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INTRODUCTION

This issue of "Current Steel Structures Research" for the *Engineering Journal* focuses on a selection of research projects from two major Canadian universities. The descriptions will not discuss all of the current projects at the schools—there are simply too many. But selected studies provide a representative picture of the research work and demonstrate the importance of the schools to Canada, as well as the United States and indeed to the efforts of industry and the profession worldwide. But the close collaboration between institutions and individuals in the two countries has always been essential, and the results have impacted industry significantly.

The universities and their many researchers and graduate students are very well known in the world of steel construction: the University of Toronto in Ontario and the École Polytechnique in Montréal, Québec. The size of their respective civil engineering faculties—especially their structural engineering groups—is indicative of their leading roles in Canadian academic research. The studies presented in the following reflect elements of specific projects as well as other long-time efforts. As has been typical of American, European and worldwide engineering research projects for years, many of the projects are multiyear efforts, and a number of them are also multipartner efforts. This calls for very careful cooperation, planning and implementation, including the education of graduate students and advanced researchers. The outcomes of the projects focus on industry needs and incorporation of results into design standards.

The researchers in Toronto and Montréal have been active for many years, as evidenced by their leading roles in research and development across North America. They have also been frequent participants in the work in other countries. A large quantity of high-quality technical papers, reports, design guides and conference presentations have been published, contributing to a collection of studies that continue to offer solutions to complex problems for designers, fabricators and erectors.

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References are provided throughout the paper where they are available in the public domain. However, much of the work is still in progress, and in some cases reports or publications have not yet been prepared for public dissemination.

SOME CURRENT RESEARCH WORK AT UNIVERSITY OF TORONTO

Selected Projects of Professor Jeffrey A. Packer

Professor Packer has been at the University of Toronto since 1980 and is currently the Bahen/Tanenbaum Professor of Civil Engineering. He is one of the premier researchers in the world who addresses the broad area of tubular (HSS) members and structures. Thus, his research group performs experimental (small- to large-scale), numerical (nonlinear finite element analysis) and analytical research with primary focus on the behavior and design of steel structures. His current tubular structures research emphasizes welded, bolted, nailed and cast connections and joints. Loading conditions are key to many studies and have included quasi-static, fatigue, impact, blast and seismic conditions.

HSS under Impulsive Loading: The blast resistance of buildings and many infrastructure components has become an important design consideration. Embassies and other government buildings, petrochemical facilities and many other civilian structures are considering blast protection as a design feature. In fact, both the United States (ASCE, 2011) and Canada (CSA, 2012) have developed national blast design standards for civilian buildings. A design guide tailored to blast design of steel structures has recently been published by AISC (2013).

The direction from which blast loading is imposed is generally unknown, and this makes round or square HSS an ideal design element because such sections have no weak axis in bending. Further, wide flange shapes—if relatively thin—may be susceptible to flange folding and web distortion under high blast pressures. Researchers at the recently created Centre for Resilience of Critical Infrastructure at the University of Toronto (www.crci.utoronto.ca) examined the behavior of cold-formed HSS through full-scale air blast tests. The tests were conducted in a remote desert location with various types of shaped charges and explosives. Two sizes of HSS columns with several wall thicknesses were tested as simply supported beams (without axial load).

Further, vertical members were tested in duplicate (unfilled and concrete-filled) pairs; they were clad with steel decking to provide a uniformly distributed lateral blast load to one side. Figure 1 shows one of the wall assemblies after the test, viewed from the rear of the wall and inside the containment structure. The HSS has been significantly plastically deformed.

In conjunction with the blast testing of HSS members, the performance of cold-formed HSS material is being studied experimentally under various high strain rates using a Split-Hopkinson pressure bar, as shown in Figure 2. The test specimens are taken from several locations around the HSS, including the corners, and are tested in compression as well as tension. The high strain rate properties and dynamic strength enhancement will be implemented into numerical models for use in simulation of HSS members under blast or impact loading.

The Charpy V-Notch (CVN) impact toughness of typical North American cold-formed HSS has typically been low (Kosteski, Packer and Puthli, 2005). Further, the CVN specimen to be used in product manufacturing standards is specified to be taken in the longitudinal direction from the flat face of a square or rectangular HSS. This gives the most optimistic toughness reading for the cross-section and has led to the cautionary note in ASTM A500 (2013) that the product "…may not be suitable for those applications such as dynamically loaded elements in welded structures." The recent release of an improved HSS product, the new ASTM A1085 with a CVN requirement of 25 ft-lb at 40 °F, is a considerable improvement. The measurement of CVN for various product types is continuing within the study of HSS under dynamic loading conditions. Complete toughness-temperature transition curves are generated experimentally for multiple locations and orientations around the HSS cross-sections.

Fig. 1. HSS structural wall assembly after testing (photograph courtesy of Professor J. A. Packer).

Welding of HSS: The design of welds to and between HSS members has been a topic of research at the University of Toronto since the 1980s. Two approaches to the design of such welds have been proposed:

1. Design the welds to achieve the yield strength of the connected branch member walls, which presents an upper bound on weld size as it satisfies any loading conditions.

2. Design the welds to be "fit for purpose" and resist the applied loads.

The latter approach, due to the flexibility of HSS connections, requires the use of effective weld lengths or effective weld section properties, which are now prescribed for square and rectangular HSS members in Section K4 of the 2010 AISC *Specification*. Some weld effective properties in the *Specification* are supported by research, while others, developed previously from informed knowledge, are now being investigated.

One recent project undertaken by the University of Toronto research team for AISC has dealt with examining the branch weld effective section modulus, for HSS-to-HSS T-connections under branch in-plane bending. A series of laboratory experiments, with varying influential connection parameters and weld-critical joints, were performed in a quasi-static manner until failure by weld fracture. All of the test welds were deposited by welding robots at the Lincoln Electric Automation Division in Cleveland, Ohio. Stepped as well as matched box connections were tested, where branch width equals the chord width, and so were fillet and partial-joint-penetration (PJP) groove welds. The PJP welds were deposited along rounded corners of the chord.

Equation K4-6 in the 2010 AISC *Specification* was ultimately shown to be conservative, and a more accurate version has now been developed (McFadden and Packer, 2013). It was also concluded that the fillet weld directional strength enhancement factor should not be used for strength calculations of welded joints to square and rectangular HSS. At this time, a second phase to this AISC research project is being undertaken in which weld-critical joints in overlapped HSS

K-connections are being tested, within a large-scale truss. The validity of further aspects of Table K4.1 in the AISC *Specification* will be analyzed.

Figure 3a shows a tested connection with a typical weld fracture, and Figure 3b shows the setup for the welded connections.

Elliptical Hollow Section Connections: For the last several years, research work examining the strength and performance of elliptical hollow sections (EHS) has been conducted at the University of Toronto. Although produced in Europe to Euronorm standard EN 10210 (CEN, 2006), architects worldwide like the geometry and general appearance of these shapes. As a result, a number of projects have used EHS in North America. Much work has been done on the section classification of the EHS (e.g., for compression and other members), but connections continue to be difficult. An international team with members from the University of Toronto, the National University of Singapore and Delft University of Technology in the Netherlands was established to address welded EHS-to-EHS connections. T-connections and X-connections have now been tested and analyzed, with chord and branch members oriented in multiple directions. It has been shown that the design of the EHS connections

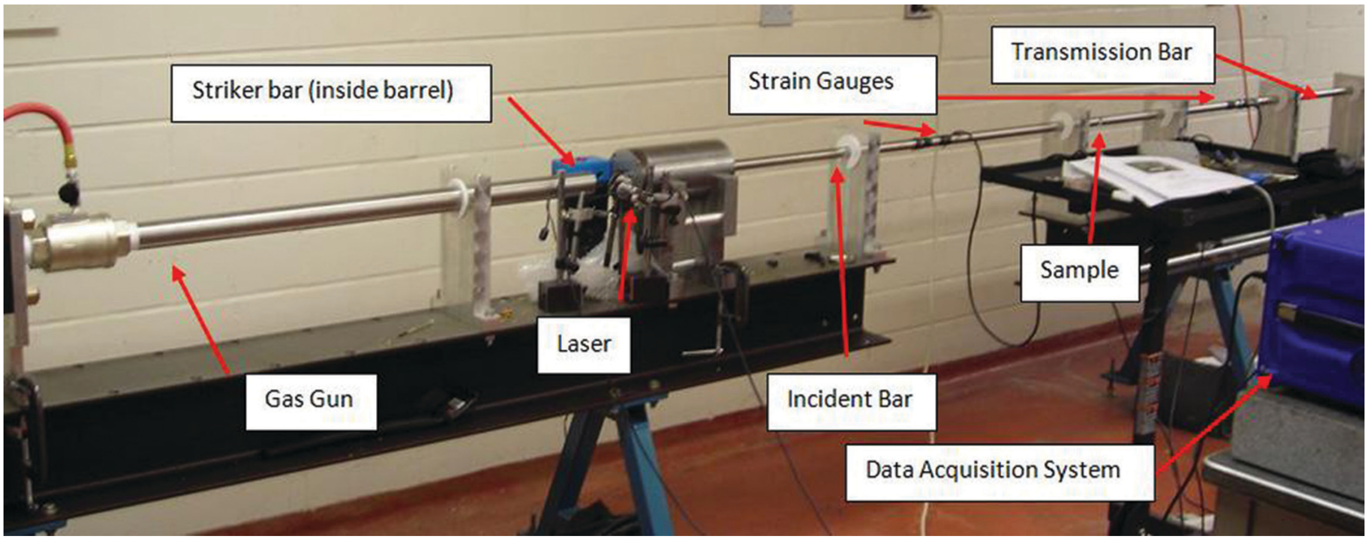
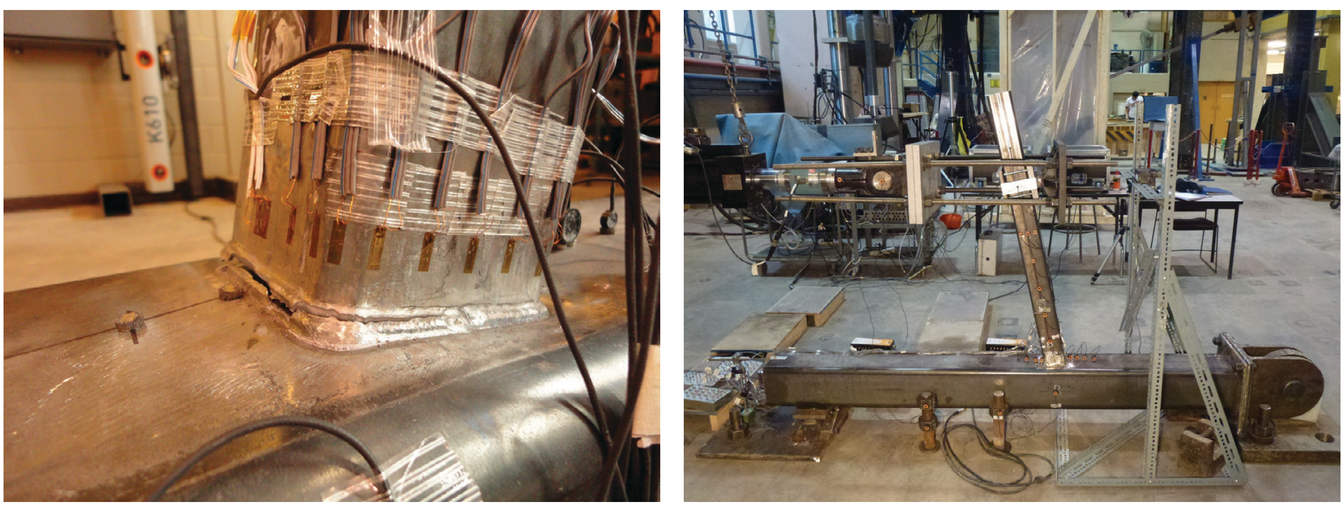


Fig. 2. Split-Hopkinson pressure bar apparatus at University of Toronto (photograph courtesy of Professor J. A. Packer).



(a) *(b)*

Fig. 3. Testing of welded connections: (a) typical weld fracture; (b) test setup (photographs courtesy of Professor J. A. Packer).

can be related to their circular and rectangular HSS counterparts (Packer et al., 2012; Haque and Packer, 2012; Shen et al., 2013a, 2013b). The complexity of the design of these connections has been effectively mitigated.

Figure 4a shows the appearance of a tested connection, Figure 4b illustrates the finite element model and its response at the same load level as the physical connection.

Cast Steel Connectors: Since 2004, much research work at the University of Toronto has addressed cast steel connectors for seismic applications. Steel castings have already been re-introduced in structural applications in Europe and China, but North America has been slow to capitalize on their advantages: the high material quality now achievable in steel castings, the ability to provide 3D freedom with connection arrangements, the ability to pre-engineer connectors to address specific connection performance requirements, and improved aesthetics (Herion et al., 2010). An innovative casting for application to the ends of a circular HSS brace member for a braced steel frame under inelastic seismic loading was engineered to provide a connection design solution (de Oliveira, Packer and Christopoulos, 2008). This high strength connector (HSC) was designed to remain elastic during seismic energy dissipation at mid-length of the brace and in the gusset plates. Subsequent research led to a new energy-dissipating device, in which the steel casting itself was designed to perform inelastically as a cast steel–yielding fuse. This yielding brace system (YBS) dissipates energy through the stable, single-curvature flexural yielding of specially designed "fingers" and provides an almost perfectly symmetric hysteretic response (Gray, Christopoulos and Packer, 2013). At the material level, the properties and fracture resistance of modern structural steel castings have also been checked (Iwashita, Packer and de Oliveira, 2012).

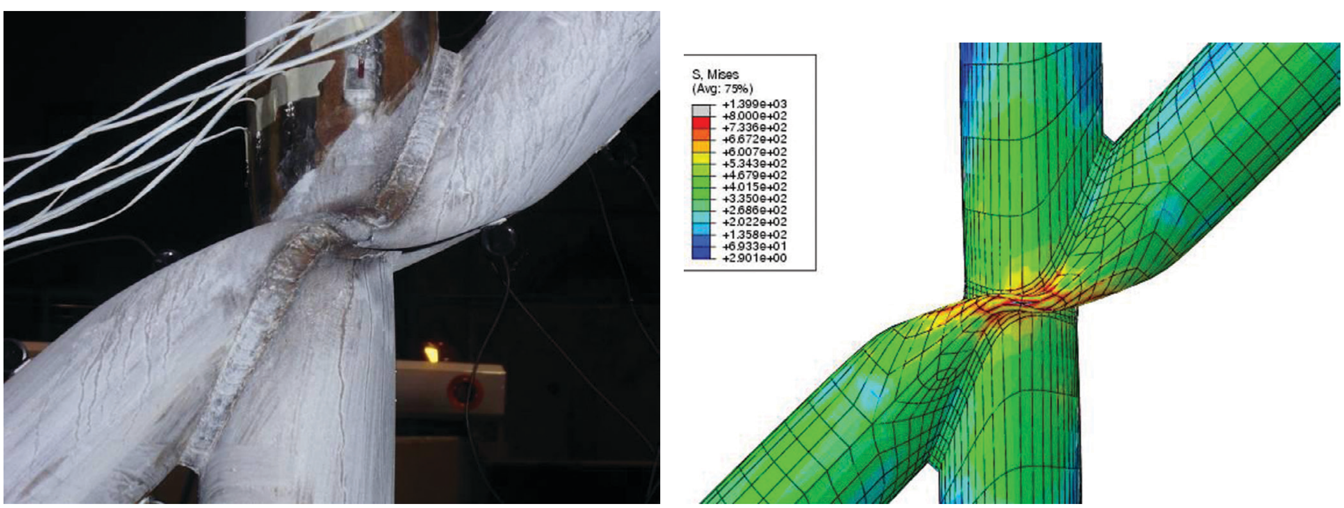
Figure 5a shows the use of high strength connectors in a braced frame in California; Figure 5b shows a full-scale seismic loading protocol test assembly for a yielding brace connector with a wide-flange diagonal brace.

Selected Projects of Professor Constantin Christopoulos

Professor Christopoulos is the director of the Structural Testing Laboratories at the University of Toronto and holds the Canada Research Chair in Seismic Resilience of Infrastructure. His current research focuses on the development of new high-performance, seismic-resistant systems that enhance the response of buildings subjected to extreme dynamic loads.

Self-Centering Energy Dissipative Braces for the Protection of Structures against Extreme Loading: Since 2003, Professor Christopoulos has collaborated with Professor Tremblay of École Polytechnique of Montréal on the development of the self-centering energy dissipative (SCED) bracing system. The system exhibits a flag-shaped response when subjected to extreme loading conditions. It is not only capable of accommodating large deformations, but also eliminates residual deformations and is practically undamaged even after a major earthquake. The concept has now been extended to incorporate the telescoping SCED (T-SCED) system (Erochko, Christopoulos and Tremblay, 2012). Full-scale experimental validations of the T-SCED system have shown that it is capable of accommodating up to 3.9% drift without any damage and with full recentering capabilities.

Development of Self-Centering Steel Moment-Resisting Frames: Professor Christopoulos has furthered the



(a) *(b)*

Fig. 4. Elliptical hollow section connection: (a) testing; (b) modeling (figures courtesy of Professor J. A. Packer).

development of post-tensioned "damageless" steel moment frames. The work has led to the development of friction-damped self-centering steel frames, as illustrated in Figure 6. The work has also led to a new design approach for detailing the beams to enable them to develop inelastic mechanisms for very large drift demands.

Eccentrically Braced Steel Frames with Replaceable Yielding Links: Professors Christopoulos and Tremblay developed this concept for seismic design of steel buildings. The work resulted in the development of a new method for constructing eccentrically braced frames with replaceable yielding links. This allows for more efficient construction and the ability to inspect and replace the yielding elements following an earthquake. Figure 7 shows one of the tested connections.

Base Rocking Steel Frames with Higher Mode Control Mechanisms: Professors Christopoulos and Tremblay have also led work on the development of steel rocking braced frames. Using higher mode limiting mechanisms it has been demonstrated that the seismic response of structures with base flexural mechanisms can be significantly enhanced by either using multiple flexural mechanisms over the height of the structure or by incorporating a self-centering shear mechanism in the first story to control the higher mode response.

(a) *(b)*

Fig. 5. (a) Use of high-strength cast connectors in a braced frame; (b) testing of a frame subassembly with a yielding brace connector (photographs courtesy of Professor J. A. Packer).

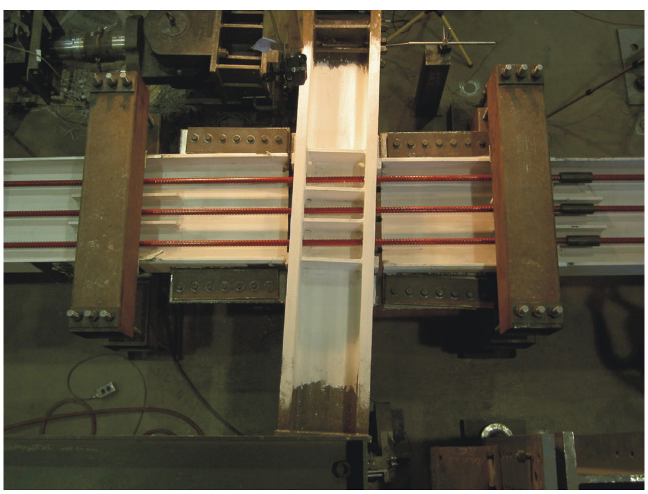


Fig. 6. Undamaged post-tensioned moment-resisting frame at 4% drift (photograph courtesy of Professor C. Christopoulos).

Fig. 7. Replaceable eccentrically braced link at 0.11 rad of rotation (photograph courtesy of Professor C. Christopoulos).

Selected Projects of Professor Oh-Sung Kwon

Hybrid (Analytical-Experimental) Simulation Method: Professor Kwon's research group focuses on simulation methods for a structural system subjected to dynamic load and seismic fragility assessment. A simulation framework has been developed, where the physical specimens can be integrated with a numerical model (Kwon, Elnashai and Spencer, 2008). The framework also allows geographically distributed simulation to exploit unique equipment at various institutions. In comparison with pure numerical simulations, the hybrid method can predict the response of a structural system more accurately when the system level response is influenced by the behavior of the physically represented components. It is especially beneficial when a reliable numerical model does not exist for a new structural component.

The hybrid simulation method was recently applied to evaluate the seismic fragility of a six-story steel structure with telescoping self-centering energy dissipating (T-SCED) braces (Erochko et al., 2012). The T-SCED brace exhibits a stable flag-shape hysteresis loop, which allows self-centering capability and seismic energy dissipation. In the hybrid simulation, one of the braces in the six-story building was experimentally represented at the University of Toronto's Structural Testing Facility, and the rest of the structure was analytically modeled. Because the T-SCED brace does not experience damage, it was possible to run more than 40 simulations to evaluate the seismic fragility of the structure statistically. The results showed that the six-story structure with T-SCED braces meets the performance limits specified in ASCE 41 and confirmed that the structure does not develop permanent deformation after earthquakes (Kammula et al., 2012).

SOME CURRENT RESEARCH WORK AT ÉCOLE POLYTECHNIQUE OF MONTRÉAL

The Department of Civil Engineering at École Polytechnique of Montréal (EPM) was established in 1873 and has since played a key role in the training of structural engineers in eastern Canada. Research in structural engineering was initiated in the late 1950s, with the construction of a major structural testing facility in 1958. The laboratory houses a 1,200-ft^2 strong floor and was subsequently equipped with a large reaction steel frame and various load control actuators and large capacity testing machines. In the early 1990s, it was decided to focus a significant part of the structural engineering research to seismic engineering, and a shake table facility was installed in 1995. This was the first implementation in North America of an earthquake simulator with a digital three-variable controller system (Filiatrault et al., 1996). In the aftermath of the 1994 Northridge earthquake, the facility was used for dynamic testing of reduced beam section beam-to-column connections (Tremblay, Tchebotarev and Filiatrault, 1997).

In 2003, the laboratory underwent a major expansion that was completed in 2006. The facility now offers 6,800 ft^2 of net strong floor testing area with a clear height up to 40 ft and a 33-ft tall L-shaped reaction wall with two 40-ft long wings. Loading equipment includes two new tension/compression frames, one with a 560-kip capacity and a larger one with a 2700-kip capacity, together with 10-ft wide and 26-ft high test space, 18 high performance actuators with capacities ranging from 22 kips to 450 kips, five digital control systems, and real-time hybrid testing capabilities. The shake table was moved; it can now accommodate 36-ft-tall specimens, and the actuator force capacity was increased to 100 kips. A new multi-axis loading system is currently being installed; it will be capable of imposing any combination of forces and deformations with 6 degrees of freedom to large structural components. Figure 8 shows a partial view of the facility. The system has eight actuators, four of which are horizontally mounted against the reaction wall (Tremblay et al., 2009; Tremblay, 2012).

The unique combination of large test beds and advanced testing equipment has permitted the completion of major testing programs on full- and reduced-scale structural components and systems. The facility plays a key role in the Canadian structural engineering research community. For instance, between 2006 and 2012, a total of 66 major projects involving graduate students have been completed. Half of the projects involved two or more researchers; a total of 15 researchers from seven other universities participated in the experimental programs. Some current and future projects are described in the next section, including tests on ductile fuses and collapse response following brace connection fracture, including buckling restrained braces.

Selected Projects of Professor Robert Tremblay

Ductile Fuses for HSS and I-Shaped Braces: In the past 20 years, a number of test programs have been performed at EPM to enhance the understanding of the inelastic cyclic response of bracing members and braced frames. Braces exhibit pronounced unsymmetrical response due to the difference between their tensile and compressive resistances and the degradation of the compressive strength under cyclic loading. These responses are illustrated in Figure 9a. For SCBF and OCBF, the braces are sized to resist the required compressive strength C_u, and their expected tensile strength may significantly exceed the required tensile strength ($T_{exp} > T_u$). Because compression and tension braces are used in pairs at every level, this overstrength offers beneficial effects on the response of braced frames because it compensates for the degrading resistance of the compression braces. In multistory frames, it also contributes to distributing the inelastic demand over the building height. However, brace connections, beams and columns must be designed for loads associated to the expected resistance, T_{exp}, which may have a significant effect on the cost of the structure.

That force demand can be minimized by introducing a ductile brace fuse created by locally reducing the brace cross-section area, as shown in Figure 9b. The fuse length, L_F, must be sufficient to accommodate the anticipated plastic deformations. Such fuses can be sized to yield both in tension and compression, such that brace buckling can also be avoided. However, achieving a stable inelastic straining in compression requires careful detailing to prevent local buckling and to avoid premature failure due to low-cycle fatigue. This is likely to add significant complexity. A simpler approach focuses on designing the fuses to be stronger than the braces in compression. The fuse yielding then only occurs in the form of successive unidirectional (tension) inelastic excursions. This allows for taking full advantage of the ductility of the steel without having to consider local inelastic buckling and low-cycle fatigue. This concept has been applied to the HSS braces shown in Figure 10 and the I-shaped braces in Figure 11. Full-scale tests have been performed for both designs (Kassis and Tremblay, 2008; Egloff and Tremblay, 2012).

For HSS braces, the brace member is cut near the end connection, and four angles are welded to connect the two HSS portions. The legs of the angles are trimmed to develop the required compressive strength. The fuse is enclosed in a built-up box to prevent angle buckling away from the HSS. Full-scale testing has been completed recently in a multipurpose test frame that simulates the boundary conditions of a braced frame story (Figure 10b). A typical test result is shown in Figure 10c: the brace tensile strength is reduced, as intended, while preserving the compressive resistance and the axial deformation capacity. The fuse design for I-shaped braces was initially proposed by the Canam Group, whereby portions of the brace cross-section at the flange to web intersection are removed (Vincent, 2008). The tensile axial

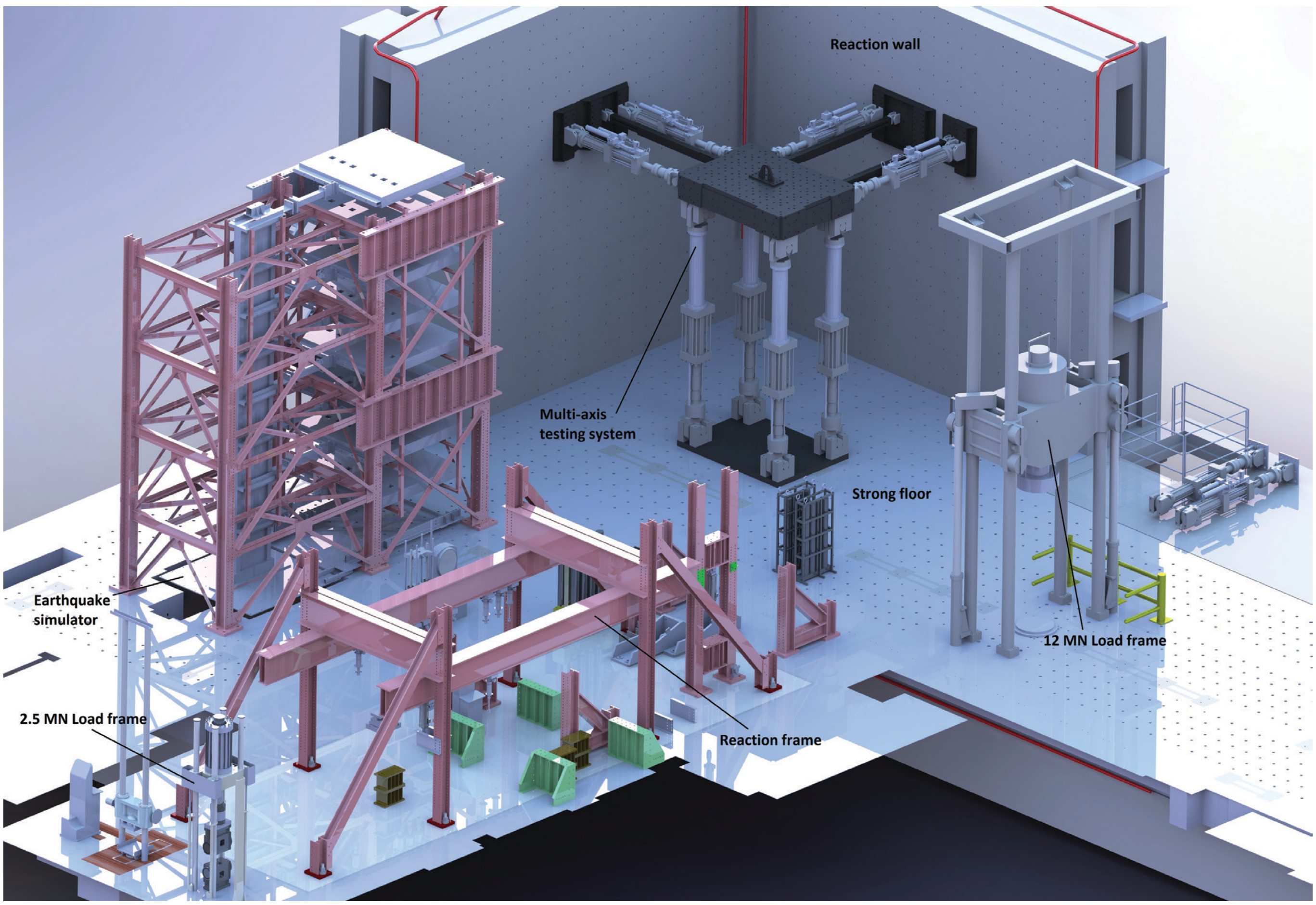


Fig. 8. Three-dimensional view of the EPM structural engineering laboratory (illustration courtesy of Professor Robert Tremblay).

strength can be reduced with a minimum impact on the brace buckling strength, as shown in Figure 10a. Confinement created by cold-formed C-shapes and flange cover plates must be provided to maintain the integrity of the cross-section upon brace buckling. Design equations were first proposed for the detailing of the fuse and confinement system, based only on detailed finite element analyses (Egloff and Tremblay, 2012). Full-scale testing was then performed to confirm the validity of the design expressions for various fuse geometries.

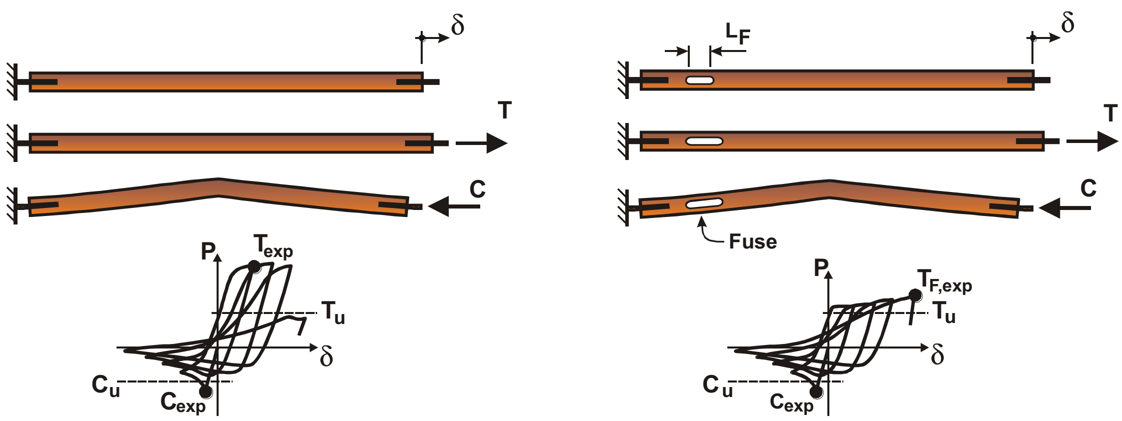
Both fuse systems can be constructed by steel fabricators. They are particularly suitable for long, slender braces, where the difference between tension and compression brace resistance is large, which is often the case in tall, single-story buildings. Reducing the brace overstrength for these structures has no or limited detrimental consequences on the building overall response. Ongoing research includes numerical simulations to verify the impact on multistory structures.

Seismic Response of X-Braced Frames: Results from past numerical studies and experimental programs have suggested that a K-factor of 0.5 is appropriate when determining the in-plane and out-of-plane buckling strengths of bracing members in an X-bracing system. In most tests, the braces were directly connected to each other at intersection points. This does not correspond to the case where bolted splice connections are used, as is commonly done in practice. This is illustrated in Figure 12a. In this situation, the flexibility of the overlapping plates may reduce the brace compressive resistance. The situation may be even more critical if single shear splices are used because an eccentricity is introduced that induces bending moments.

A project was started to resolve these questions and to develop design criteria for seismic and nonseismic applications. Fourteen specimens with HSS braces and single- and double-shear splice connections were tested. The brace size, the thickness of the connecting plates and the type of joint (single or double shear) were varied. Specimens with back-to-back double-angle braces were also tested. Testing was conducted using the same test frame as for the brace fuses (Gélinas, Tremblay and Daravan, 2012, 2013).

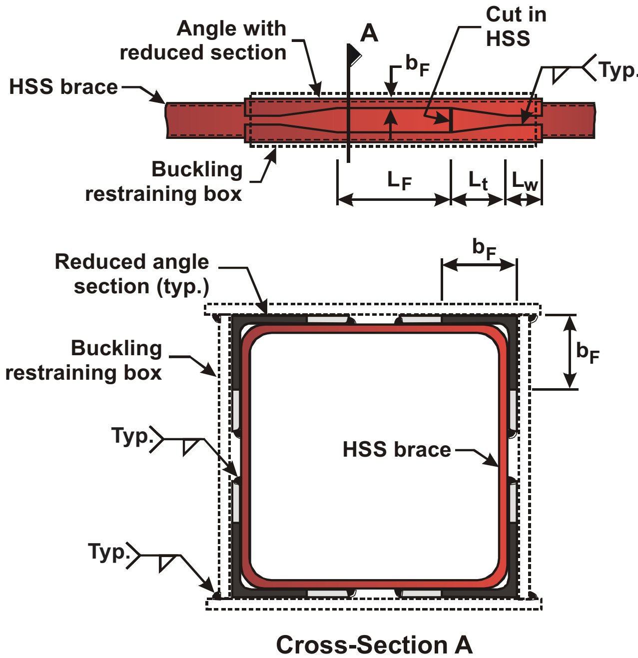
The HSS specimens were designed and detailed to force brace buckling out-of-plane of the frame. Examples of buckling modes that were generally observed for the continuous and discontinuous braces are shown in Figures 12b and 12c. In most cases, the discontinuous brace in tension could develop sufficient out-of-plane stiffness at the brace intersecting point to force buckling of the continuous brace at mid-length. Conversely, buckling of the discontinuous brace segments occurred in the form of a three-hinge mechanism, forming in the connecting plates at the brace intersecting point and the corner gusset plate (only a mid-connection plate is shown in Figure 12b). The buckling load associated with this mode was less than the brace compressive strength determined with K = 0.5. In some specimens, the cyclic rotation demand on the connecting plates led to premature failure of the plates in the tension cycle, which would not be adequate for ductile seismic performance. Analytical and numerical models have been developed to reproduce the measured brace capacities and simplified equations are being formulated to design connections that can develop the brace axial strength and prevent the buckling modes that were observed (Tremblay, Daravan and Gélinas, 2013).

Slotted Gusset Plate Connections for Slotted HSS Brace Connections: Slotted end connections as shown in Figure 13 are commonly used for HSS braces. The HSS members are slotted at their ends in the fabrication shop. At the construction site, they are inserted into the gusset plates to which



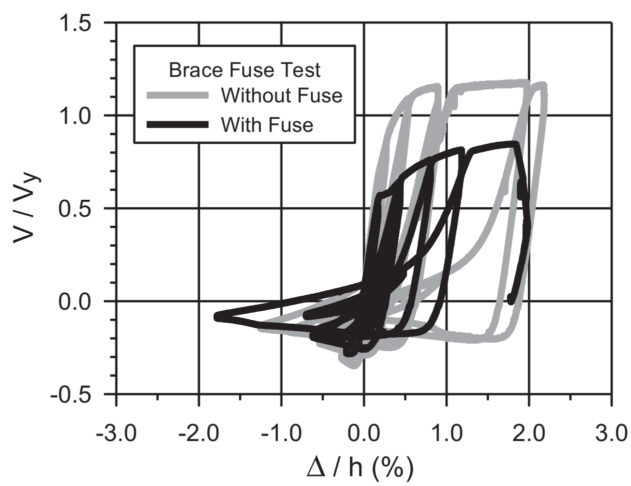
(a) *(b)*

Fig. 9. Hysteretic brace responses: (a) without brace fuses; (b) with brace fuses (illustrations courtesy of Professor Robert Tremblay).



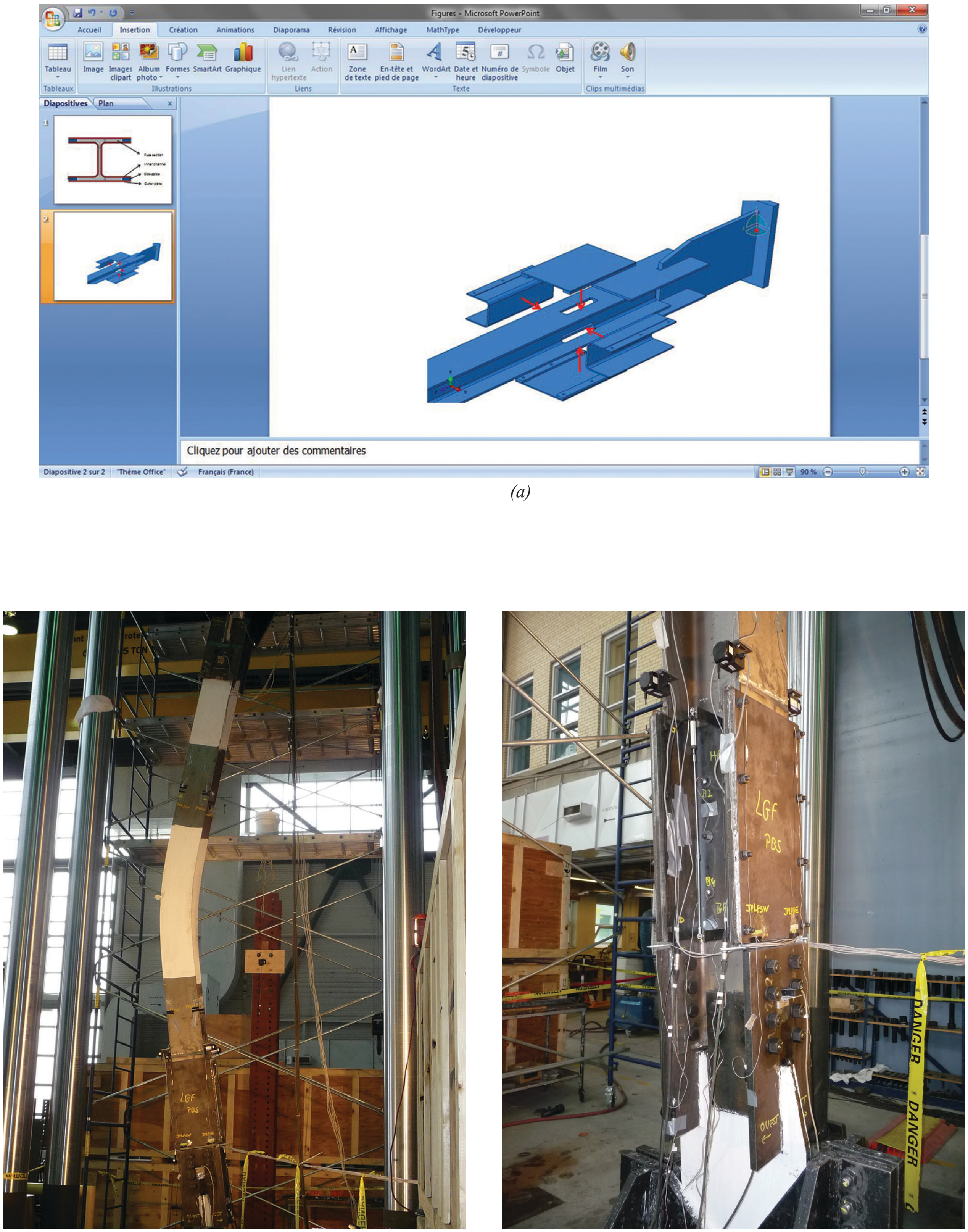
(a)

(b)



(c)

Fig. 10. Ductile tension fuses for HSS braces: (a) fabrication details; (b) full-scale testing in a re-usable test vertical frame; (c) typical hysteretic responses with and without fuses (illustrations courtesy of Professor Robert Tremblay).

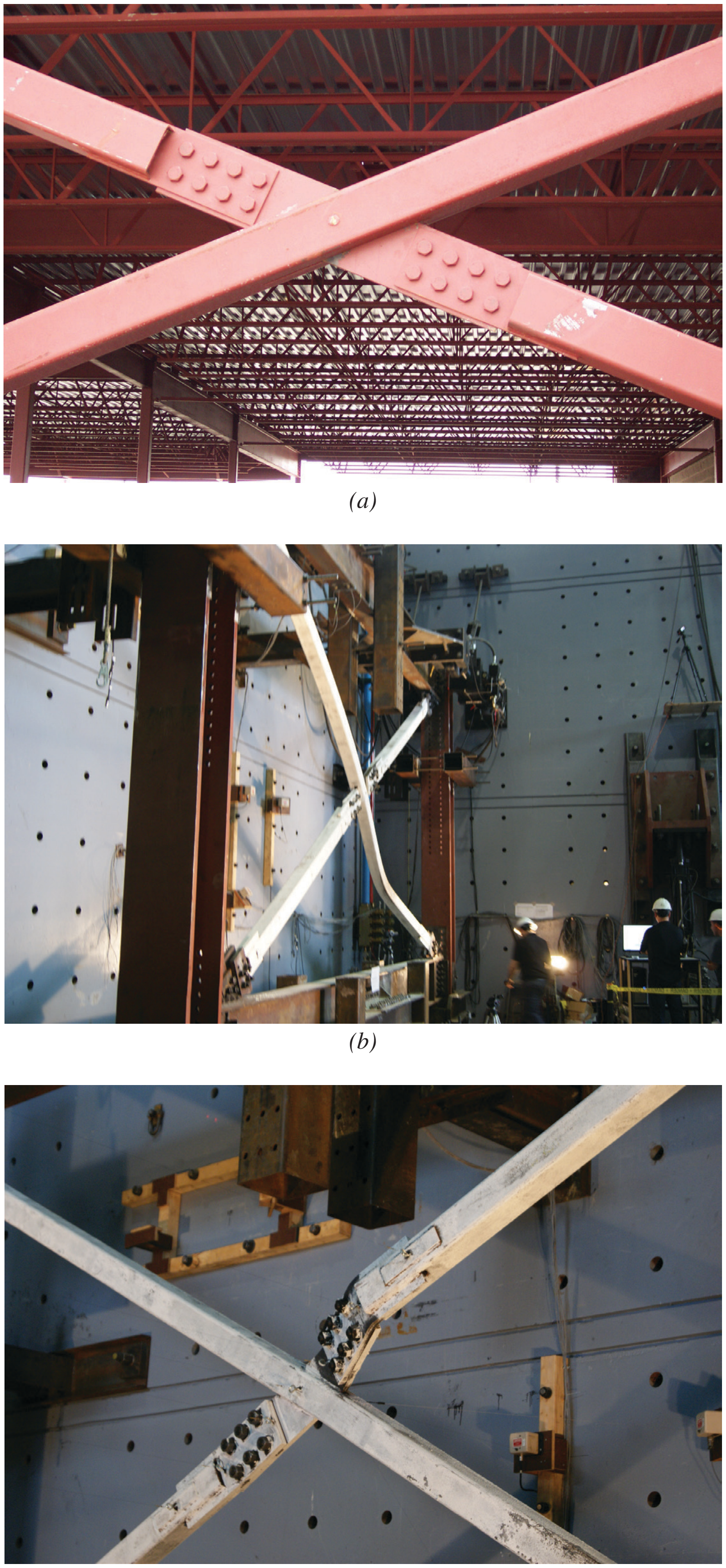


(a)

(b)

(c)

Fig. 11. Ductile tension fuse for I-shaped brace: (a) detail; (b) buckled brace with fuse in test; (c) bolted end connection with fuse (illustrations courtesy of Professor Robert Tremblay).



(a)

(b)

(c)

Fig. 12. Single splice mid-connection with through-plates in the continuous brace: (a) actual connection; (b) buckling of the continuous brace; (c) detail of the buckled shape at mid-connection with a three-hinge buckling mode of the discontinuous brace (photographs courtesy of Professor Robert Tremblay).

they are welded using two pairs of fillet welds. When used in special concentrically braced frames, these connections must be capable of developing the full tensile yield strength of the braces. However, test results from other projects have shown that a fracture may occur in the HSS on the net section at the ends of the slots before the brace yields. This will especially occur when shear lag causes a stress concentration at the weld ends. The problem can be solved by adding net section reinforcement. Other studies have shown that the target performance can still be achieved with unreinforced slotted HSS connections, provided that the net section resistance exceeds the brace yield capacity. This can be satisfied when shear lag effects are minimized by using long welds compared to the HSS cross-section dimensions.

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Professors Jeffrey Packer, Robert Tremblay, Constantin Christopoulos and Oh-Sung Kwon have provided significant assistance in the development of this paper. Their efforts are sincerely appreciated.

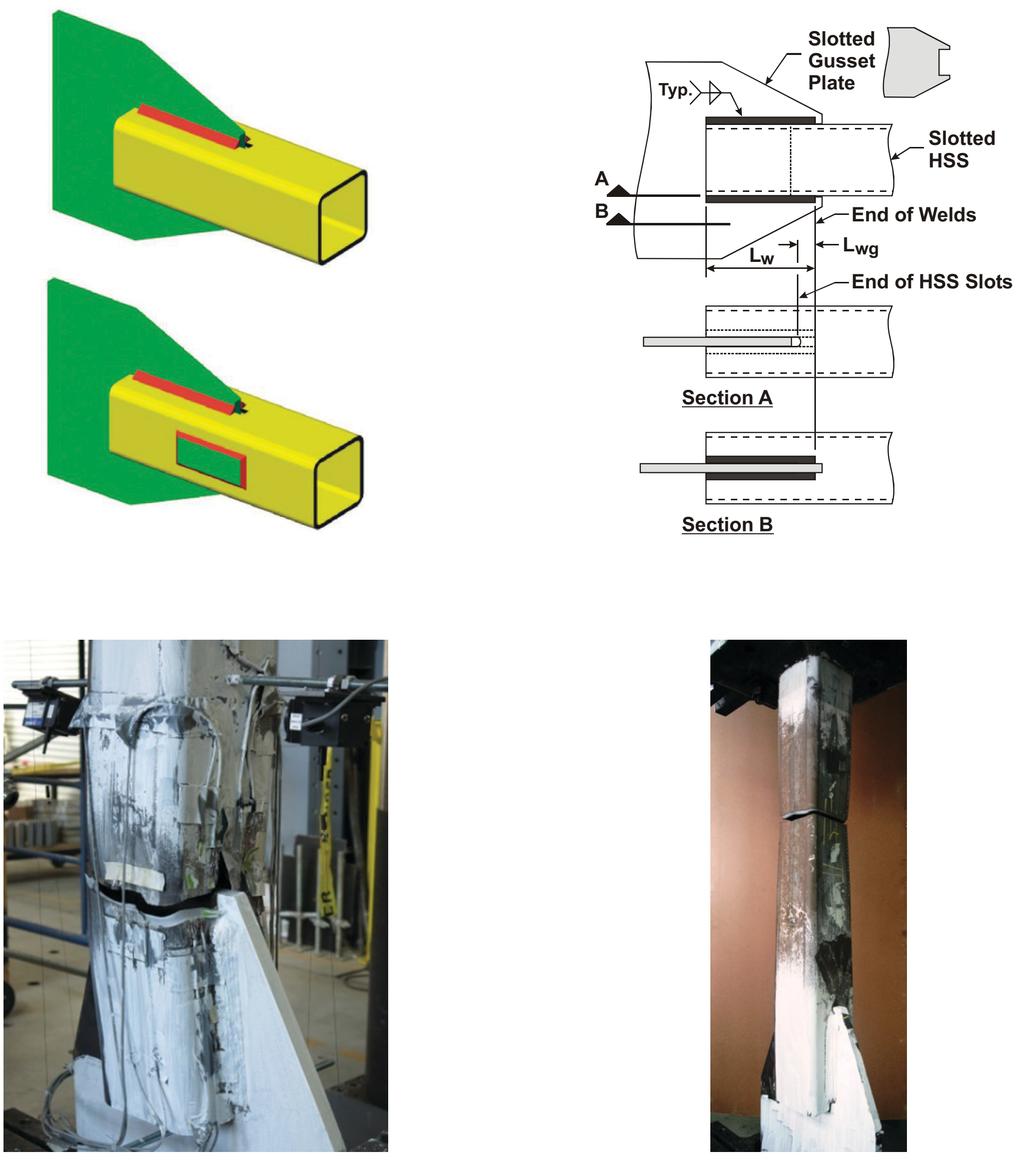

Fig. 13. Various slotted connections and failure modes for HSS bracing members (illustrations courtesy of Professor Robert Tremblay).

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