EXPERIMENTAL SEISMIC RESPONSE OF SLOTTED CONNECTIONS AT THE INTERSECTION OF HSS BRACES IN X-BRACING SYSTEMS

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ABSTRACT

This paper discusses an ongoing experimental study evaluating the seismic performance of bolted connections used in steel X-braced frames. Ten full-scale specimens with one continuous and one discontinuous tubular (HSS) diagonal bracing members connected at the intersection points are studied. The frames are of the conventional construction (Type CC) and moderately ductile (Type MD) categories of CSA S16-09. Two types of slotted brace connections are examined: single shear splices exhibiting inherent out-of-plane eccentricity and symmetrical double shear splice plates. The influence of the brace size, connection geometrical parameters such as plate thickness, plate width, length of overlapped plates, and the presence of stiffeners are also examined. The tests showed that instability of the connections is a common behaviour for all the studied cases. Local stability effects must therefore be accounted for in the design and response of this type of X-bracing systems.

1. INTRODUCTION

Steel braced frames are among the seismic force resisting systems commonly preferred by design engineers for building and industrial structures in Canada. These frames must be designed and detailed in accordance with the provisions of the 2010 National Building Code of Canada (NBCC) (NRCC, 2010) and the special seismic requirements contained in the CSA S16-09 standard for the design of steel structures (CSA 2009). According to NBCC 2010, the design seismic loads are obtained using ductility-related and overstrength-related force modification factors, R_d and R_o, respectively. While the former is related to the ability of the structure to dissipate the induced seismic energy through inelastic response, the R_o factor reflects the dependable lateral overstrength offered by the system (Mitchell *et al.*, 2003).

In CSA S16-09, steel concentrically braced frames (CBFs) are categorized into three categories: Type CC (conventional construction) with R_d = 1.5, Type LD (limited ductility) with R_d = 2.0, and Type MD (moderately ductile) with R_d = 3.0. A factor R_o = 1.3 is assigned to all three systems. The seismic response of Type CC CBFs is expected to be essentially elastic with limited ductility demand. The system relies on the inherent ductility of steel and other sources of energy dissipation that normally exists in ordinary steel frames (e.g., friction) to dissipate the seismic energy input. In moderate and high seismic regions, the connections along the structure lateral load path must be either detailed to exhibit a ductile failure mode or designed for seismic induced forces magnified by 1.5. This amplified seismic force level corresponds to elastic seismic response level (R_d = 1.0). Amplified seismic design loads also apply to member design for Type CC CBFs used in structures taller than 15 m. Nonetheless, the seismic design of these frames remains easier in comparison to the comprehensive and lengthy capacity design procedure applicable to the more ductile Type LD and Type MD CBF systems. For these two CBF categories, brace connections as well as beams and columns, must be designed for seismic effects based on the probable brace axial resistances.

For all CBF categories, hollow structural sections (HSS) bracing members are often preferred in design as they exhibit superior compressive resistance per unit weight compared to W sections or double angle braces. The X-bracing configuration with bracing members intersecting at a middle connection is also commonly used to take advantage of the shorter effective length for the compression brace resulting from the lateral support provided by the tension-acting brace, either in the plane or out of the plane of the frame, as demonstrated in past studies (e.g., El-Tayem and Goel, 1986; Picard and Beaulieu, 1987).

Due to the mid-connection, X-bracing typically have one continuous brace member and one discontinuous brace member, and various details are employed to transfer the load between the two disconnected brace segments at the middle connection. Satisfactory cyclic inelastic performance was observed experimentally in X-braced frames with HSS members when tubes were directly welded to each other and continuity plates were used at the mid-connections (Tremblay *et al.*, 2003). Bolted connections with splice plates are also typically used in X-bracing, as illustrated in Figure 1. In both cases, a plate known as through plate passes through the continuous brace and single shear lap (SS) joints (Figure 1a) or double shear (DS) lap joints (Figure 1b) are used. For this type of connections, numerical studies by Davaran (2001) and Davaran and Hovaidae (2009) showed that the bending stiffness of the connecting plates at the mid-connection may affect the buckling compression capacity of the bracing members. Concerns have also been raised regarding the possibility that single shear (SS) lap joints with inherent eccentricity could lead to local instability of the connection when loaded in compression.

a) b)

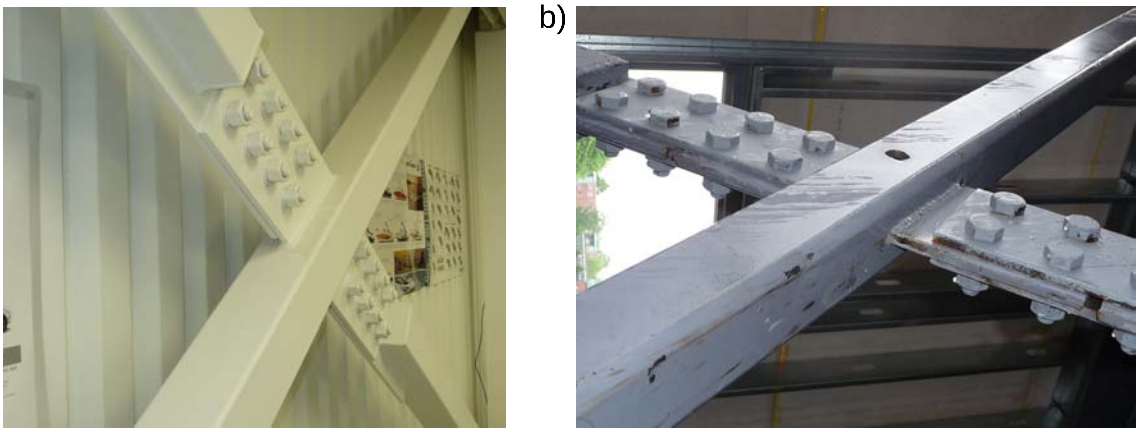

Figure 1. Bolted connections at brace intersection in X-bracing:
a) Single shear lap splices; b) Double shear lap splices.

A full-scale cyclic quasi-static test program has been initiated to investigate the stability response of X-bracing built with HSS members. Type CC and Type MD braced frames designed with single shear splices exhibiting out-of-plane eccentricity and symmetrical double shear splice plates are studied. Preliminary results from 4 tests revealed that buckling of the mid-connection can precede buckling of the diagonal members, leading to reduced brace compression resistance and localized inelastic demand on the connections (Gélinas *et al.*, 2012). This paper describes the behaviour of 10 specimens as obtained in that test program.

2. TEST PROGRAM

X-bracing specimens are tested in a 4.087 m tall x 7.5 m wide test frame vertically mounted in the Structural Engineering Laboratory at Ecole Polytechnique of Montreal (Figure 2). The focus of the test program is on the effect of the mid-connection design on frame response. Hence, the beams and columns of the frame are sized to remain elastic when inelastic response develops in the braces and their connections. At the frame base, the columns are connected to the strong floor through true pins fastened for vertical reactions. The beams are connected to the columns with bolted web clip angles with horizontal slotted holes to prevent the development of bending moments in the frame connections. Two high-performance 1000 kN actuators are employed to apply a predefined horizontal cyclic displacement protocol to the specimens.

a) b)

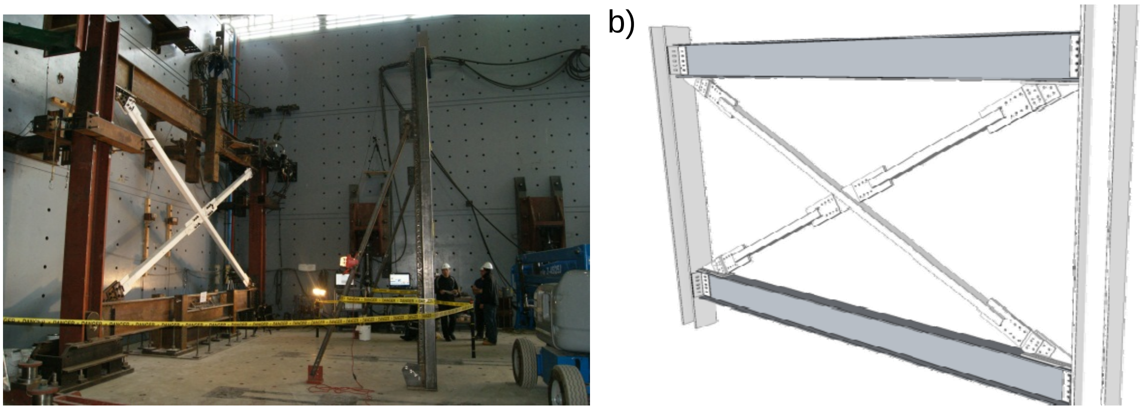

Figure 2. Experimental setup: a) Laboratory; b) Frame illustration.

The properties of the test specimens are given in Table 1. Details of the mid-connection are given in Table 2 and illustrated in Figure 3. The specimen ID indicates the type of splice connections (DS *vs* SS), the thickness and width of the splice plates, and the CBF type (Type CC *vs* Type MD). Through plates passing in the continuous braces had the same width and thickness as the splice plates for the SS connections. Through plates were two times thicker than the splice plates for DS specimens. In Test 4, angles were used to connect the discontinuous brace segments to the through plate, without slotted holes, as shown in Figure 3b. All braces are made of ASTM A500, gr. C, HSS members with nominal yield strength, F_y = 345 MPa, ultimate tensile stress, F_u = 427 MPa, and nominal section properties: A = 3400 mm² and r = 48.3 mm for HSS 127x127x8, and A = 2170 mm² and r = 38.6 mm for HSS 102x102x6. Table 1 also gives the brace length between the hinge location in the corner gusset plates, L_h, as well as the measured brace section properties.

The brace connections were designed for brace axial forces, S_f, determined assuming the nominal brace section properties. For Type CC CBF specimens, the specimen labels with CCS and CCM extensions in Tables 1 & 2 indicate that the connections were designed for magnified seismic loads, $S_f = 1.5 \times C_r$, and non-magnified seismic loads, $S_f = C_r$, respectively, where C_r is the factored brace axial compression resistance determined assuming a brace effective length $KL = 0.5\ L_h$. For Type MD specimens, the connections were designed for the brace probable resistance in tension, $S_f = A \times R_y \times F_y$, and the brace where R_yF_y = 460 MPa, as prescribed in CSA S16-09 for HSS braces. The connection design forces are given in Table 2. Stability of the connections was not verified in the design, except for the unsupported segments of the splice plates between the HSS and last row of bolts.

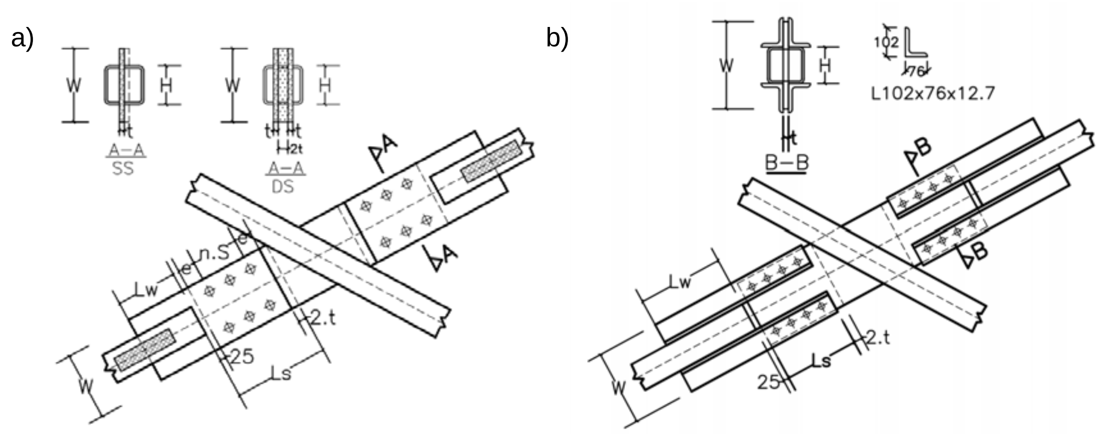

Figure 3. Middle connection geometrical parameters for single shear (SS) lap and double shear (DS) lap configurations with: a) splice plates; and b) angles.

Table 1. Specimen properties.

Test	**Specimen**	**HSS brace**	**Brace length**	**Continuous member**			**Discontinuous member**	
No	**ID**	**d x b x t (mm)**	**L_h (mm)**	**A (mm^2)**	**r (mm)**	**F_y (MPa)**	**A (mm^2)**	**r (mm)**
4	DS-L4x3-MD	127x127x8	6502	3592	48.2	439	3408	48.3
6	DS-9.5x220-CCS	127x127x8	6510	3583	48.2	439	3487	48.3
7	SS-16x280-CCS	127x127x8	6507	3639	48.1	440	3552	48.2
8	SS-25x200-CCS	127x127x8	6491	3661	48.1	440	3529	48.2
9	SS-16x280(S)-CCS	127x127x8	6510	3474	48.3	440	3677	48.1
10	DS-8x180-CCM	127x127x8	6510	3587	48.2	440	3547	48.2
11	DS-9.5x300-CCM	127x127x8	6502	3574	48.2	440	3477	48.2
12	DS-8x240-MD	102x102x6	6510	2364	38.5	512	2436	38.4
13	DS-8x150-CCM	102x102x6	6510	2412	38.5	512	2393	38.5
14	SS-16x150-CCM	102x102x6	6507	2340	38.5	512	2340	38.5

Table 2. Details of mid-connections.

Test No	**Specimen ID**	**W (mm)**	**t (mm)**	**e (mm)**	**S (mm)**	**L_s (mm)**	**Bolts n x d_b (mm)**	**S_f (kN)**
4	DS-L4x3-MD	304	19.05	50	75	325	8 x 22.225	1564
6	DS-9.5x220-CCS	220	9.525	50	75	307	6 x 22.225	1050
7	SS-16x280-CCS	280	15.875	71	95	389	6 x 31.75	1050
8	SS-25x200-CCS	203	25.4	71	95	406	6 x 31.75	1050
9	SS-16x280(S)-CCS	280	15.875	71	95	389	6 x 31.75	1050
10	DS-8x180-CCM	180	7.9375	50	75	232	4 x 22.225	700
11	DS-9.5x300-CCM	320	9.525	50	75	240	4 x 19.05	700
12	DS-8x240-MD	240	7.9375	50	75	307	6 x 22.225	998
13	DS-8x150-CCM	152.4	7.9375	36	75	203	4 x 15.875	357
14	SS-16x150-CCM	152.4	15.875	50	75	232	4 x 22.225	357

Specimens 7 and 9 were identical specimens with single shear lap joints, except that one stiffener was added to one side of the splice plates at the mid-connection to improve the connection compression strength for Specimen 9. The angle connection used for specimen DS-L4x3-MD (Figure 3b) can be distinguished from the rest of connections regarding its increased stiffness at junction of HSS to gusset plate. This connection is built without any slot in the discontinuous HSS members and is suggested as an alternative detail to prevent buckling of the mid-connection. Except for Specimen No.13, the space left between the double splice plates of the discontinuous HSS members in all DS specimens was filled with a shim (filler) plate. In Specimens Nos. 10 and 12, the shim plate was connected to the splice plates using tack welds. In Specimens Nos. 6 and 11, the shim plate was entirely welded to the splice plates along its edges.

3. OBSERVED BEHAVIOUR

All specimens were subjected to a symmetrical displacement protocol with stepwise increasing amplitudes (Gélinas *et al.*, 2012). The storey drifts when storey shear decreased to 80% of the peak storey shear and the observed failure modes are described in Table 3. Failure occurred in tension for all specimens. For 5 specimens, failure took place at the location where plastic hinging developed upon buckling. For these cases, low-cycle fatigue (LCF) contributed to failure. Except for Specimen No. 4, all specimens experienced buckling in the connections, as shown in Figures 4 and 5 for Specimens Nos. 7 (SS) and 6 (DS), respectively. For 8 of these 9 specimens, buckling occurred first in the mid-connection of the discontinuous HSS members. In Specimen No. 11, buckling was observed first in the corner connection of the discontinuous member. Hence, discontinuous brace members in X-configuration with traditional single or double shear lap slotted connections could not develop the conventional buckling mode in the member, as expected in design. Buckling of the discontinuous bracing member was observed in Test No. 4. Buckling of the continuous bracing member was observed for all SS specimens as well as for DS Specimen No. 4. For all other DS specimens, buckling of the continuous brace occurred first at the end connections. Further detail on the observed behaviour of SS and DS connections are discussed in sections 3.2 and 3.3. The measured hysteretic responses for the diagonals of some of specimens are presented in Figure 6.

Table 3. Maximum storey drift and failure modes.

Test No.	Storey Drift[1] (%h_s)	Failure mode
4	1.4	Tension - net area (slot) of continuous HSS at mid-connection
6	1.0	Tension - net area (bolt holes) of splice plate of continuous HSS at end connection (LCF)
7	0.8	Tension - net area (weld end) of splice plate of discontinuous HSS at mid-connection (LCF)
8	1.5	Tension - net area (slot) of continuous HSS at mid-connection slot
9	1.0	Tension - net area (weld end) of splice plate of discontinuous HSS at mid-connection (LCF)
10	0.8	Tension - through plate, after tearing at mid-connection
11	1.4	Tension - gusset plate of discontinuous HSS at end connection (LCF)
12	1.1	Tension - net area (slot) of continuous HSS at end connection, after tearing
13	0.8	Tension - net area (slot) of continuous HSS at end connection, after tearing
14	0.5	Tension - net area (weld) of splice plate of discontinuous HSS at mid-connection (LCF)

[1] Storey drift when storey shear reduces to 80% of peak storey shear (h_s = storey height)

3.1 Brace compressive strengths and effective lengths

Maximum recorded brace axial compression forces (C_{max}) and tension forces (T_{max}) forces are given in Table 4. The C_{max} values are compared to the predicted brace compressive resistances, $C_{u,pred}$, as determined with the measured brace cross-section and material properties of Table 1 and assuming $KL = 0.5L_h$. Except for Specimen No. 4, the test-to-predicted ratio for all discontinuous members are lower than 1.0, with an average value of 0.77, which resulted in effective length factors, K_{eff} higher than 0.5 (average value = 0.64). These low compressive resistances are due to instability in the connections of the discontinuous braces. The differences are less important for the continuous braces, with an average K_{eff} equal to 0.56, indicating that the discontinuous members, when loaded in tension, can still restrain out-of-plane buckling of the continuous member under compression. All connections could develop in tension the design connection forces.

Table 4. Brace axial strengths and effective length factors.

Test No.	Discontinuous Bracing Member			Continuous Bracing Member			Tensile Strength	
	C_{max} (kN)	C_{max} $/C_{u,\ pred}$	K_{eff}	C_{max} (kN)	C_{max} $/C_{u,\ pred}$	K_{eff}	T_{max} (kN)	T_{max} $/S_f$
4	1125	1.27	0.37	984	1.05	0.47	1627	1.04
6	847	0.93	0.53	948	1.02	0.49	1541	1.47
7	454	0.49	0.85	808	0.85	0.58	1426	1.36
8	535	0.58	0.77	763	0.80	0.61	1431	1.36
9	829	0.87	0.57	871	0.96	0.52	1420	1.35
10	586	0.64	0.72	629	0.67	0.69	867	1.24
11	474	0.52	0.82	550	0.59	0.76	1015	1.45
12	477	0.93	0.53	604	1.21	0.43	1199	1.20
13	433	0.86	0.55	474	0.93	0.52	715	2.00
14	284	0.58	0.71	483	0.98	0.51	585	1.64
	Average =	0.77	0.64		0.91	0.56		1.41

3.2 Specimens with single shear (SS) lap connections

Specimens Nos. 7 and 8, 9 and 14 are in this group. The first nonlinear event was buckling of the connecting plates at the mid-connection in a two-hinge mechanism (Figure 4b). Except for Specimen No. 8, the specimens in this group failed by net section rupture of the splice plate at mid-connection, with pronounced low cycle fatigue effects due to cyclic plastic rotation in the hinge located near the end of the discontinuous HSS. The behaviour of Specimen No. 7 is illustrated in Figure 4. Specimens Nos. 7 and 8 were identical except that thicker splice plates were used at the mid-connection of Specimen No. 8 (25 mm *vs* 16 mm). This increased plate thickness resulted in a higher buckling resistance (C_{max} = 454 kN *vs* 535 kN for Specimens Nos. 7 and 8) and instability occurring later in the test, at storey drifts of 0.2% and 0.3% h_s, respectively, for Specimens Nos. 7 and 8 (see Table 3). Specimen No. 9 was a duplicate of Specimen No. 7 except that a stiffening plate was welded over the splice plates at mid-connection. This also resulted in higher compressive resistance (829 *vs* 454 kN). For these three specimens, buckling of the continuous brace took place at a storey drift of 0.5% h_s. In Table 4, the average K_{eff} factors for this group are 0.725 and 0.56 for the discontinuous and continuous members, respectively. For the SS specimens, the maximum and minimum storey drifts at failure, when storey shears reduced to 80% of peak storey shear, belong to Specimens Nos. 8 and 14 (1.5% h_s and 0.5% h_s, respectively). Though the addition of stiffeners in Specimen No. 9 successfully improved the buckling load of the discontinuous member compared to Specimen No. 8, the ductility of the latter was higher (1.5% h_s *vs* 1.0% h_s).

3.3 Specimens with double shear (DS) lap connections

This group comprised Specimens Nos. 6, 10, 11, 12 and 13. For all these specimens, connection buckling was the first non linear event: at the mid-connection for Specimens Nos. 6, 10, 12 and 13, and at the end connection for Specimen No. 11. Later in the displacement protocol, global buckling of the continuous HSS occurred in a single curvature pattern which gradually changed to double curvature as the discontinuous brace straightened upon increasing further the imposed displacement. In most of the specimens, global buckling of the continuous HSS was observed in the first few cycles only, as buckling at its corner connections eventually developed at larger frame displacements. Repeated buckling of the mid-connections of the discontinuous HSS and of the end connections of the continuous braces were observed until failure of the specimens. The behaviour of Specimen No. 6 is illustrated in Figure 5.

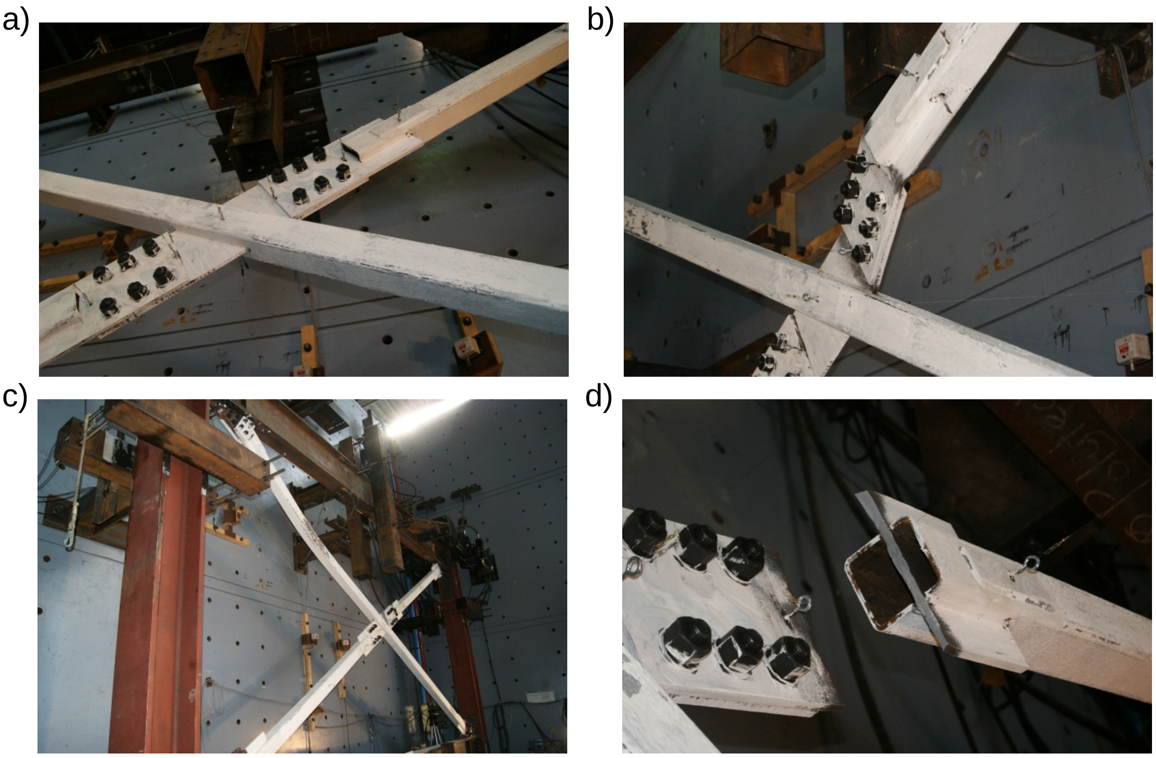

Figure 4. Behaviour of single shear connection (Specimen No. 7): a) Connection; b) Buckling of connection in the discontinuous HSS; c) Buckling of the continuous HSS; and d) Failure at mid-connection.

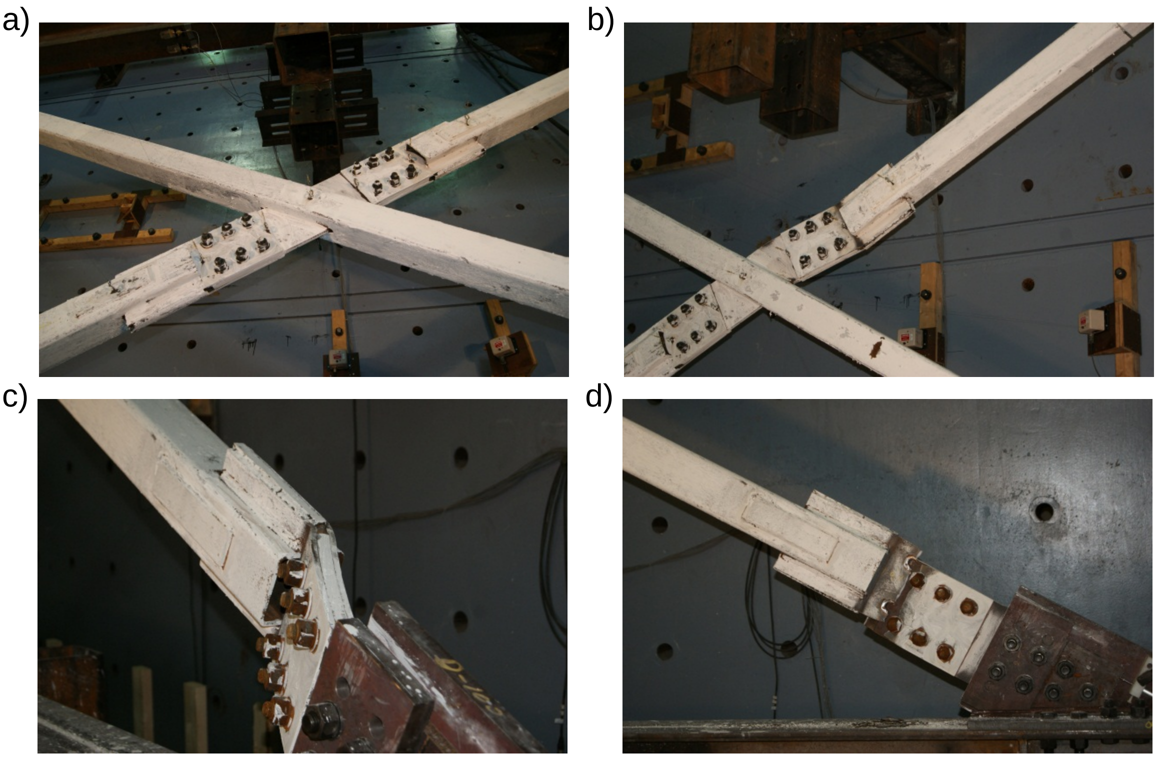

Figure 5. Behaviour of a double shear connection (Specimen No. 6): a) Connection; b) Buckling of the connection in the discontinuous HSS; c) Buckling of the connection in the continuous HSS; and d) Failure at end connection.

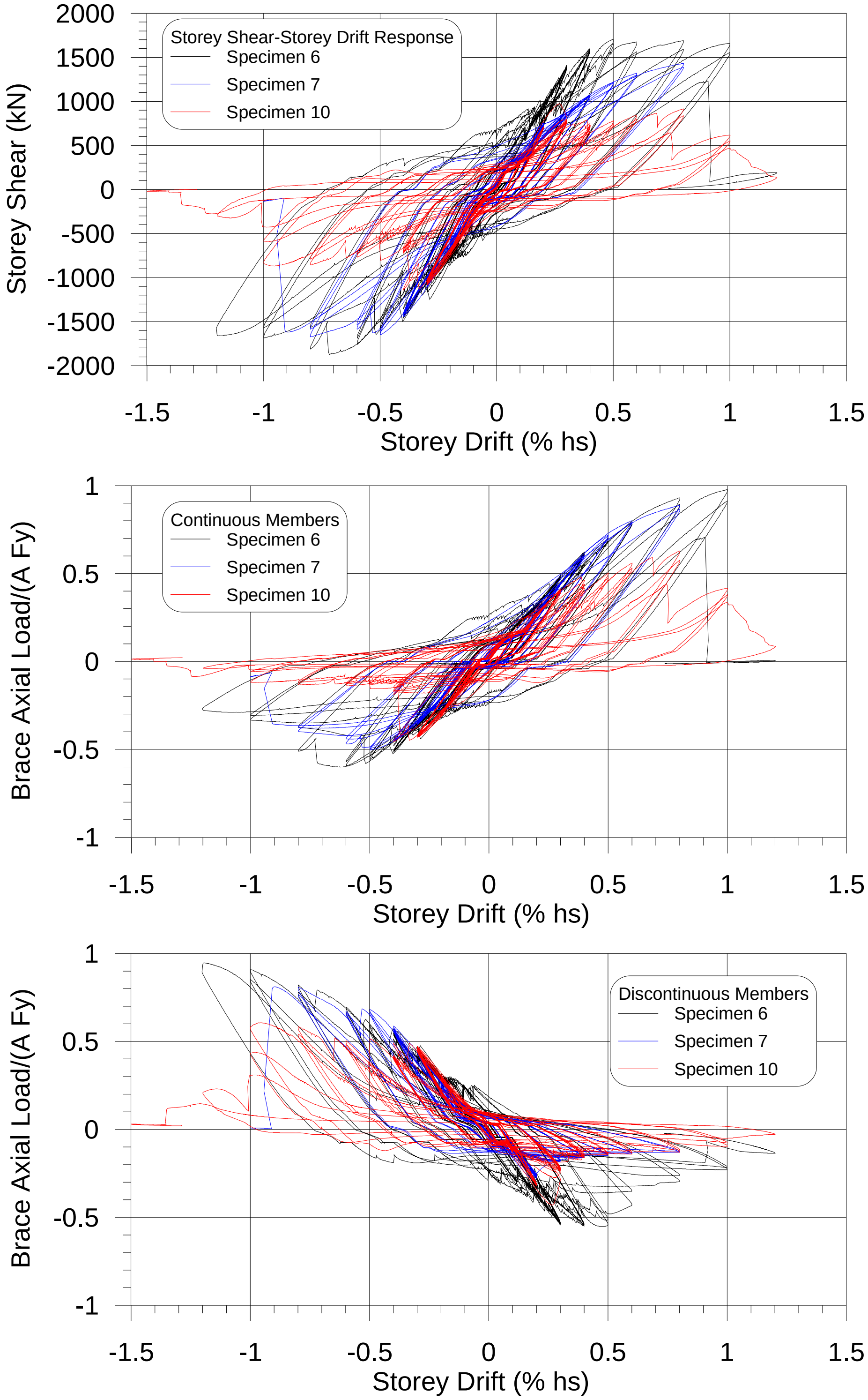

Figure 6. Hysteresis behavior for specimen 6, 7 and 10. 1) Total storey shear; 2) Continuous diagonal; 3) Discontinuous diagonal.

The other studied parameters in the DS group were the presence of the shim (filler) plate within the HSS slots (between the double splice plates) and the welding technique used to connect these plates. No shim plate was used in Specimen No. 13. In Specimens Nos. 10 and 12, shim plates were tack welded whereas continuous welds were used over the entire shim plate length for Specimens Nos. 6 and 11. The results show that inserting shim plates can improve the behaviour of DS connections, especially when the shim plates are fully welded.

3.4 Connection tension strength and ductility demand

Failure of Specimens Nos. 4 and 8 took place in the continuous brace at the reduced (net) section created to insert the through plate (see Table 3). The remaining 8 specimens failed at the connections transferring axial brace loads at the ends of the HSS segments. In Table 4, the maximum recorded brace tension forces, T_{max}, exceeded the design connection forces, S_f, in all cases, with an average test-to-design (T_{max}/S_f) load ratio of 1.41. For Type MD Specimen No. 4, however, the observed failure of the continuous brace at the net section is not acceptable as brace gross yielding in tension must be achieved. Net section reinforcement plates must be provided as necessary at the through plate location, similar to reinforcement used at the brace end connections. Failure at net section in the other Type MD specimen (No. 12) is also undesirable. This failure was likely due to local connection instability and could probably be avoided by stiffening the brace connection.

In Table 3, the average storey drifts for the 9 specimens with slotted SS or SD configuration is 0.99% h_s, varying from 1.5% h_s for (No. 8) to 0.5% h_s (No. 14). In Specimen No. 8, the thick (25 mm) splice plates designed for magnified seismic loads exhibited good ductility under repeated bending in compression followed by straightening in tension. Conversely, Specimen No. 14 had the smallest splice plates (16 mm x 150 mm) for an SS connection. This specimen exhibited the smallest deformation capacity together with pronounced strength degradation. Figure 6 shows the hysteresis behavior of specimens 6, 7 and 10. All specimens are Type CC CBFs with HSS 127x127x8 members. Specimens Nos. 6 and 10 differ by the technique used for connecting the shim plates to the splice plates: continuous welds in Specimen No. 6 *vs* tack welds in Specimen No. 10. The latter exhibited a better hysteretic response in Figure 6. Type CC Specimens Nos. 6 and 7 respectively have DS and SS connections, Connections of both specimens were designed for magnified seismic loads. The frame with DS connections (No. 6) showed higher ductility compared to the frame with SS connections.

4. CONCLUSIONS AND RECOMMENDATIONS

Cyclic quasi-static testing has been performed on 10 full-scale X-braced frame specimens with HSS bracing members and common bolted-slotted connections to study the effect of mid-connection design on the frame seismic inelastic response. Type MD and Type CC braced frame categories were examined, and two different connection design loads were used for the Type CC CBFs. The other main parameter investigated in this study was the splice connection configuration: single shear (SS) and double shear (DS) lap joints. The main conclusions from this test program can be summarized as follows:

1) X-braced frames built with commonly used slotted-bolted brace connections developed local instability in the mid-connections and end (corner) connections, instead of global buckling of the diagonal bracing members.

2) As a result of mid-connection buckling, the discontinuous braces exhibited a reduced compressive resistance compared to the continuous member. For both braces, the effective brace length was longer than half the total brace length between the hinges in the corner gusset plates.

3) The behaviour of single shear overlapping plate connections (SS) was improved by using thicker splice plates or adding stiffener plates on one side of the overlapped plates.

4) Adding shim plates to fill the slots in DS lap joints can improve the strength of the connection and lead to better performance of the whole frame, especially when the shim plates are fully welded to the splice plates.

5) All brace connections could resist the tension force intended in design. The storey drift capacity of the frames was limited, however, varying between 0.5 and 1.5% of the storey height. For Type MD CBFs, the observed storey drift capacity (1.4% and 1.1% h_s for Specimens Nos. 4 and 12, respectively) may not be sufficient. Moreover, non ductile failure occurred on net section, prior to brace gross yielding, which is unsatisfactory.

In view of the observed buckling response of the connections in X-bracing specimens with slotted connections that reflect current construction practice, it is recommended that further research be performed to better characterize the different possible connection buckling modes and develop methods to accurately predict their related compression resistances under cyclic loading conditions. Additional research is also needed to propose connection details that will eliminate connection buckling in X-braced frame structures for which ductile seismic response is required.

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REFERENCES

CSA. (2009), *CSA-S16-09, Design of Steel Structures,* Canadian Standards Association, Willowdale, ON.

Davaran, A. (2001), "Effective Length Factor for Discontinuous X-Bracing Systems". *Journal of Engineering Mechanics,* ASCE, vol. 127(106) (pp. 106-112).

Davaran, A., and Hoveidae, N. (2009), "Effect of mid-connection detail on the behavior of cross-bracing systems." *Journal of Constructional Steel Research*, vol. 65(4) (pp. 985-990).

El-Tayem, A. A., and Goel, S. C. (1986). "Effective length factor for the design of X-bracing systems". *Engineering Journal*, AISC, vol. 23(1) (pp. 41-45).

Gélinas, A., Tremblay, R. and Davaran A. (2012), "Seismic Behaviour of Steel HSS X-Bracing of the Conventional Construction Category". ASCE/SEI Structures 2012 Congress, Chicago, IL, March 29-31.

Mitchell, D., Tremblay, R., Karacabeyli, E., Paultre, P., Saatcioglu, M., and Anderson, D. L. (2003), "Seismic force modification factors for the proposed 2005 edtion of the National Building Code of Canada". *Canadian Journal of Civil Engineering*, vol. 30(2), (pp. 308-327).

NRCC. (2010), *National Building Code of Canada*, 13th ed., National Research Council of Canada, Ottawa, ON.

Picard, A., and Beaulieu, D. (1987), "Design of diagonal cross bracings. Part 1: Theoretical study". Engineering Journal, AISC, vol. 24(3) (pp. 122–126).

Tremblay, R., Archambault, M.H., and Filiatrault, A. (2003), "Seismic performance of concentrically braced steel frames made with rectangular hollow bracing members". *Journal of Structural Engineering*, ASCE, vol. 129(12) (pp. 1626-1636).