INELASTIC CYCLIC TESTING OF LARGE SIZE STEEL BRACING MEMBERS

R. Tremblay[1] M. Haddad[2], G. Martinez[3], J. Richard[4], and K. Moffatt[5]

[1] *Professor,* [3]*Post-Doctoral Fellow,* [4]*Graduate Research Assistant, Dept. of Civil, Geological and Mining Engineering, Ecole Polytechnique of Montreal, Montreal, Canada*
[2] *Assistant Professor, Dept. of Civil Engineering, Jerash Private University, Jerash, Jordan*
[5]*Senior Design Engineer, Cima+, Laval, Canada*
Email: robert.tremblay@polymtl.ca

ABSTRACT :

This paper outlines a comprehensive experimental program that has been initiated to examine the cyclic inelastic response of large size steel bracing members. Cyclic quasi-static tests were carried out on 34 brace specimens, including square tubing, circular tubing, and W (H) shape members. The brace slenderness, the slenderness ratio of the brace cross-section elements, and the loading protocol were also varied. The depth of the brace cross-sections ranged between 152 and 305 mm and the tensile yield capacity of the braces varied from 2.0 to 9.0 MN. The response of the brace specimens was characterized by overall buckling, tensile yielding, and local buckling. The latter developed in the plastic hinge that formed at the brace mid-length. Failure of the brace occurred by fracture in the plastic hinge region. The shape of the brace cross-section, the slenderness of the elements of the brace cross-section and the brace overall slenderness were found to influence the fracture life of the brace specimens.

KEYWORDS: Bracing member, Fracture, Global buckling, Local buckling, Slenderness

1. INTRODUCTION

Cyclic quasi-static tests have been conducted in the past three decades to characterize the inelastic seismic performance of steel bracing members made of rectangular or circular hollow structural shapes (HSS) (Lee and Goel 1987; Liu and Goel 1988; Tang and Goel 1989; Walpole 1996; Shaback and Brown 2003; Tremblay et al. 2003; Elchalakani et al. 2004; Goggins et al. 2005; Uriz 2005; Yang and Mahin 2005; Fell et al. 2006, Han et al. 2007). These tests revealed that local buckling eventually occurs in the plastic hinge that forms at the brace mid-length upon inelastic buckling of the braces in compression. The high localized strain demand that results from local buckling eventually leads to fracture of the brace. It was shown that delaying local buckling by limiting the slenderness of the brace cross-section elements can enhance the fracture life of bracing members. More slender braces were also found to generally exhibit longer fracture life than stockier braces. Special detailing requirements that reflect these observations have been included in the seismic provisions of the CSA-S16 Standard for the design of steel structures in Canada (CSA 2001). This test data has also been used to develop empirical models to predict the fracture life of rectangular HSS bracing members (Lee and Goel 1987; Tang and Goel 1989; Hassan and Goel 1991; Tremblay 2002, Tremblay et al. 2003). Uriz (2005) implemented a low-cycle fatigue model to account for fracture in braces modeled with a fiber discretization of the cross-section. Fell et al. (2006) used a void growth fracture model in finite element analysis to predict occurrence of fracture, including strain triaxiality effects and local strain amplification due to local buckling.

Past tests on HSS braces have been performed on small brace specimens that had maximum cross section dimension of 165 mm or less. In practice, HSS members up to 305 mm in size are commonly used in braced steel frame construction but no physical test data is available to support current code provisions or predictive analytical models for such large bracing members. In particular, braces with large cross-section dimensions have a low slenderness ratio and, hence, are potentially more prone to premature fracture under severe inelastic seismic demand than the smaller braces for which test data exists. This situation was the motivation for initiating an extensive test program on large size braces at Ecole Polytechnique of Montreal. Testing has been performed on HSS bracing members with cross-section dimensions varying from 152 mm to 305 mm. Emphasis was put on rectangular hollow sections (RHS) braces as this shape is among the most widely used for bracing members. Past research also showed that this shape is the most susceptible to premature fracture. Circular hollow section (CHS) and W-shaped braces of similar size were also investigated for comparison purposes. This paper describes this test program and presents preliminary results that illustrate the influence of key parameters such as the type of cross-section, the brace slenderness and the slenderness of the brace cross-section elements.

2. TEST PROGRAM

2.1. Selection and Design of Test Specimens

The test program included a total of 34 specimens: 19 RHS braces, 9 CHS braces, and 6 W-shaped braces. The RHS and CHS members were cold-formed structural tubing fabricated according to ASTM A500, grade C. For the RHS members, the specified minimum yield stress, F_y, was equal to 345 MPa and the specified minimum ultimate tensile strength, F_u, was 427 MPa. The corresponding values for the CHS braces were F_y = 317 MPa and F_u = 427 MPa. The W profiles were made of ASTM A992 steel (F_y = 345 MPa, F_u = 448 MPa).

Various bracing configurations and dimensions encountered in building applications were surveyed to determine a representative bracing arrangement for the test specimens (Fig. 1a). Of all cases, the average brace inclination with respect to the horizontal was 35° and this angle was adopted for the reference prototype bracing (Fig. 1b). Figure 1b also shows the single tapered gusset plate design that was selected for the study. The gusset was assumed to be welded to the beams and columns and detailed to accommodate inelastic rotation associated with brace buckling.

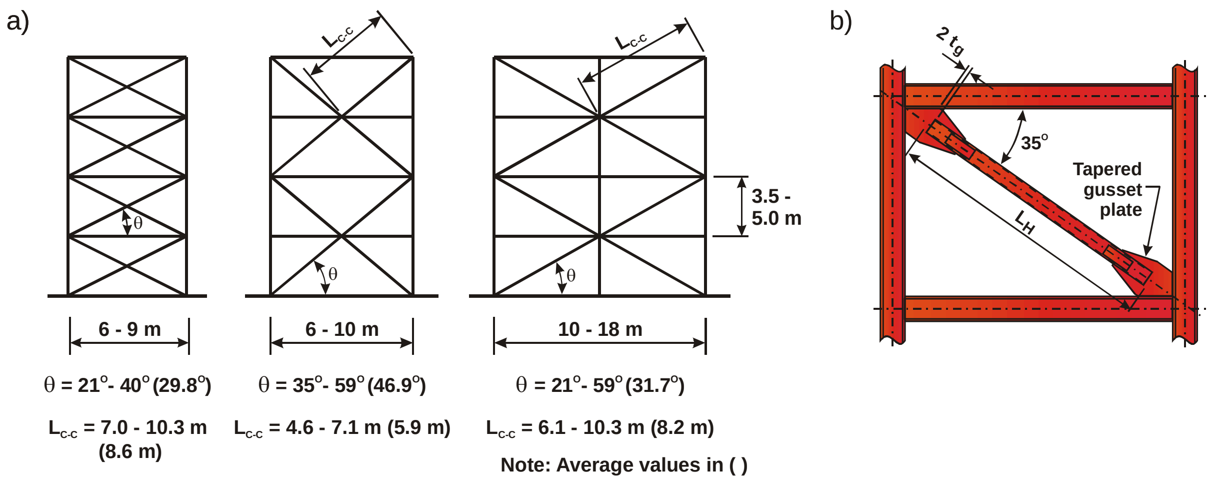

Figure 1 Bracing configurations: a) Cases surveyed; b) Selected geometry.

The survey of the sample buildings revealed that the brace length between working points, L_{c-c}, would typically vary between 4.6 and 10.3 m, with a mean value of 7.7 m. The brace specimens were therefore selected among all possible brace designs from available shapes in North America that can develop a design axial compression strength (ϕP_n) ranging between 1000 and 5000 kN for brace effective lengths, KL, equal to 5, 6, 7, 8, and 9 m. Figure 2a shows the compressive strength and unit weight of the possible design solutions for each type of cross-section. The shapes that were selected for the test program are also indicated on the graphs.

For the RHS braces, square tubes with a cross-section width, *b*, of 152, 254, and 305 mm were chosen. The selected CHS members had an outside diameter, *D*, equal to 273 mm. The wall thickness, *t*, of the RHS and CHS specimens was varied to obtain b_0/t ratios for the brace cross sections that were greater than, equal to, and lower than the code prescribed limits for ductile bracing members. Note that $b_0 = b - 4t$ for rectangular tubes and $b_0 = D$ for circular tubes. In the CSA-S16 seismic provisions, these b_0/t limits are respectively equal to 17.6 and 28.6 for the RHS and CHS members. Figure 2b shows the distribution of the chosen test specimens with respect to these code limits. As also illustrated in Fig. 2b, the length of the HSS specimens was adjusted to obtain overall brace slenderness ratios, KL/r, equal to 40 and 60. These values cover the lower end of the KL/r values exhibited by the possible brace designs. Since stockier braces are likely to possess a shorter fracture life when subjected to inelastic demand from earthquakes, this selection represented the most critical scenario for assessing the potential for premature brace fracture in actual structures. Five different W-shapes were chosen to examine the influence of b_0/t and KL/r ratios: W360x134, W310x129, W310x97, W310x86, and W250x101. For these profiles, b_0/t is determined for the flanges (b_0 is half the flange width) and the CSA-S16 limit is equal to 7.8. The b_0/t limits in the AISC seismic provisions (AISC 2005) are comparable to, although slightly lower than those prescribed in CSA-S16: 15.4 for RHS, 27.8 for CHS, and 7.2 for W shapes.

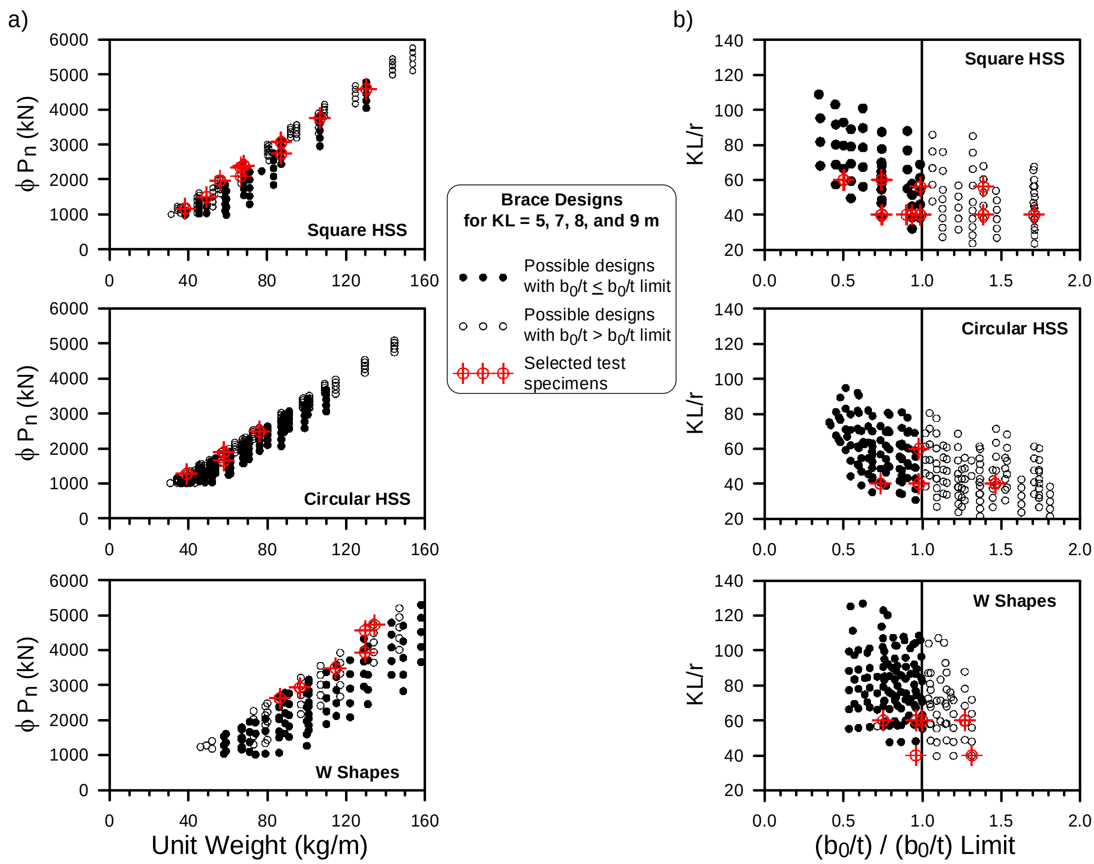

Figure 2 Selection of brace specimens among possible brace designs: a) Brace capacities and unit weight; b) Brace KL/r and b_0/t ratios.

Figure 3a shows the brace configurations and connection details that were considered. The connections were designed to resist a tension force equal to $R_yF_yA_g$ and a compression force equal to $1.1R_yP_n$, as prescribed in the AISC seismic provisions. In these expressions, R_y is the ratio of the expected yield stress to the specified minimum yield stress for the braces (R_y = 1.4 for HSS braces and 1.1 for W shapes), A_g is the gross area of the brace cross-sections, and P_n is the nominal compressive strength of the braces. Out-of-plane buckling was assumed for all three shapes. For this condition, inelastic end rotation upon buckling is permitted in the hinge created in the gusset plate by leaving a free length equal to twice the gusset plate thickness, t_g, at the end of the bracing member. For RHS braces, an in-plane buckling design was also examined for two specimens by inserting a knife plate between the gusset plate and the brace. In this detail, rotation is expected to develop in the free distance left in the knife plate. A slotted connection was adopted for all tubular braces. For these connections, the net section at the end of the slots had to be reinforced with cover plates to avoid fracture at this critical location. Details of the design of the cover plates can be found in Haddad and Tremblay (2006). For the W-shapes, connecting flange and web plates were used to ensure uniform stress distribution in the brace cross-sections. Five of the HSS specimens were fabricated without net section reinforcement: three without any special detailing to verify the assumptions made in design, and two with an enhanced slotted detail. Three HSS specimens were provided with a special plastic hinge detail at their mid-length. This hinge design aimed at developing ductile response, without local buckling, under reversed cyclic loading.

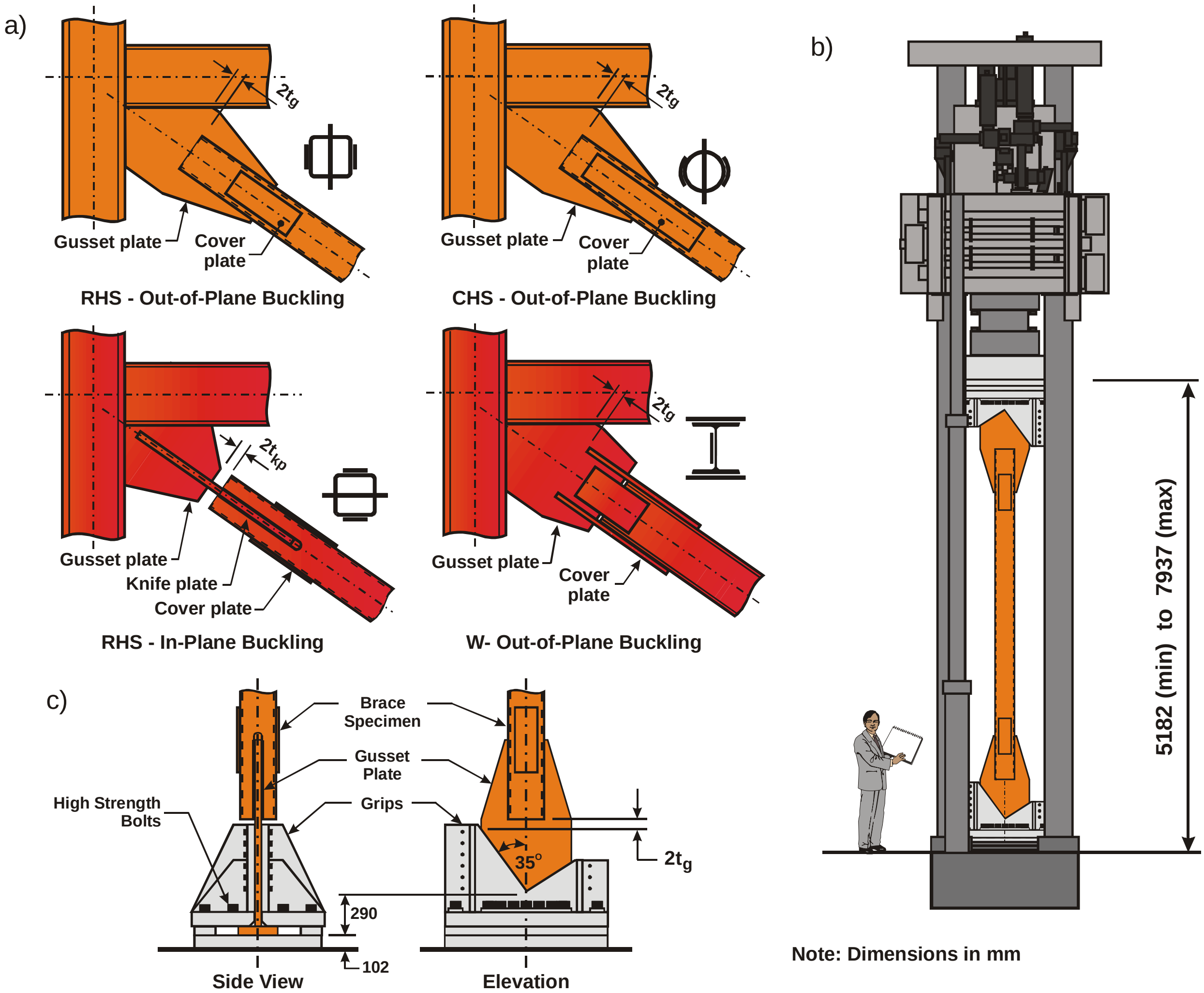

Figure 3 a) End connection details for the different brace types and buckling modes considered; b) Brace specimen in the 12 MN load frame; and c) End connection and attachment grips in the test setup.

2.2. Test Setup and Loading Protocol

The brace specimens were subjected to cyclic quasi-static axial tests in a 12 MN capacity load frame (Fig. 3b). The load frame is equipped with a double acting, double ended actuator mounted on a crosshead. The elevation of the crosshead can be adjusted to accommodate specimens up to 8 m in length. A load cell and an upper platen are attached in series to the moveable (lower) end of the actuator. The brace specimens were shop fabricated with gusset plate assemblies at both of their ends. In the laboratory, the gusset plates were inserted in grips that were specially designed and detailed to reproduce the assumed beam-column-brace joint geometry (Fig. 3c). The grips were secured to the upper and lower platens of the load frame by means of pre-tensioned high strength bolts. As shown in Fig. 3c, the gusset plates were terminated with a T-stub that was designed to transfer the axial tension and compression forces applied to the test specimens through direct bearing, without any slippage.

Two displacement protocols were used in the test program. Protocol No. 1 is illustrated in Fig. 4. It is a symmetrical displacement history with stepwise incremented amplitudes that simulates the demand imposed by earthquakes at distance. Up to the 20th cycle, the signal reproduces the median demand that is anticipated for moderately ductile (Type MD) braced steel frames designed according to CSA-S16 when subjected to earthquake ground motions compatible with the 2% in 50 years seismic hazard level in Vancouver, BC (Izvernari et al. 2007). The largest interstorey drift, Δ/h_s, in this first sequence is 1.5%. Two smaller amplitude cycles are then applied to assess the residual brace strength and stiffness at that point. Those two cycles are followed by cycles of increasing amplitudes to reproduce the demand expected in higher seismic zones such as the west coat of the U.S. (Fell et al. 2006). The loading protocol was developed in terms of interstorey drift. For testing, it was transformed into axial brace displacements, δ, assuming that all the deformations take place over the brace length comprised between the end hinges, L_H (see Fig. 1b):

$$\frac{\delta}{L_H} = 1.3\left(\frac{\Delta}{h_s}\right) \cos\theta \, \sin\theta \qquad (2.1)$$

In this expression, θ is the brace inclination angle (35°). The factor 1.3 is the ratio of the length L_H and the length $L_{c\text{-}c}$ for typical braced frame designs. This transformation is conservative for the small amplitude cycles associated to elastic response but it represents more closely the actual demand in the inelastic range. In the tests, the imposed displacement was controlled by means of two cable position transducers spanning over the length L_H on each side of the specimens. A second protocol representative of near field seismic events was considered for a subset of specimens. This second protocol is not presented herein as only test results obtained with Protocol No. 1 are discussed later.

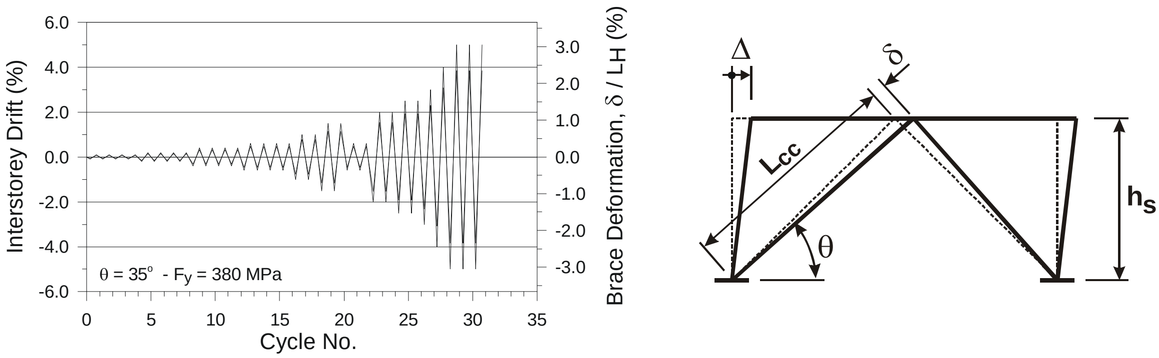

Figure 4 Test displacement protocol No. 1.

3. PRELIMINARY TEST RESULTS

The performance under Protocol No. 1 of brace specimens with end cover plates but without the special mid-length hinge detail is briefly discussed in this section. In these tests, all bracing members experienced several cycles of axial yielding in tension and global buckling in compression. Upon buckling, a plastic hinge formed at mid-length of the braces. Local buckling of the cross-section eventually developed in that hinge region, which resulted in amplified localized strain demand that led to brace fracture.

Figures 5a and 5b show the response of a CHS brace upon global buckling in compression. Figures 5c to 5e respectively illustrate the deformed shape after local buckling for each of the three different brace cross-sections. The observed deformed shape varies significantly depending on the type of cross-section. For square tubing, local buckling of the compression flange developed inward, inducing pronounced localized curvature and strain demand in the corner regions of the cross-section. In all RHS specimens, steel cracking and brace fracture initiated in this critical area. For CHS braces, inward local buckling developed in the portion of the cross-section that was subjected to the highest compressive stresses due to combined axial and flexural response upon brace buckling. This resulted in a gradual ovalization of the cross-section. Upon imposing higher axial deformations, the cross-section eventually locally flattened and a pronounced kink in the plastic hinge formed (Fig. 5d). High strain demand developed at both ends of that kinked profile, which triggered cracking of steel and, soon after, fracture of the brace. Global buckling of all W-shaped braces took place about their weak axis, resulting in local buckling of the two half-flange segments located on the side of the cross-section supporting the highest compression. These flange local buckles were smoother than in the HSS members, imposing lower strain demand, and the braces could withstand several additional cycles before initiation of cracking and brace fracture.

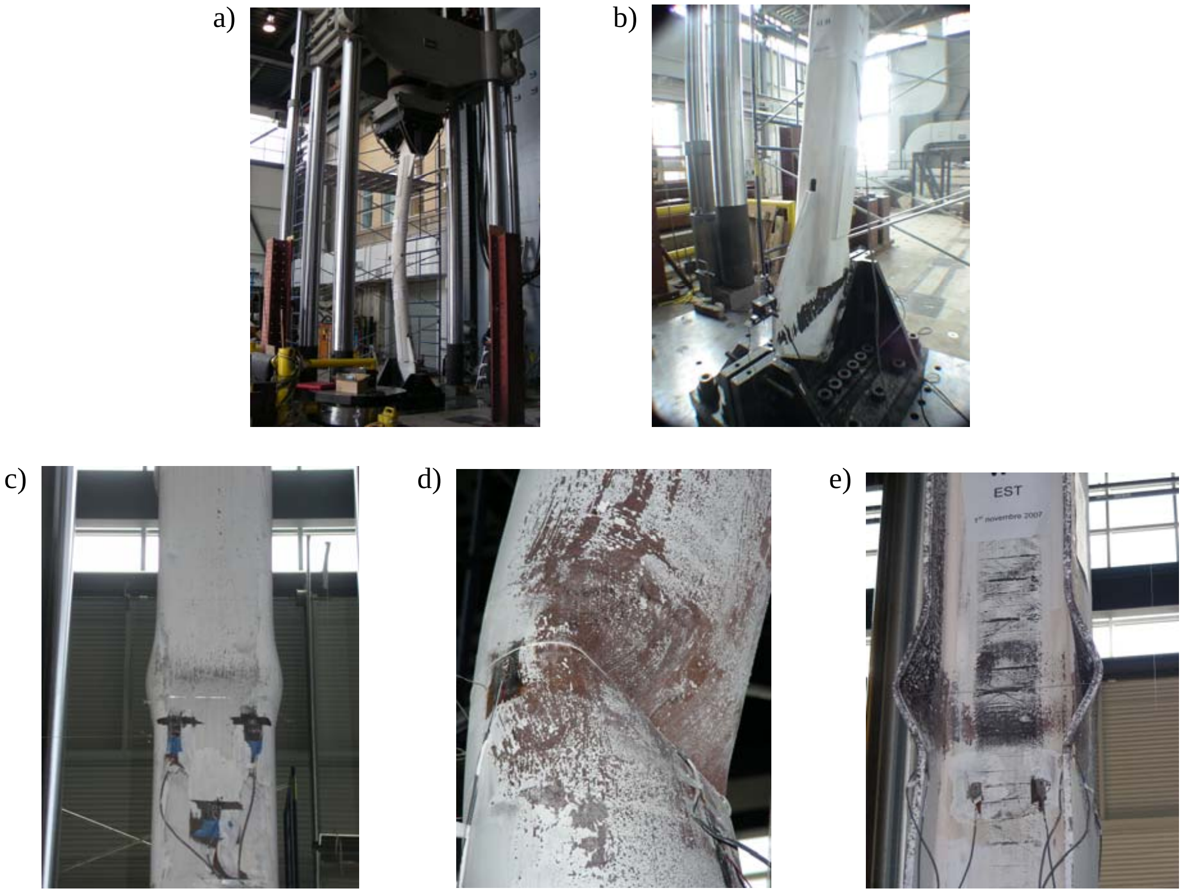

Figure 5 Observed brace behaviour: a) Overall buckling; b) Inelastic rotation in the gusset plate free length associated with buckling; c) Local buckling of a RHS brace specimen; d) Local buckling of a CHS brace specimen; and e) Local buckling of a W-shape brace specimen.

Figure 6 shows the hysteretic response obtained for 6 different brace specimens. The response of all HSS braces correlate well with the results of previous experimental studies: the braces could repetitively attain their full tensile yield strength while their compressive resistance gradually diminished under increasing cyclic compression deformation demand. Specimen RHS4 was an HSS254x254x13 that just met the CSA-S16 b_0/t limit. It had a global slenderness ratio of 40. Fracture of that brace took place at an interstorey drift of 1.0%, which is less than the median demand estimate from design earthquake level. Specimen RHS2 was identical to RHS4 except that the wall thickness was increased to reduce the b_0/t to 75% of the code limit. This modification permitted to withstand the two cycles at Δ/h_s = 1.5% before fracture. Specimen RHS19 had the same reduced b_0/t ratio but a larger slenderness (60). With this combination, fracture was delayed further to occur in the second cycle at Δ/h_s = 2.0%. Specimens CHS1 and CHS2 were made of HSS 273x9.5 circular tubing. The b_0/t ratio of these specimens was nearly equal to the code limit of 28.6 and only the overall slenderness was different. This time, the KL/r did not seem to affect the fracture life as both specimens failed in the second cycle corresponding to Δ/h_s = 1.5%, just meeting the median anticipated demand. These results also indicate that CHS and RHS braces with similar b_0/t and KL/r properties offer comparable fracture performance. Specimen W6 had a b_0/t ratio equal to 75% of the code limit and relatively higher global slenderness. In contrast with the HSS specimen with similar properties, that brace could withstand the entire displacement protocol, up to an interstorey drift of 5.0%, before fracture took place.

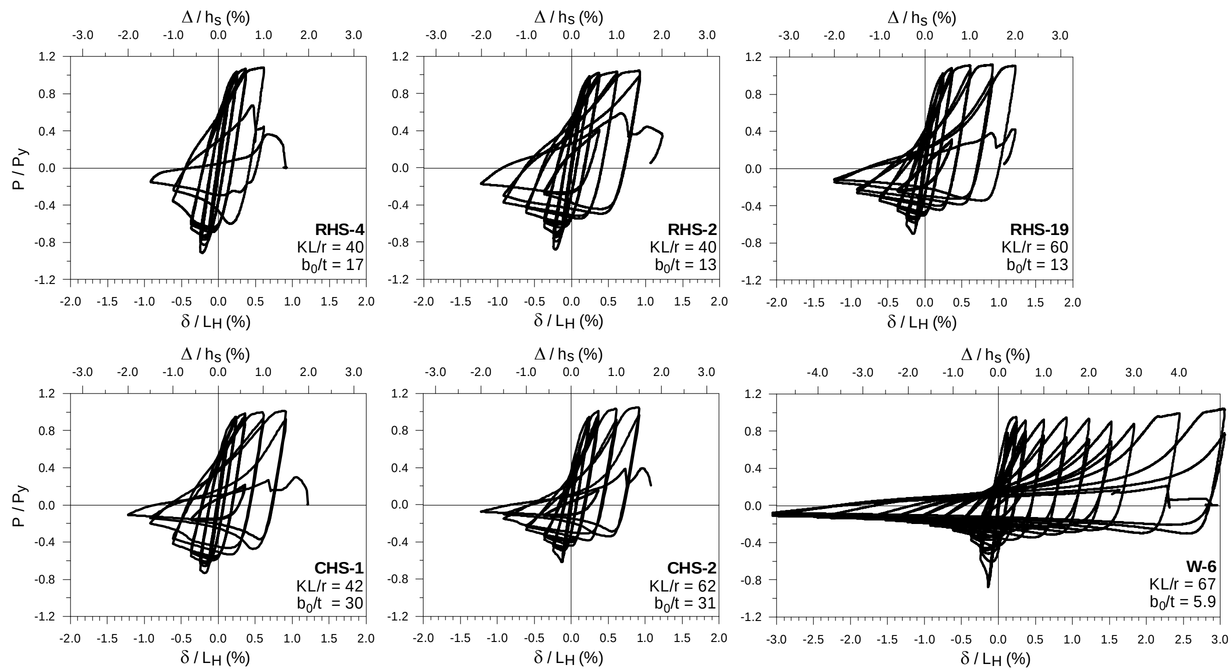

Figure 6 Typical brace hysteretic responses.

4. CONCLUSION

A test program has been recently completed to examine the performance of large size steel bracing members subjected to severe cyclic inelastic axial deformation demand. The tests showed that local buckling response and subsequent localized strain demand could vary significantly with the shape of the brace cross-section. Fracture of the HSS braces, either RHS or CHS, was found to develop rapidly after occurrence of local buckling. Local buckling and, thereby, fracture of the HSS braces were generally delayed when reducing the cross-section slenderness and increasing the brace overall slenderness.

Preliminary results indicate that stocky HSS braces that comply with current cross-section slenderness limits for ductile seismic response may not withstand the median demand that is anticipated for braced steel frames designed according to current Canadian codes. Further study is needed to assess the impact of the observed brace response on global structure response. Meanwhile, caution should be exercised when specifying stocky HSS braces for seismic applications. The test program showed that well proportioned W sections could represent a promising alternative to achieve ductile seismic performance.

ACKNOWLEDGMENTS

Financial support for this project has been provided by the Natural Sciences and Engineering Research Council of Canada, the Canadian Foundation for Innovation, and the American Institute of Steel Construction. Nucor and Atlas Tubes Inc. donated material for the test specimens. The specimens were graciously fabricated by the Canam Group. The authors wish to express their appreciation to the technical staff of the Hydro-Quebec Structural Engineering Laboratory at Ecole Polytechnique of Montreal.

REFERENCES

AISC. (2005). Seismic Provisions for Structural Steel Buildings, American Institute of Steel Construction Inc., Chicago, IL.

CSA. (2001). CAN/CSA-S16-01 Limit States Design of Steel Structures, including CSA-S16S1-05 Supplement No. 1. Canadian Standard Association, Toronto, ON.

Fell, B.V., Kanwinde, A.M., Deierlein, G.G., Myers, A.T., and Fu, X. (2006). Buckling and Fracture of Concentric Braces under Inelastic Cyclic Loading. Steel Tips. Structural Steel Education Council, Moraga, CA.

Goggins, J.M., B.M. Broderick, B.M., Elghazouli, A.Y., and Lucas, A.S. (2005). Experimental cyclic response of cold-formed hollow steel bracing members. *Eng. Struct.* , **27:7**, 977–989.

Haddad, M. and Tremblay, R. (2006). Influence of Connection Design on the Inelastic Seismic Response of HSS Steel Bracing Members. *Proc. of the 11th Int. Symp. and IIW Int. Conf. on Tubular Structures, ed. by J. Packer and S. Willibad*, Taylor & Francis, Leiden, The Netherlands, 639-646.

Han, S.-W., Kim, W.T., and Foutch, D.A. (2007). Seismic Behavior of HSS Bracing Members according to Width–Thickness Ratio under Symmetric Cyclic Loading. *ASCE J. of Struct. Eng.*, **133:2**, 264-273.

Hassan, O.F. and Goel, S.C. (1991). Modeling of Bracing Members and Seismic Behavior of Concentrically Braced Steel Structures. *Research Report UMCE 91-1*, Dept. of Civ. Eng., Univ. of Michigan, Ann Arbor, MI.

Izvernari, C., Lacerte, M., and Tremblay, R. (2007). Seismic Performance of Multi-Storey Concentrically Braced Steel Frames Designed According to the 2005 Canadian Seismic Provisions. Proc. 9th Canadian Conf. on Earthquake Eng., Ottawa, ON. Paper No. 1419.

Lee, S., and Goel, S. C. (1987). Seismic Behavior of Hollow and Concrete-Filled Square Tubular Bracing Members. *Rep. No. UMCEE 87-11*, Dept. of Civ. Eng., University of Michigan, Ann Arbor, MI.

Liu, Z., and Goel, S. C. (1988). Cyclic Load Behavior of Concrete-Filled Tubular Braces," ASCE J. Struct. Eng., **114:7**, 1488-1506.

Shaback, B., and Brown, T. (2003). Behaviour of Square Hollow Structural Steel Braces with End Connections under Reversed Cyclic Axial Loading. *Can. J. of Civil Eng.*, **30:4**, 745-753.

Tang, X., and Goel, S. C. (1989). Brace Fractures and Analysis of Phase I Structure. *ASCE J. Struct. Eng.*, **115:3**, 1960–1976.

Tremblay, R. Archambault, M.H., and Filiatrault, A. (2003). Seismic Performance of Concentrically Braced Steel Frames made with Rectangular Hollow Bracing Members. *ASCE J. of Struct. Eng.*, **129:12**, 1626-1636.

Tremblay, R. (2002). Inelastic Seismic Response of Steel Bracing Members. *J. of Const. Steel Research*, **58:5-8**, 665-701

Uriz, P. (2005). Towards Earthquake Resistant Design of Concentrically Braced Steel Structures. *Ph.D. Thesis*. Dept. of Civ. Eng., University of California, Berkeley, CA.

Yang, F., and Mahin, S.A. (2005). Limiting Net Section Fracture in Slotted Tube Braces. *Steel Tips Series*, Structural Steel Education Council, Moraga, CA.

Walpole, W. R. (1996). Behaviour of cold-formed steel RHS members under cyclic loading. *Research Rep. No. 96-4*, Univ. of Canterbury, Christchurch, New Zealand.