

Full-Scale Experimental Validation of Standardized Seismic-Resistant Cast Steel Brace Connectors

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ABSTRACT :

Although concentrically braced frames are an efficient and popular choice for the lateral force resisting systems of mid- to low-rise steel frames, their bracing connections may be prone to premature failure unless complex design and fabrication procedures are undertaken. Frames utilizing Hollow Structural Section (HSS) braces, slotted to receive a concentric welded gusset plate, are particularly susceptible to connection fracture due to stress concentrations arising from the shear lag phenomenon at the net-section.

To address this issue, a cast steel connector was developed at the University of Toronto that provides a bolted connection between a circular hollow section (CHS) brace and a typical corner gusset plate. Unlike standard, fabricated, slotted-tube type connections, the cast connector is designed such that shear lag in the brace member is virtually eliminated. Thus, there is no need for cumbersome reinforcing plates in the connection region. The connector was designed using solid modeling software and finite element analysis, and cast using ASTM A958 grade steel, which is very similar to most wrought grades.

This paper presents the results of a series of full-scale brace tests in which four moderately slender, compact, cold-formed CHS brace assemblies were cyclically loaded to failure. Each of the brace assemblies employed a different size of CHS and cast steel connector, with the latter in turn bolted to a typical corner gusset plate. Each of the tests followed the buckling restrained brace protocol specified in the American Institute of Steel Construction (AISC) Seismic Provisions. All of the braces survived several cycles beyond the protocol and failed in tension at the center of the brace. In each test the cast steel connectors showed no sign of yielding, illustrating that the use of a cast steel connector is a viable means of connecting CHS brace members for seismic applications.

KEYWORDS: Ductile concentrically braced frames, seismic resistant connections, hollow structural section bracing, cast steel

1. INTRODUCTION

In North America, concentrically braced frames (CBFs) are a popular choice for the lateral force resisting systems of low- to mid-rise buildings due to their relatively simple design, fabrication, and erection. Hollow structural section (HSS) members are a particularly popular choice for the bracing elements in these frames. Laboratory tests and post-earthquake investigation (AIJ 1995, Tremblay et al. 1996, Bonneville and Bartoletti 1996, Yang and Mahin 2005, Fell et al. 2006) have shown that slotted HSS-to-plate brace end connections are prone to fracture during the cyclic tensile yielding and inelastic buckling that a brace may experience during a seismic event. The failure of such a connection is typically a result of the concentration of inelastic deformations at the net-section of the connection region due to shear lag. To address this undesirable failure

mode, the net-section of the connection can be increased by welding reinforcing plates to the outside of the brace member over the length of the slotted region of the HSS (Yang and Mahin 2005, Haddad and Tremblay 2006).

The reinforced net-section connection detail has resulted in an industry trend to use RHS (rectangular hollow section) braces rather than CHS braces because reinforcing plates are easier to install on the flat surfaces of RHS. Unfortunately, laboratory tests (Lee and Goel 1987, Shaback and Brown 2003, Tremblay et al. 2003, Yang and Mahin 2005) have shown that North American cold-formed RHS braces may not have the ductility, under cyclic inelastic loading, that is assumed in the design of ductile concentrically braced frames. When compared to RHS, CHS have lower residual stresses because of differences in the way that they are manufactured. This means that CHS braces may outperform RHS braces in terms of their inelastic ductility; however, reinforced slotted end connections are more difficult to fabricate for CHS than for RHS members.

In response to the need for a reliable and simple solution to detailing and fabricating CHS seismic brace connections, de Oliveira et al. (2008) developed a standardized cast steel connector that effectively eliminates both the shear lag and net-section issues of conventional, fabricated, slotted HSS bracing connections (Figure 1).

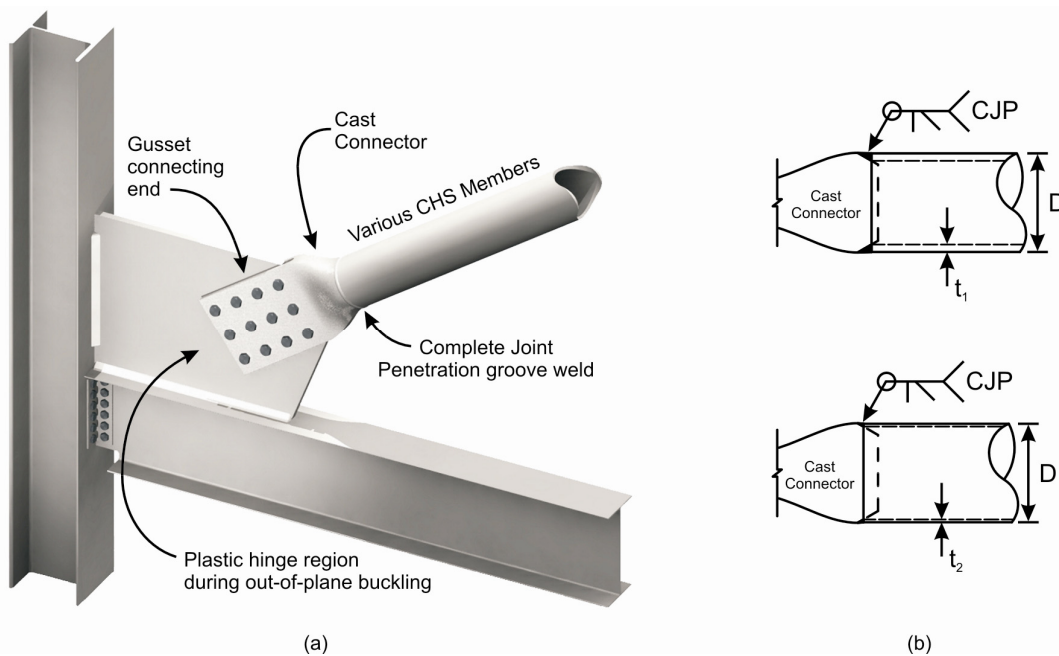


Figure 1 Standardized cast steel CHS brace connector: (a) brace-connector assembly in frame; (b) beveled nose detail that accommodates CHS sections of different wall thickness

2. PROTOTYPE DESIGN AND PROOF-OF-CONCEPT TESTING

2.1. Design Philosophy

The objective in designing a cast steel connector for CHS seismic braces was to create a component that could be shop welded to the end of a brace member and that would subsequently accommodate bolting (or welding) of the connector-brace assembly to a typical corner gusset plate connected to the beam-column intersection. The connector capacity exceeds the expected yield strength of the brace, $R_y F_y A$, by means of a complete joint penetration (CJP) weld between the end of the connector and the CHS brace segment. As illustrated in Figure 1, the CHS connecting end has a conical shape that protrudes into the end of the CHS brace. The bevel angle was specifically chosen so that when the connector's nose is inserted into the end of a square-cut CHS brace, a 60° groove is formed between the nose and the end of the CHS member. Welding between the CHS and the

connector can then be carried out according to a qualified procedure, with the connector-brace being mounted on a turning roll to simplify the welding operation. Since this detail allows for the connection of the brace's entire cross section, the weld, if properly applied, can accommodate large cyclic inelastic brace deformations without any risk of fracture. Another benefit of the beveled nose detail is that it enables a single connector design to be used for any brace member of a given outer diameter. For example, a thin walled tube will slide further up the nose of the casting than would a thick walled tube, but the groove angle always remains at 60° and a full penetration weld can still be achieved (Figure 1b).

The gusset-receiving end of the connector was designed to ensure that when the axial force is transferred from the brace member through the cast connector to the gusset plate, the force remains concentric to the work point of the frame. This is achieved by providing connecting plates that can slide onto either side of the corner gusset. The connecting plates are large enough to accommodate a slip-critical bolted connection that can carry the expected yield strength of the brace for the heaviest commonly available CHS that fits the conical nose of the connector. However, for thinner walled braces, or if only a bearing-type connection is required, a smaller and more economical bolt arrangement can be used. In the case where fewer bolts are employed, the gusset-receiving end can be trimmed to the required length. Finally, if field welding is required to provide fit-up tolerance or for building plumbing, four longitudinal fillet welds along the sides of the casting's plates can be used to connect the connector to the gusset plate.

The connector is intended to be used with a well-detailed gusset plate that incorporates an allowance for the inelastic rotations associated with brace out-of-plane post-buckling deformations. This allowance can be achieved by leaving a free unsupported region of the plate that will accommodate the formation of a plastic hinge. It has been suggested by others that this region can be linear, having a width of at least twice the gusset plate thickness (Astaneh-Asl 1998), or of an elliptical shape (Yoo et al. 2008).

All of the aforementioned characteristics are intended to satisfy the AISC seismic requirements for Special Concentrically Braced Frames (AISC 2005) and the CSA-S16 seismic provisions for Moderately Ductile (Type MD) Braced Frames and Braced Steel Frames with Limited Ductility (Type LD) (CSA 2005).

2.1. Proof-of-Concept Testing

The viability of the connector concept described above was established through both static and cyclic brace tests conducted at the University of Toronto. The initial prototype, shown in Figure 2, was designed to receive a CHS brace member having an outer diameter of 168 mm. Two cyclic tests were conducted on short brace-connector assemblies fabricated using HSS 168x9.5 and HSS 168x6.4 sections. The quasi-static cyclic testing was carried out according to a loading protocol developed specifically to test the connectors' ability to transmit repeated cycles of tensile and compressive inelastic loading of the short brace-assemblies. As the brace-assemblies were stocky, the forces transmitted during compressive excursions were conservatively high. A monotonic tension test was also conducted on a brace assembly fabricated with an HSS168x13 section, the heaviest available brace of that diameter. All of the CHS used in these tests were dual certified to ASTM A500 Grades B and C (ASTM 2007). In all cases, the connectors remained fully elastic during the destructive testing of the brace assemblies. An in-depth presentation of the development and testing of the connector concept is provided in de Oliveira et al. (2008).

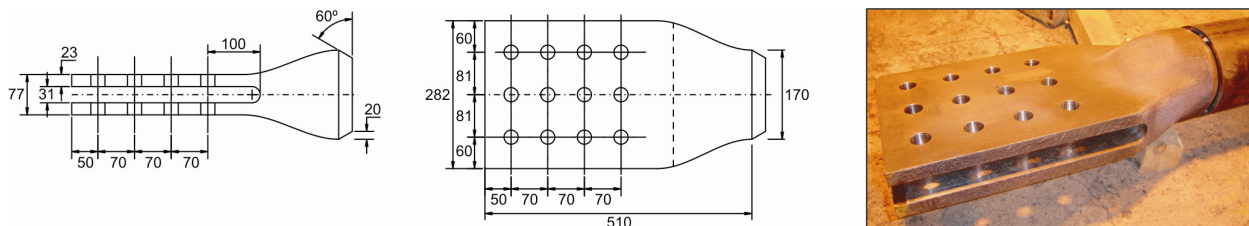


Figure 2 Prototype cast connector for 168 mm outer diameter CHS

3. FULL-SCALE TESTING

Having validated the connector concept, the technology was licensed to a corporate entity which then developed a line of four connectors to fit CHS bracing having outer diameters of 102, 141, 168, and 219 mm. In a collaborative study between researchers at the University of Toronto, École Polytechnique de Montréal, and the Cast Connex Corporation, a cyclic test program was subsequently conducted on full-scale brace specimens equipped with the cast steel connectors. The purpose of these tests was to validate the performance of the brace connectors when used in bracing of typical lengths and with boundary conditions representative of typical field conditions.

The brace-connector specimens were fabricated using the heaviest walled circular tubing produced to ASTM A500 Grade C that is available in North America for each connector. CHS produced to ASTM A500 Grade C has the following minimum specifications: a yield strength of 317 MPa, an ultimate strength of 427 MPa, and an elongation of 21% in 50 mm. Pretensioned ASTM A490 bolts were used in the bolted connections between the gusset plates and the cast connectors. For each specimen, the minimum number and diameter of bolts that would provide a connection strength commensurate with the expected yield strength of the brace, $R_y F_y A$, was used. As specified in the AISC seismic provisions, the expected yield strength was computed using an overstrength factor, R_y , of 1.4.

The laboratory specimens had the specified geometrical properties shown in Table 3.1. In the table, L_u is the unbraced length of the brace and L_{CHS} is the length of circular HSS used in fabricating the brace test specimen. The sectional properties of the CHS brace sections, t_{des} (design wall thickness; equal to 0.93 times the nominal wall thickness of the section), A (gross cross-sectional area; calculated using the nominal diameter and t_{des}), and r (radius of gyration; also determined using the nominal diameter and t_{des}) are also provided in the table. The thickness and width of the gusset plate in the free length beyond the ends of the connectors are reported as t_g and b_g , respectively. Finally, the sectional and overall slenderness ratios (D/t_{des} and L_u/r) of the brace assemblies are also tabulated.

Table 3.1 Specified Geometrical properties of the test specimens

HSS Designation	L_u [mm]	L_{CHS} [mm]	t_{des} [mm]	A [mm ²]	r [mm]	$t_g \times b_g$ [mm]	D/t_{des} [-]	L_u/r [-]
HSS 102x8.0	4293	3503	7.39	2190	33.5	13 x 318	13.8	128
HSS 141x9.5	6617	5564	8.86	3690	47.0	19 x 330	16.0	141
HSS 168x13	6147	5004	11.81	5810	55.4	25 x 391	14.3	111
HSS 219x16	6160	4650	14.76	9480	72.4	32 x 511	14.8	85

3.1. Laboratory Setup

The full-scale tests were conducted on single brace-connector assemblies at the Hydro-Québec Structural Engineering Laboratory at École Polytechnique de Montréal. A 12 MN MTS load frame, with a high performance double-acting, double-ended structural actuator having a total stroke of 500 mm, was used for the tests. Specially designed grips for the load frame were used to emulate the out-of-plane stiffness of the gusset that connects to the beam-column intersection in a typical concentrically braced frame, as illustrated in Figure 3.

The applied load was monitored using the actuator's load cell while two cable position transducers, measuring deformations over the length of the tubular sections, were used to control the axial deformations applied to each brace specimen. The transducers were mounted on opposite sides of each specimen, just beyond the ends of the HSS. Transducers were also used to measure the in- and out-of-plane deformation of the brace at its mid-length during testing.

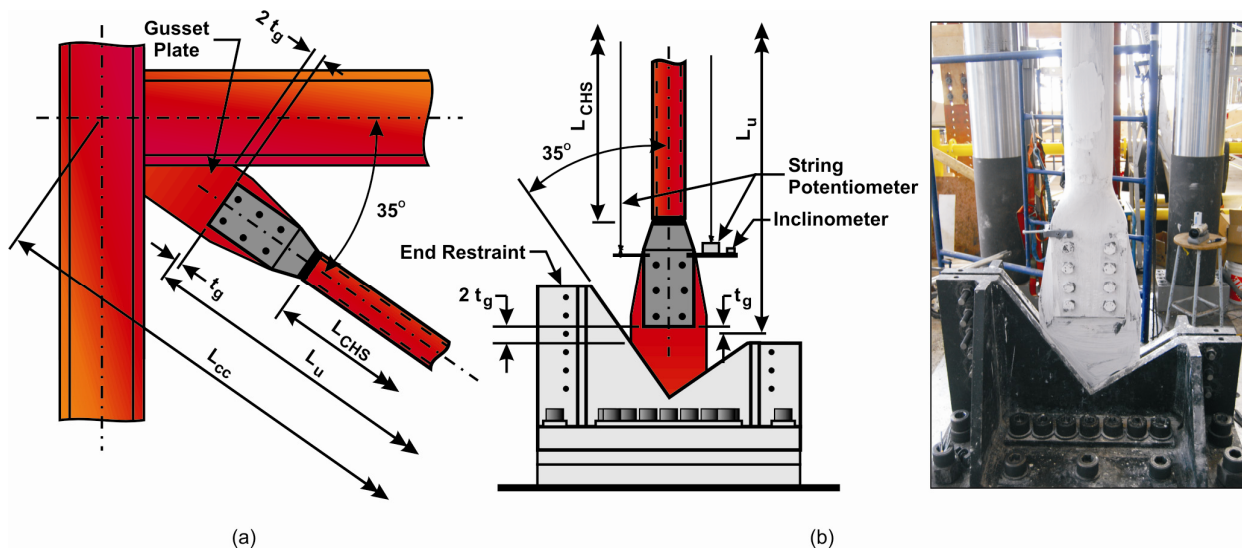


Figure 3 (a) Reference bracing configuration; (b) grip details

3.2. Testing Protocol

Quasi-static cyclic testing was carried out according to the qualifying test protocol proposed for buckling-restrained braces (BRBs) in the AISC seismic provisions. The protocol is based on the brace's expected deformation at yield, Δ_{by} , and the brace deformation associated with the design story drift, Δ_{bm} . For this testing, a design story drift of 2-percent was assumed as is recommended in ASCE 7-05 (ASCE 2005) for Other Structures of the Occupancy Category I or II, which represents the majority of steel building structures. Recent research studying the behavior and response of concentrically braced frames has shown that CBF testing protocols should impart brace deformations as large as those associated with a 4-percent drift level (Fell et al. 2006). This was achieved during the full-scale testing as the AISC BRB protocol prescribes two full compression-tension cycles at the brace deformation associated with twice the design story drift, which in this case, as noted above, was set at 2-percent.

The test program was such that a cumulative inelastic strain demand of 200 times the brace's expected deformation at yield was achieved by the time the protocol had been followed up to the 4-percent drift level. In all four tests, the brace assemblies had not fractured upon the completion of the protocol, thus additional cycles were applied at increasing story drift increments from 4-percent drift until failure occurred. For all of the tests, the compressive excursion to the brace deformation associated with the desired drift level preceded the tensile excursion in any given cycle.

3.3. Test Results

All four of the assemblies behaved as would be expected of ductile CHS buckling braces during the cyclic quasi-static testing. The first low amplitude cycles showed only minor inelasticity in the system; elastic brace buckling was observed early in the protocol in all cases. As the connections had not been designed to be slip-critical, bolt slip was noted in some of the tests. In these instances, bolt slip only occurred once during a tensile excursion as the slip load of the pretensioned bolts was never exceeded in compression in any test. Subsequent larger compressive excursions resulted in out-of-plane inelastic buckling of the bracing element (Figure 4b), with significant tensile yielding occurring over the majority of the CHS 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 4c). At higher amplitude compressive excursions, a discrete plastic hinge formed at the mid-length of the brace during

inelastic buckling (Figure 4d) which eventually resulted in local buckling of the CHS wall.

As previously noted, all four of the brace assemblies survived the full BRB protocol. This result was expected, as the braces tested were of intermediate slenderness (L_u/r between 85 and 141) and were quite compact (D/t between 13.8 and 16.0) in comparison to the code-prescribed diameter-to-thickness limits. In all cases, the eventual failure of the brace occurred at the mid-length of the CHS element during a tensile excursion that followed the onset of significant local buckling of the CHS. Local buckling was demonstrated first through an ovalization of the cross section. At larger compressive deformations, the ovalization was followed by the formation of a crescent-shaped snap-through local buckle in the compressive face of the CHS segment. In all cases, failure of the brace occurred in the tensile excursion immediately following the formation of the crescent-shaped local buckle (Figure 4e).

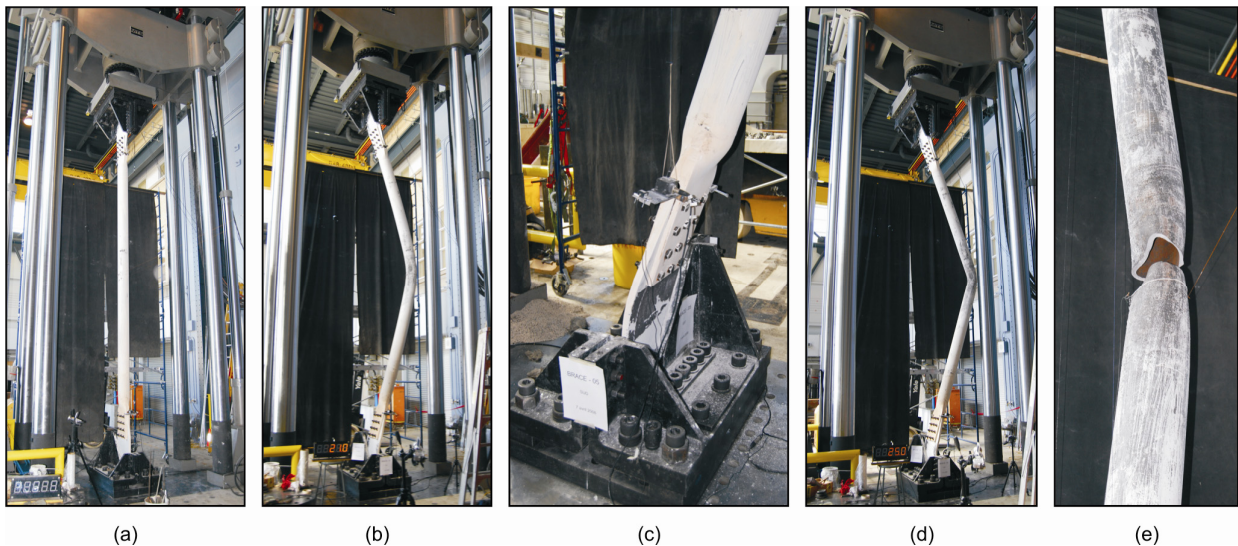


Figure 4 HSS 219x16 brace-connector assembly: (a) undeformed; (b) experiencing inelastic buckling; (c) plastic hinge formation in free length of gusset plate; (d) localization of plastic hinge in CHS member; (e) fracture of brace at mid-length after the formation of a crescent-shaped snap-through local buckle

Figure 5 shows the hysteretic response of each of the brace-connector assemblies tested. In each of the plots, the force transmitted axially through the brace is plotted against the imparted axial strain in the CHS, which is given by Δ/L_{CHS} . A vertical dashed line shows the axial strain corresponding to 4-percent drift and thus the completion of the BRB protocol as prescribed by AISC. Note that three of the four braces were capable of transmitting force at axial strains which significantly exceeded the 4-percent requirement. Also shown in the plots are the tensile strength, $F_y A$, and the expected yield strength, $R_y F_y A$, as calculated using the design sectional properties, the minimum specified yield strength, and the code prescribed overstrength factor of 1.4 for ASTM A500 Grade C material. It is interesting to note that the loads that were developed during some of the tests were considerably higher than the aforementioned design criteria, particularly for the smaller diameter CHS sections. This is likely a direct result of the more severe cold-working involved in manufacturing thick-walled CHS having a tight radius. Despite this overstrength, the cast connectors remained fully elastic and were completely intact after every test.

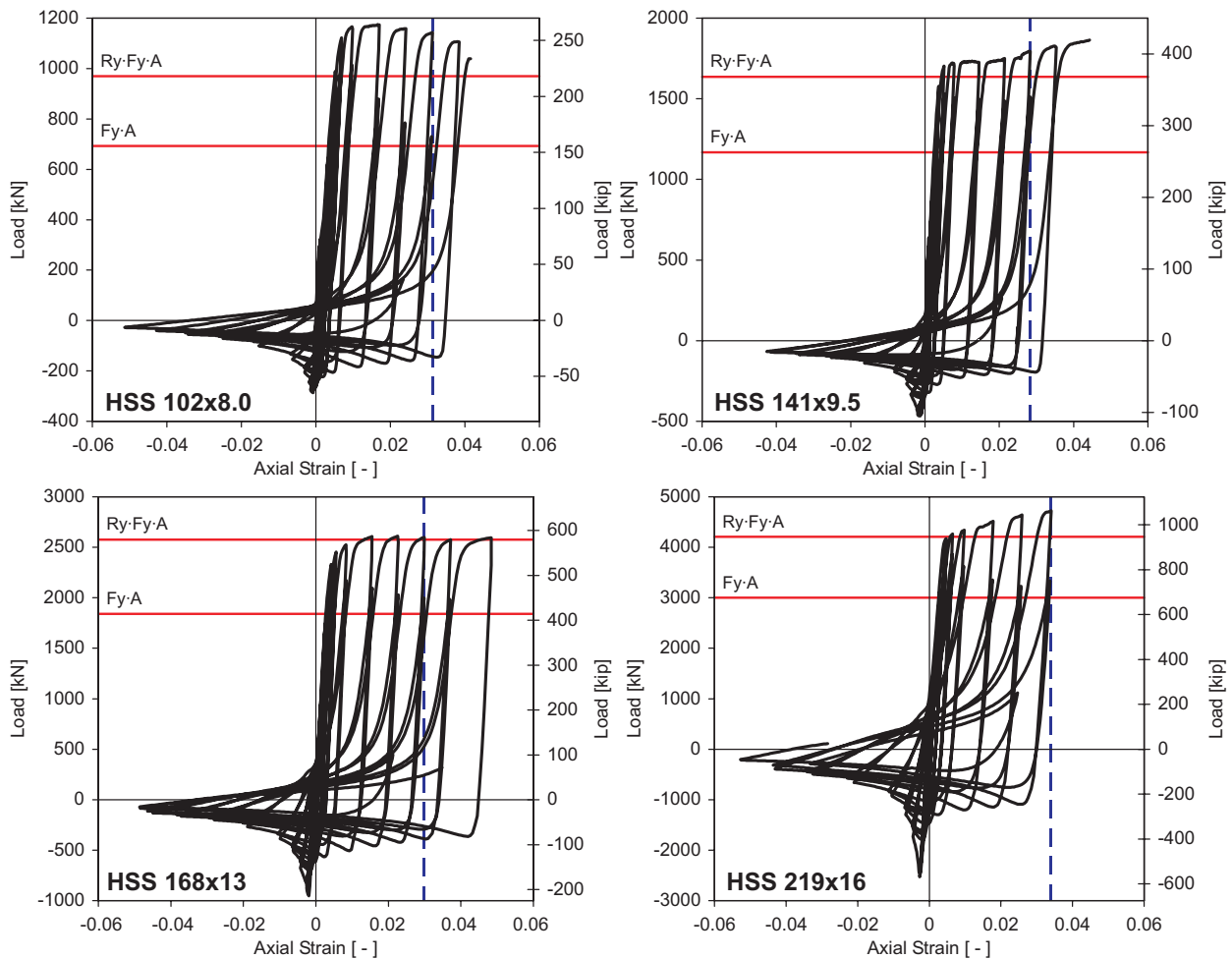


Figure 5 Hysteretic responses of the brace assemblies equipped with standardized cast connectors

4. CONCLUSIONS

The laboratory results showed that the circular hollow section brace assemblies equipped with the cast steel connectors performed very well when subjected to cyclic inelastic loading, meeting the AISC-prescribed BRB seismic loading protocol in all cases. This testing, in conjunction with the proof-of-concept testing that was carried out previously at the University of Toronto, confirms that the standardized cast steel connectors meet the requirements for ductile seismic bracing connections for braces of typical lengths, under a variety of loading protocols, and with boundary conditions representative of typical field conditions.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the research and commercialization funding and support of the Ontario Centres of Excellence (OCE), the financial support of the Natural Sciences and Engineering Research Council of Canada (NSERC), and Simon Trudel Languedoc's contribution to the experimental work.

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