclic inelastic loading. These premature failures, which may lead to structural collapse, occur due to a concentration of inelastic strain at the reduced section of the HSS connection. Thus, current North American seismic design provisions recommend the use of net-section reinforcement whenever slotted HSS-to-gusset connections are specified in seismic-resistant frames. As reinforcement is more readily fabricated for flat surfaces, the industry has tended toward the specification of square or rectangular HSS rather than circular HSS for seismic resistant braces. This is an unfortunate circumstance as the energy absorbing capability of North American cold-formed rectangular HSS under cyclic inelastic loading is now thought to be less than that implied by the current design provisions (Lee and Goel 1987, Shaback and Brown 2003). Further complicating seismic-resistant HSS connection design, there is wide variation and much dispute over the overstrength value for HSS steel grades in current American and Canadian steel design standards. To ensure HSS remain the industry choice for seismic resistant bracing, a connection detail for circular hol-
low section (CHS) braces that can withstand inelastic cyclic loading of the brace is required.

2 CAST STEEL SEISMIC BRACE CONNECTOR

In an effort to address the seismic brace connection dilemma, the author, under the supervision of Professors Jeffrey A. Packer and Constantin Christopoulos of the University of Toronto, developed a component to eliminate shear-lag in CHS-to-gusset connections that could also withstand cyclic inelastic loading of the CHS member. The component was designed to be shop welded to the ends of a CHS segment, with the connector-CHS assembly to be subsequently field-bolted to gusset plates in a steel braced frame as shown in Figure 1.

Due to the geometric complexity of the connector, the components were designed to be produced using steel casting manufacturing. As casting manufacturing is predisposed to mass production, the research team elected to standardize the cast steel connectors, which was achieved as follows. The CHS connecting end of the cast connector was designed to accommodate any CHS of a given outer diameter, regardless of the section’s wall thickness (Fig. 2). Since the CHS is welded to the connector using a CJP weld, any grade of CHS brace can be used, provided that the appropriate weld electrode is selected.

The gusset-receiving end of the connector was designed to accommodate a slip-critical connection for a load equal to the expected yield capacity of the heaviest-walled CHS section that could be accommodated by the connector, including material overstrength. This feature provides the option to design the connection using slip-critical or bearing type bolted connections for any size of CHS, with the number of bolts specified by the designer commensurate with the expected strength of the connected CHS member. This design flexibility also allows the designer to use whichever overstrength value is specified by the governing design code without having to completely re-design the connection (North American codified values for CHS material overstrength currently range from 1.1 to 1.6 depending on the grade and seismic design code). This presents a significant advantage over traditional reinforced HSS-to-gusset connections which entail detailing that is more sensitive to the grade and size of the connected HSS brace member. Further, the ability to shop-weld the CHS segment to the connector and field-bolt the connector-brace assembly to the frame eliminates the need for field welding of seismic-critical welds. Finally, the double-shear bolted connection on the gusset receiving end of the connector halves the number of bolts that would otherwise be required to bolt a shop-welded brace to the frame using splice plates. In sum, use of the standardized connector significantly simplifies connection design, detailing, fabrication, and erection of seismic-resistant HSS brace connections.

3 CAST CONNECTOR PROTOTYPE DESIGN

To validate the seismic-resistant connector concept through laboratory experimentation, a connector having the features presented above was required. A prototype connector was designed to accommodate CHS having an outer diameter of 168 mm. This size was selected for several reasons. First, CHS of this diameter is readily available with a wide range of wall thicknesses from most North America steel tube manufacturers. Further, the
nominal radius of gyration for most of the available 168 mm diameter tubes provides slenderness ratios that are below 200 at typical brace member lengths (a requirement for tension-compression braces). Finally, the yield capacity of 168 mm diameter CHS ranges from approximately 550 to 3,000 kN, depending on the wall thickness and steel grade selected. This wide range of yield forces provides the designer with the ability to select the appropriate level of lateral strength for each storey of a medium-rise structure while specifying the same cast connector throughout the building.

For the purpose of standardization, the gusset receiving end of the prototype connector 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 168x13 CAN/CSA-G40.20/G40.21 (CSA 2004) Grade 350W and HSS 6.625x0.500 ASTM A500 (ASTM 2003) Grade C. This was achieved using 12 1-inch diameter ASTM A490 (ASTM 2006) 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. Although the use of pretensioned bolts is required by North American seismic design codes, the use of slip-critical connections is not. However, increasing the number of bolts beyond the number that would be required for a bearing-type connection ensures that the connector will remain virtually fully elastic in the bolt region during a seismic event. This results in a more robust connection and ensures that all inelastic yielding is confined to the CHS segment.

Design of the prototype cast steel connector was carried out using 3-dimensional solid modeling software with consideration for the flow of force though the connector as well as the limitations of casting manufacturing. For sand casting, the steel casting process used to produce the prototype connectors, transitional geometry must be kept as smooth as possible to ensure quality casting. Further, it is beneficial for cast components to be shaped to promote directional solidification, thereby reducing the need for risering, chilling, or other costly casting considerations. Finally, components cast using the sand casting process must taper slightly from the parting line between the moulds into which the molten steel is poured. The foundry that was retained to manufacture the prototype connectors suggested minor design changes which improved the final component’s castability. Figure 3a shows the prototype’s final dimensions while Figure 3b shows a photograph of a connector tack-welded to a CHS segment.

In typical concentrically braced frames, the brace member itself is the energy-absorbing element. Thus, according to the principles of capacity design, the cast connector must remain elastic during tensile yielding or compressive buckling of the brace. The elastic behaviour of the connector was established using finite element stress analysis prior to manufacturing.

For the purposes of finite element stress analysis, a 3-dimensional solid model of the connector was produced which also included a complete joint penetration groove weld between a 500 mm long (3 diameters) HSS 168x13 brace member of nominal diameter and thickness. Because of symmetry, finite element modeling of only a quarter of the assembly was required, as shown in Figure 3c. Finite element analysis was carried out in ANSYS (SAS 2004). Solid bodies were meshed using elements defined by 10 nodes and having three degrees of freedom at each node. As discussed by Moaveni (1999), higher-order elements like these are best suited for modeling solid bodies that are curved or have irregular boundaries.

Figure 3. Prototype cast connector: (a) dimensional drawing, (b) connector tack-welded to CHS, (c) finite element model of prototype and CHS segment

![Figure 3. Prototype cast connector: (a) dimensional drawing, (b) connector tack-welded to CHS, (c) finite element model of prototype and CHS segment](image-url)
none of the shear lag effects associated with slotted tube-to-gusset connections. Validation of the finite element model used for the design of the connector was carried out and is presented below.

4 PROTOTYPE MANUFACTURING AND WELDING CONSIDERATIONS

The prototype cast steel connectors produced for this study 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 (2003). 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 impact test value requirement of 27 Joules at -20°C was specified to ensure the connection had a suitable toughness at the weld region between the connector and the brace.

To ensure that the castings were sound they were subjected to visual examination, non-destructive examination, and ultrasonic examination by the foundry. 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 complete joint penetration groove weld. Pre-qualification for this CJP welding procedure was authorized by the Canadian Welding Bureau after trial welds performed to the procedure by a certified welder were destructively examined by the Bureau. Figure 4 shows a chemically etched (10% Nital) section cut through the CJP groove weld between a cast connector and a HSS 168x9.5 member. This section clearly shows that the weld fully engages the entire cross-section of the CHS.

5 EXPERIMENTAL PROGRAM

Laboratory validation of the connector concept consisted of both cyclic inelastic and monotonic tensile testing of brace-connector assemblies. Cyclic testing was carried out on two brace-connector assemblies fabricated with different circular hollow sections that had been certified to both ASTM A500 Grade C and Grade B: HSS 168x6.4 and HSS 168x9.5. Testing was carried out in a 2750 kN MTS testing frame; size and stroke limitations of the machine required that both test specimens be made up of two cast steel connectors welded to the ends of a 1.4-metre long CHS segment. Welding between the CHS and cast connectors was carried out according to the pre-qualified CJP welding procedure previously described. The connectors were bolted with 12 1-inch diameter ASTM A490 high strength pretensioned bolts to 32 mm gusset plates that were shaped to fit the testing frame’s hydraulic grips.

As the brace members tested were rather stocky (nominal slenderness, KL/r, in the range of 45) in comparison to typical brace lengths for concentrically braced frames, a cyclic loading protocol was developed which ensured that several inelastic cycles could be applied to the brace-connector assemblies prior to inducing fracture in the CHS segment. This was achieved by applying a non-symmetric protocol that cycled between tensile yielding of the brace-assembly and a compressive displacement associated with the onset of buckling. Once the compressive force being transmitted through the brace began to decrease, loading was reversed, thereby limiting the damage imparted to the CHS brace while maximizing the load transferred through the connections. After the pattern had been repeated over several cycles of inelastic loading, 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. This deformation also resulted in the onset of local buckling at the plastic hinge which formed at the midspan of the brace. The protocol was then completed by applying increasing tensile deformation until the brace member fractured. Because the braces tested in this manner were stocky, the compressive forces attained in the brace-connector assembly were 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. Figure 5 shows the brace-connector assembly details and a specimen experiencing inelastic buckling during a compressive cycle of the cyclic testing procedure.

After cyclic testing, two cast connectors were cut from the ends of one of the two brace-connector assemblies that had been tested. After machining the connector noses back to their original shape, the two connectors were welded to a 1.4-metre long HSS 168x13 tubular section, which had also been dual-certified to ASTM A500 Grades C and B, for tensile testing in a 5350 kN capacity testing machine. Figure 6 shows the specimen details and the fractured test specimen after completion of the monotonic tensile test.

6 TEST RESULTS

Figures 7 and 8 show the load-displacement plots produced during the cyclic testing of the HSS 168x6.4 and HSS 168x9.5 brace-connector assemblies, respectively. As is evident from the figures, the connectors were able to transmit tensile loads in excess of the measured CHS yield capacity in both of the tested specimens. Note that the peak tensile loads induced in the braces are significantly higher than those that would have been required by both the Canadian and American seismic design standards, particularly if the designer had specified ASTM A500 Grade B material but had received the dual-certified material used for these tests. A connection detailed using traditional slotted HSS-to-gusset connections and designed to carry the codified expected force for the ASTM A500 Grade B material would very likely have fractured if it were subject to the same test protocol.

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 the HSS 168x9.5 as the slip load was never exceeded in compression or in the testing of the HSS 168x6.4 brace assembly. After several high amplitude tension-compression cycles, the brace-assemblies exhibited plastic hinging at their gusset plates and local buckling at the tube section’s midspan. The final failure mode in both tests was fracture at the brace’s midspan after necking of the tube’s cross-section had become clearly visible. Strain gauge readings and whitewash applied to the connectors showed that the connectors remained elastic during both tests.

Figure 9 shows the load-displacement plot produced during tensile testing of the HSS 168x13 brace-connector assembly. During this test, tensile displacement was monotonically increased beyond the peak load of 2970 kN up to rupture of the specimen, which occurred in the CHS segment a short distance above the brace midpoint after significant tube necking was observed. Readings from strain gauges showed that the connectors remained elastic.
in the gauged regions during the test. As with the cyclically tested specimens, whitewash applied to the connectors did not flake during any stage of the testing whereas whitewash applied to the CHS segment flaked, particularly in the necked region of the tube.

7 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 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. Otherwise, all other finite element modeling assumptions were the same as those previously described. Figure 10 shows experimental surface strains in the connector compared to strains predicted by the finite element model at several load levels measured during testing of the HSS 168x9.5 assembly.

As seen in the figure above, the strain distribution predicted by finite element analysis across the width of the cast connector is consistent with the measured distribution. 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.

8 FULL-SCALE VALIDATION TESTING

Subsequent to the initial short-brace testing, a full-scale connector-brace assembly was tested in a 12,000 kN MTS testing frame. The brace assembly, fabricated using HSS 168x13 produced to ASTM A500 grade C, had an unbraced length of 6.120 metres, a nominal slenderness of approximately 110 and a nominal diameter-to-thickness ratio of 14.72. The setup for the full-scale test included boundary conditions that more closely simulated the gusset
configuration and out-of-plane stiffness that would be present in the field, as shown in Figure 11.

Cyclic inelastic testing was carried out according to the symmetric testing protocol outlined in Appendix T of the AISC Seismic Provisions (AISC 2005). The brace performed well (Figs 12-14) with the cast connectors easily outlasting the inelastic life of the CHS brace. The final fracture occurred at the midspan of the brace after the onset of local buckling.

Figure 11. (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 12. Cyclic inelastic testing of a full-scale HSS 168x13 brace assembly

Figure 13. Connector remained elastic during cyclic inelastic loading of brace-assembly; plastic hinge forms in gusset plate (left), final brace-connector assembly failure occurs in the CHS brace after the onset of local buckling in the brace (right)

Figure 14. Cyclic load-displacement response of a full-scale (6.120 m) HSS 168x13 assembly

9 CONCLUSIONS

This paper presents a novel method of connecting HSS bracing members in seismic-resistant concentrically braced frames. Laboratory results showed that the brace assemblies fabricated using cast steel connectors performed very well when subjected to both cyclic and tensile inelastic loading. Further laboratory validation of the connector concept is currently underway and is focused on confirming that the excellent performance of the connectors is also achieved in braces of typical lengths, under a variety of loading protocols, and with testing criteria that more accurately reproduces the boundary conditions expected in the field with respect to member
end rotations. A more detailed presentation of the development and testing of the connector concept is provided in de Oliveira et al. (2008).

Since the completion of this research, the seismic-resistant connector technology has been licensed to Cast ConneX Corporation, which distributes a line of seismic-resistant HSS connectors for use in braced frames across North America (Fig. 15). These innovative connectors may herald a new era of pre-engineered connection design and fabrication for arduously loaded HSS members.

Figure 15. Cast ConneX™ High-Strength Connectors

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