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INELASTIC PERFORMANCE OF SCREW CONNECTED LIGHT-GAUGE STEEL STRAP BRACED WALLS

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ABSTRACT

Guidelines that address the seismic design of light-gauge steel frame strap braced walls are not provided in the 2005 NBCC or in the CSA S136 Standard. A research project was undertaken to evaluate three typical screw connected 2440 x 2440 mm single-storey strap braced wall configurations with respect to their potential for resisting lateral in-plane loads in the inelastic range of behaviour. A total of 15 X braced walls, specifically designed and detailed following a capacity based approach, were tested under lateral loading using monotonic and reversed cyclic protocols. The strap braces were expected to undergo gross cross-section yielding along their length, while the other elements in the SFRS were selected to be able to carry the probable brace capacity. Based on observations and measurements made during testing a number of conclusions were drawn. An extended track detail is necessary to avoid compression failure of the top and bottom tracks. Strain rate sensitive steels cause larger than expected yield forces to occur under dynamic loading. The high strain rates also result in a change in ductility of the brace material, which in some cases can result in net section fracture of the brace instead of the expected yielding failure mode. Net section failure is not a result of damage accumulation during cyclic tests, but rather from a loss in ductility due to high strain rates. An F_u / F_y ratio of at least 1.20 is needed to ensure that screw connected strap braces do not fracture under high strain rate seismic loading. An R_y value higher than 1.2 should be considered for design.

Introduction

The use of cold-formed steel as the main framing element in a structure is becoming more popular for the construction of low to mid-rise buildings across North America, including those found in seismic areas. The installation of diagonal flat steel strap cross bracing may be a practical solution to provide for lateral strength, stiffness and stability of the structure (Fig. 1). The overall lateral strength and stiffness of this bracing system may not be related solely to the steel straps; many other elements in the lateral load carrying path play a role, such as the strap connections, the gusset plates, the anchorage including holddown and anchor rod, etc. Nonetheless, in this type of structure the straps are generally assumed to act as the fuse element in the seismic force resisting system (SFRS). Seismic design provisions for cold-formed steel structures are not provided in the NBCC (NRCC, 2005) or in the CSA S136 Standard (2001). In contrast, ASCE 7-05 (2005) allows for the use of a seismic response modification coefficient of R = 4.0

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for strap braced bearing wall systems in the US; which indicates a reliance on a moderate level of ductile / inelastic performance of the SFRS as well as some overstrength. Use of this R value necessitates that the material specific seismic design and detailing requirements of the American Iron and Steel Institute (AISI) Lateral Design Standard (2004a) and the AISI Specification (2004b) be met. Strap bracing is also to be designed in accordance with the AISI Specification or the AISI Standard on General Provisions (2001), which for the most part do not contain any relevant seismic detailing information. The AISI Standards and Specification are written in terms of strength requirements for seismic design; no mention of expected ductility requirements or recommended ductile connection / anchorage details is made. The US Army Corps of Engineers has also published a document that addresses the seismic design of cold-formed steel structures, TI 809-07 (2003), which is based in-part on the work described by Kim et al. (2006). The intent of TI 809-07 is to ensure that ductile building system performance is attained during large seismic events. The seismic design provisions in TI 809-07 are similar to those in ASCE 7-05 and the AISI Lateral Design Standard, except that additional prescriptive requirements exist, e.g. for material properties of the braces and brace / chord stud connections.

Figure 1. Cold-formed steel strap braced walls under construction.

The aim of this research project was to evaluate the inelastic lateral load carrying performance of light gauge steel frame / strap braced wall configurations, assembled using only self drilling / self tapping screws, and designed following a capacity based approach. The intent was to be able to provide guidance as to how a braced wall should be designed and detailed such that a ductile response could be achieved. The scope of study consisted of the monotonic and reversed cyclic testing of three wall configurations. The research is a continuation of previous studies on weld and screw connected strap braced walls that were designed without capacity based concepts (Al-Kharat & Rogers, 2005, 2007).

Test Program

Tests of fifteen strap braced stud wall specimens (Table 1) were carried out at McGill University using a testing frame designed specifically for in-plane shear loading (Fig. 2). The walls were 2440 x 2440 mm in size (Fig. 3) with nominal ASTM A653 (2002) Grade 230 MPa diagonal strap braces installed in an X configuration at 45^{o} on both sides. In a typical building the stud walls would be expected to carry both gravity and lateral loads. The three wall configurations can generally be referred to as light, medium and heavy construction in the cold-formed steel spectrum; that is, the expected factored lateral in-plane resistance in a wind and seismic loading situation is assumed to be 20, 40 and 75 kN, respectively. The walls were designed following a capacity based seismic design approach; all of the components in the lateral load carrying path were expected to be able to carry the force associated with the probable yield capacity of the braces without exhibiting extensive damage.

Table 1. Matrix of strap braced wall test specimens (nominal dimensions and material properties).

Specimen Properties	Test Specimens					
	Light		Medium		Heavy	
	7A-M, 7B-M[a,c]	**8A-C,8B-C[c], 8C-C[b,c]**	**9A-M 9B-M[c]**	**10A-C, 10B-C[c]**	**11A-M, 11B-M[c]**	**12A-C, 12B-C[c], 12C-C[c], 12D-C[c]**
Strap Bracing (X-brace on both sides of wall)						
Thickness (mm)	1.09		1.09		1.73	
Width (mm)	63.5		127		152	
Grade (MPa)	230		230		230	
Chord Studs (Double studs screwed together back-to-back)						
Thickness (mm)	1.09		1.37		1.73	
Dimensions (mm)	92x41x12.7		152x41x12.7		152x41x12.7	
Grade (MPa)	230		340		340	
Interior Studs (Single studs)						
Thickness (mm)	1.09		1.09		1.09	
Dimensions (mm)	92x41x12.7		152x41x12.7		152x41x12.7	
Grade (MPa)	230		230		230	
Tracks						
Thickness (mm)	1.09		1.37		1.73	
Dimensions (mm)	92x31.8		152x31.8		152x31.8	
Grade (MPa)	230		340		340	
Gusset Plates						
Thickness (mm)	NA		1.37		1.73	
Dimensions (mm)	NA		250x250		300x300	
Grade (MPa)	NA		230		230	

[a] Monotonic protocol [b] CUREE reversed cyclic protocol [c] Extended track detail used

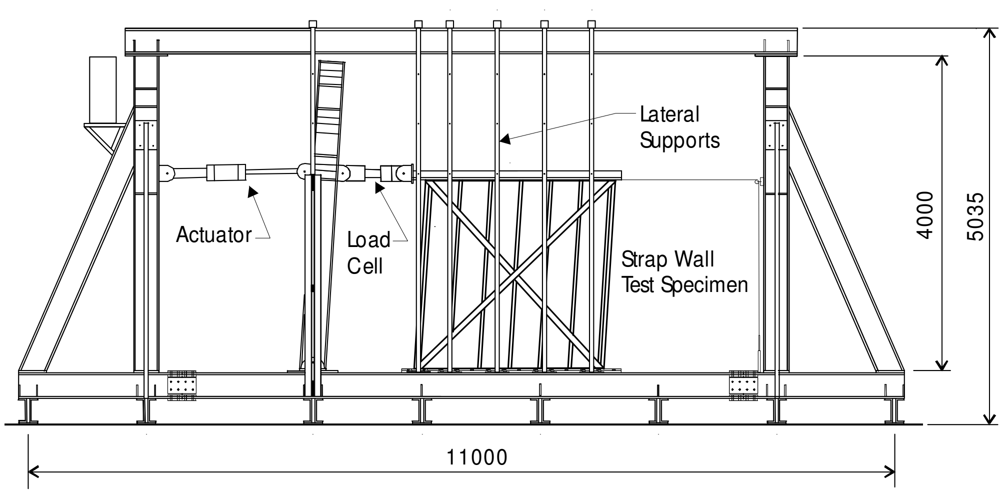

Figure 2. Schematic drawing of displaced strap braced wall specimen in test frame (mm).

The light walls were composed of 92 mm wide studs and tracks with 1.09 x 63.5 mm strap braces. The medium and heavy walls were constructed of 152 mm wide studs and tracks, along with 1.09 x 127 mm and 1.73 x 152 mm straps, respectively (Table 1). The light walls were constructed using 1.09 mm thick studs throughout, whereas for the medium walls the interior and chord studs were 1.09 mm and 1.37 mm thick, respectively; and for the heavy walls the interior and chord studs were 1.09 mm and 1.73 mm thick, respectively. Chord studs were connected back-to-back using No. 10-16 x ¾” Hex head self drilling / self taping screws. The interior studs were placed at a nominal spacing of 406 mm. All connections between the studs and tracks were made with No. 8 x ½” wafer head self drilling / self tapping framing screws. The strap braces were connected to the frame members or gusset plates with No. 10 x ¾” wafer head self drilling / self tapping framing screws. Simpson Strong-Tie S/HD10 holddown anchors were used for the light walls, and S/HD15 holddowns were installed in the medium and heavy walls.

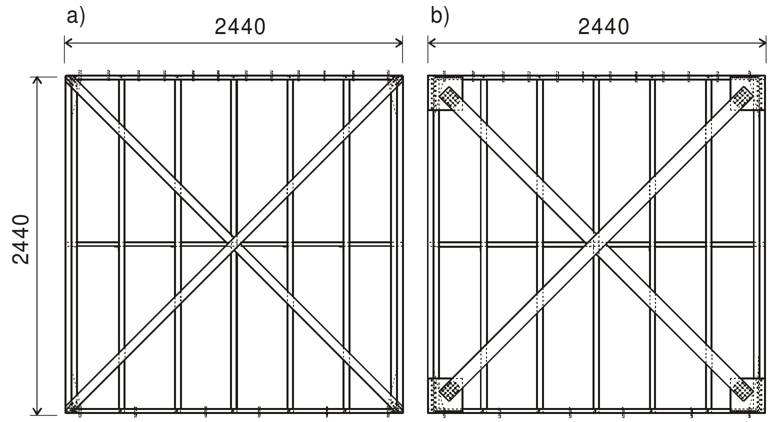

Figure 3. Schematic drawing of strap braced test walls: a) light, b) medium (mm).

Wall Design

A previous series of braced wall tests, carried out during the summer of 2004, were not designed following a capacity based approach (Al-Kharat & Rogers, 2005, 2007). Failure of these test specimens was located in the track-to-chord stud connections, where compression and punching failure of the track were observed, along with screw connection failure in the light walls. In an attempt to improve upon the behaviour of these previously tested strap braced walls, which demonstrated a general inability to maintain their yield load level for extended lateral displacements, it was decided to design the screw connected test specimens described in this paper using a capacity based approach. This approach involved the selection of a fuse element in the SFRS, followed by the design of the remaining elements such that they were able to carry the probable capacity of the fuse. In the case of braced walls it is generally assumed that the straps will act as the fuse element, and hence should be able to reach and maintain their tension yield strength throughout the duration of an earthquake. It is necessary to accurately predict the probable tensile yield strength of the strap braces and to detail the end connections such that the brace does not fail by fracture of the net section. The other elements in the SFRS are then chosen such that their capacity is greater than that of the fuse element.

The three brace sizes were first selected following the CSA S136 (2001) design provisions using factored lateral loads of 20 kN (light), 40 kN (medium) and 75 kN (heavy). The remaining elements in the lateral load carrying path were then selected given the probable capacity of the strap braces assuming that they would reach their yield load without fracturing at the screw holes. The probable nominal tension resistance (T_n) was determined using Eq. 1, which is similar in format to that found in CSA S16 (2005) for braced hot-rolled steel structures.

$$T_n = A_g R_y F_y \tag{1}$$

R_y was estimated to be 1.2 for ASTM A653 steels. The nominal gross cross-sectional area, A_g (Table 1), and the nominal yield strength, F_y = 230 MPa of the strap braces were then used to determine the probable force needed for the design of the SFRS. This probable tension force and its associated vertical and horizontal components were used in the design of the brace connections, gusset plates, chord studs, track, anchor rods, holddowns and shear anchors. A brief description of the brace and track design is provided below. A detailed description of the design steps that were taken for each of the elements is provided in Al-Kharat and Rogers (2006).

The critical step in the design process was to ensure that the braces did not fail by fracture at the net section. Once the number of screws required for each brace connection had been decided, then it was possible to specify the screw placement such that the nominal net section capacity in tension of the brace exceeded the probable nominal gross cross-section capacity, as illustrated in Eq. 2. A choice to use the nominal net section capacity, i.e. ϕ_u = 1.0, was made because the probable design force is based on the material properties of the brace itself. An increase in the tensile strength by the factor R_t = 1.2 was also considered appropriate since the yield capacity of the material had been increased in the calculation of the probable brace force. The use of the nominal net section capacity in this fashion is similar to what is found in the CSA S16 Standard for the design of moderately ductile concentrically braced frames under seismic loading (Cl. 27.5.4.2). Additional research is required to identify appropriate R_y and R_t values for the full range of sheet steel products and grades that are available in North America. The nominal material tensile strength was assumed to be F_u = 310 MPa for all the braces. Figure 4 illustrates the placement of screws for the medium capacity walls such that Eq. 2 was satisfied. The position of the first two or three lines of screws (towards the middle of the brace) is crucial to allow the brace to yield instead of fracturing. A haphazard placement of screws can lead to failure of a strap brace by net section fracture even if the appropriate number of screws has been installed. Details of the brace connections for the other walls can be found in Al-Kharat and Rogers (2006).

$$A_n R_t F_u \geq A_g R_y F_y \tag{2}$$

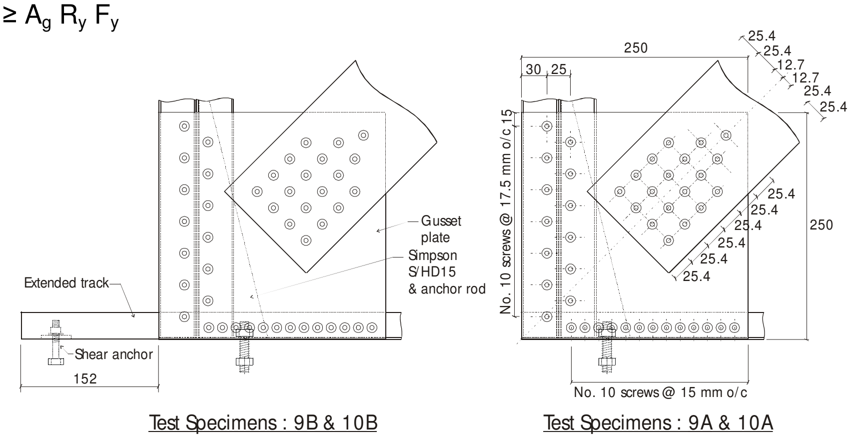

Figure 4. Schematic drawing of medium strap braced test wall corner details (mm).

The horizontal component of the brace force must be transferred through the track element to the supporting structure. The axial capacity of the track in compression and tension, as well as the bearing capacity of the track at the anchor rod and shear anchor locations were determined. The nominal capacity, ϕ = 1.0, was used in all cases due to the rarity of a design level seismic event. The compression resistance of the tracks were somewhat lower than the probable horizontal force, ranging from 0.71 to 0.89 of the capacity needed to adequately transfer the applied brace load (Al-Kharat & Rogers, 2006). Since the selected tracks did not posses adequate axial compression capacity to transfer the brace force to the supporting structure, an extended track detail was used for over half of the test specimens (Table 1, Fig. 4). The horizontal brace force is directed through the track by means of tension to the extra shear anchor that has been added outside of the wall footprint (Fig. 4). The axial capacity of the track in tension is more than adequate to carry the horizontal component of the probable brace force. The typical track detail (without an extension) was still used for each wall configuration to show that it would not be possible to satisfy the capacity based design requirement of limiting the failure to the brace element if the track thickness were selected to match the chord studs, which is a common design approach. It would have been necessary to reinforce the track, or use a thicker track section, to transfer the brace forces by means of compression. Note the track may also be subjected to a minor axis bending moment depending on the location of the holddown device and its anchor rod.

Testing

Each wall configuration was tested with both a monotonic and reversed cyclic load protocol. The monotonic procedure consisted of a steady rate of in-plane displacement (2.5 mm/min) of the top of the wall until a sudden drop in the load carrying capacity was observed or until the full travel of the actuator was reached (≈110 mm = 4.5% drift). The CUREE ordinary ground motions loading protocol (Krawinkler et al., 2000) was chosen for the reversed cyclic tests. This protocol is also specified in ASTM E2126 (2005) for the testing of walls constructed of metal framing with bracing or solid sheathing. Previous research at McGill University on light gauge steel walls incorporated this loading protocol (Al-Kharat & Rogers, 2005; Branston et al., 2006). The CUREE protocol was developed with the philosophy that multiple earthquakes may occur during the lifetime of the structure. It subjects the wall to ordinary ground motions (not near-fault) whose probability of exceedance in 50 years is 10%. The frequency of the reversed cyclic tests was kept at 0.5 Hz, except towards the end of the protocol where 0.25 Hz was used. Two additional tests 8C-C and 12D-C were run using the corresponding CUREE protocol; however the speed of testing was set at 2.5 mm/min, the same as that used for the monotonic tests. The intent was to identify the role of strain rate and damage accumulation in the performance of the walls during reversed cyclic testing.

Tests of the materials used in the construction of the walls were carried out following ASTM A370 (2002). In the case of the strap braces, these tests were run at three different cross-head rates in an attempt to evaluate the effect of an increased strain rate on the strength of the material. The coupons run at 0.6 mm/min had a similar strain rate to the monotonic wall tests, while the coupons run at 100 mm/min had a strain rate that was slightly lower than the maximum experienced by the braces in the 0.5 Hz reversed cyclic tests. The yield and tensile strengths of the braces increased as the strain rate increased (Table 2). However, the ratio of F_u / F_y was lowest for the tests run at 100 mm/min. Also note that the ratio of measured yield strength to nominal, F_y / F_{yn} was higher than 1.2 and increased with the strain rate. Properties of the other components of the braced walls are provided by Al-Kharat and Rogers (2006).

Table 2. Measured material properties of strap braces.

Wall	**Base Metal Thickness (mm)**	**F_y (MPa)**	**F_u (MPa)**	**F_u / F_y**	**% Elong.**	**F_y / F_{yn}**	**Test Speed (mm/min)**
7A-M,7B-M,8A-C, 8B-C,8C-C	1.16	305	358	1.17	31.4	1.33	0.6
	1.16	358	392	1.09	32.1	1.56	50
	1.15	362	400	1.11	33.4	1.57	100
9A-M,9B-M,10A-C, 10B-C	1.06	303	363	1.20	29.1	1.32	0.6
	1.06	320	374	1.17	31.9	1.39	50
	1.06	325	378	1.16	33.0	1.41	100
11A-M,11B-M,12A-C, 12B-C,12C-C, 12D-C	1.80	300	346	1.15	42.3	1.30	0.6
	1.80	317	357	1.13	38.5	1.38	50
	1.80	324	361	1.11	39.2	1.41	100

Modes of Failure

In general, the overall performance of the walls with extended tracks subjected to monotonic lateral loading was governed by the yielding of the straps, and no decrease in the wall resistance was noticed up to the 4.5% drift level. The monotonically tested walls constructed with the regular length tracks did show some yielding of the braces, however they typically suffered from compression failure of the track and/or chord stud and bearing failure of the track at the anchor rod location (Fig. 5). The reversed cyclic tests with the regular length tracks also exhibited failure of the track member. The reversed cyclic tests with extended tracks failed in two modes; straps in tests 8B-C and 12B-C fractured at the net section, while the straps in test 10B-C yielded along their length as was observed for the related monotonic test (9B-M) (Fig. 5). Test 10B-C exhibited the ideal behaviour that would be anticipated when a capacity based design approach is implemented. The two reversed cyclic tests that were run at the monotonic loading rate generally showed yielding of the braces. The holddowns were able to transfer the uplift forces in all cases. The unfavourable track and strap net section modes of failure reduced the ductility and energy absorption ability of the SFRS in comparison to the walls in which inelastic deformations were limited to yielding of the braces. A detailed description of the failure modes for each wall configuration is provided in Al-Kharat and Rogers (2006).

Figure 5. Track compression/bearing, brace yielding and net section fracture failure modes.

Measured Performance

The strength, S_y, and stiffness, K_e, of each braced wall was first determined from the test results. Also calculated was the ductility, μ, defined as the ratio of $\Delta_{0.8} / \Delta_y$, where $\Delta_{0.8}$ is the post peak displacement at $0.8S_y$ or the maximum displacement if no reduction in capacity occurred, and Δ_y is the displacement at first yield. Two strength parameters were calculated for comparison purposes; S_{yp} is the predicted yield strength of the wall based on the measured strap dimensions and properties, and S_{yn} is the predicted nominal yield strength of the wall based on nominal dimensions and properties (Table 1). As well, the predicted stiffness, K_p, was calculated based on the measured dimensions and properties of the straps alone, as explained in Al-Kharat and Rogers (2006). A summary of the test results and the predicted wall design properties is provided in Tables 3 and 4.

Table 3. Summary of monotonic test results.

Wall	S_y (kN)	K_e (kN/mm)	Ductility (μ) (mm/mm)	$\Delta_{0.8}$ (mrad)	Energy (kN-mm)	S_{yp} (kN)	K_p (kN/mm)	S_y/S_{yp}	S_y/S_{yn}
7A-M	32.2	2.64	6.15	73.8	1999	31.7	4.52	1.02	1.43
7B-M[a]	33.5	3.25	11.9	115	3600	31.6	4.52	1.06	1.49
9A-M	59.4	4.38	6.43	84.8	4515	57.9	8.32	1.03	1.32
9B-M[a]	59.7	4.49	8.57	110	6035	57.9	8.33	1.03	1.33
11A-M	107.3	5.94	5.05	98.8	7525	116.2	16.9	0.92	1.25
11B-M[a]	116.3	6.95	6.74	113	11685	116.4	16.9	1.00	1.36

[a] Extended track detail used

All the strap braces showed some amount of yielding; although, in a number of cases severe damage to the non-fuse elements was observed. The monotonically tested walls with the regular length tracks were able, except test 11A-M, to reach the predicted yield strength, S_{yp}, (Table 3), however the ductility levels were lower than the walls with extended tracks due to the track failure (Fig. 6a). The use of the extended track detail allowed for the damage to be limited to the brace members because the horizontal component of the load associated with the yielding of the straps could be transferred through the tracks my means of tension. The extended track detail was advantageous for all monotonic specimens, which were able to reach the predicted yield force and maintain this load level for the duration of the test protocol (Fig. 6b). The similarly constructed reversed cyclic specimens tested at 0.5 Hz (8B-C, 12B-C, 12C-C) showed that strain rate effects may lead to the failure of the braces by net section fracture, a less ductile failure mode (Fig. 7b). The cyclic tests constructed with 2.44 long tracks (8A-C, 12A-C) were usually not able to develop the yield strength of the braces (Fig. 7a); however they showed better ductility levels than the walls with extended tracks because the braces did not fracture (Table 4). Reversed cyclic tests of the light and heavy walls at a loading rate equivalent to the monotonic tests (8C-C, 12D-C) revealed that yielding of the braces over their length could be achieved even when the walls were subjected to repeated inelastic displacements (Fig. 8b). The brace fractures in the cyclic tests run at 0.5 Hz can be attributed to the increased strain rate used for testing instead of an accumulation of damage.

Table 4. Summary of reversed cyclic test results.

Wall	S_y (kN)	K_e (kN/mm)	Ductility (μ) (mm/mm)	$\Delta_{0.8}$ (mrad)	Energy (kN-mm)	S_{yp} (kN)	K_p (kN/mm)	S_y/S_{yp}	S_y/S_{yn}
8A-C(+ve)	35.3	2.52	4.11	61.1	7320	37.5	4.51	0.94	1.57
8A-C(-ve)	34.9	2.37	5.45	86.3	7320	37.5	4.51	0.93	1.55
8B-C(+ve)[a]	38.4	2.92	1.68	21.5	3134	37.4	4.50	1.03	1.71
8B-C(-ve)[a]	38.2	2.68	1.64	22.8	3134	37.4	4.50	1.02	1.70
8C-C(+ve)[a]	33.1	2.94	10.4	112	7041	31.6	4.52	1.05	1.47
8C-C(-ve)[a]	36.3	2.96	2.42 (10.6)[b]	25.8 (113)[b]	7041	31.6	4.52	1.15	1.61
10A-C(+ve)	67.6	4.16	7.03	105	15870	61.9	8.31	1.09	1.50
10A-C(-ve)	63.7	3.59	6.36	110	15870	61.9	8.31	1.03	1.41
10B-C(+ve)[a]	69.0	4.24	7.30	107	16874	62.0	8.33	1.11	1.53
10B-C(-ve)[a]	69.1	4.06	7.26	111	16874	62.0	8.33	1.11	1.53
12A-C(+ve)	118.8	7.20	5.56	97	23897	125.7	16.9	0.95	1.39
12A-C(-ve)	105.2	6.79	6.32	117	23897	125.7	16.9	0.84	1.23
12B-C(+ve)[a]	137.3	8.08	2.92	45.5	16356	125.9	16.9	1.09	1.60
12B-C(-ve)[a]	135.0	6.28	2.44	49.0	16356	125.9	16.9	1.07	1.57
12C-C(+ve)[a]	137.5	7.24	2.64	45.9	18172	125.8	16.9	1.09	1.60
12C-C(-ve)[a]	140.9	6.96	3.16	57.2	18172	125.8	16.9	1.12	1.64
12D-C(+ve)[a]	112.8	6.36	5.99	110	25548	116.4	16.9	0.97	1.32
12D-C(-ve)[a]	110.0	5.94	5.78	113	25548	116.4	16.9	0.94	1.28

[a] Extended track detail used [b]Separate values provided for the two braces; one of which fractured.

The strap braces of the medium wall that was tested with the 0.5 Hz reversed cyclic protocol (10B-C) did not fracture during testing, which was a different failure mode compared with both the light and heavy walls. The braces of the medium wall were able to reach and maintain their yield plateau in all cycles of the protocol (Fig. 8a). Note, even though the thickness of the braces for the light and medium walls was the same, the material was obtained from two different coils. In terms of predicting whether a screw connected strap brace will yield or fracture under tension loading, the ratio of F_u / F_y was found to be an important parameter of the strap material, which needs to be considered in design. An F_u / F_y ratio of 1.16 at a cross head speed of 100 mm / min (Table 2) (strain rate of $\varepsilon = 20.8 \times 10^{-3}\ s^{-1}$) was determined for the brace coupons for the medium walls. In comparison, the coupons for the braces of the light and heavy walls were only able to reach an F_u / F_y ratio of 1.11 at the same strain rate. The wall and coupon tests completed for this research project indicate that if high strain rate loading is expected, as would be the case for seismic actions, and a ductile response of the brace and wall is sought, then an F_u / F_y ratio of at least 1.16 is necessary for the brace material if the coupon tests are carried out at a strain rate of $20 \times 10^{-3}\ s^{-1}$ or higher. The F_u / F_y ratio would need to be greater than 1.20 if the coupons were completed at a slower strain rate; in the case of this test program a slow strain rate of $0.125 \times 10^{-3}\ s^{-1}$ was used.

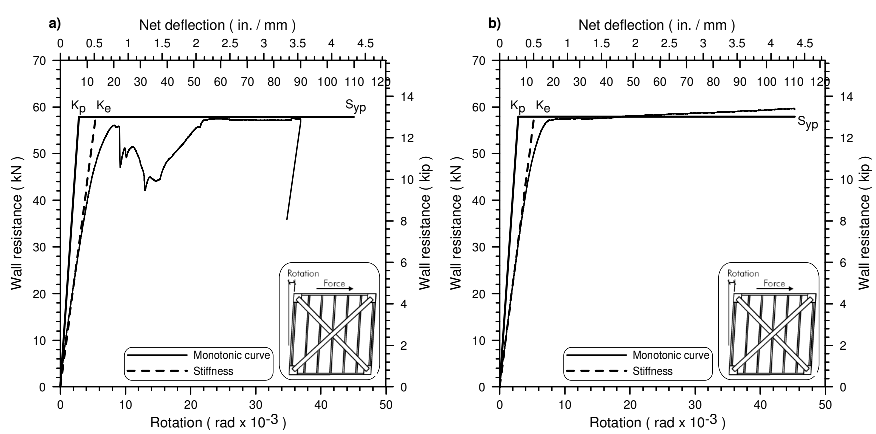

Figure 6. Monotonic resistance versus displacement curve of strap braced walls: a) 9A-M, b) 9B-M.

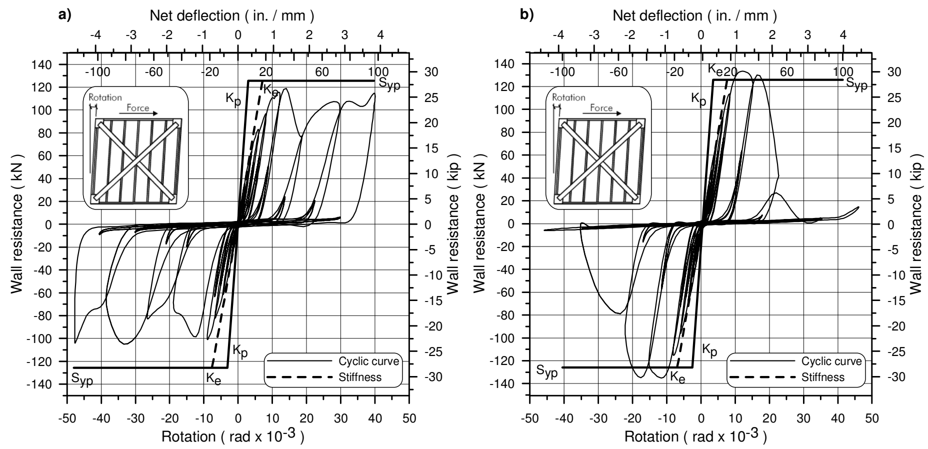

Figure 7. Cyclic resistance versus displacement curve of strap braced walls: a) 12A-C, b) 12C-C.

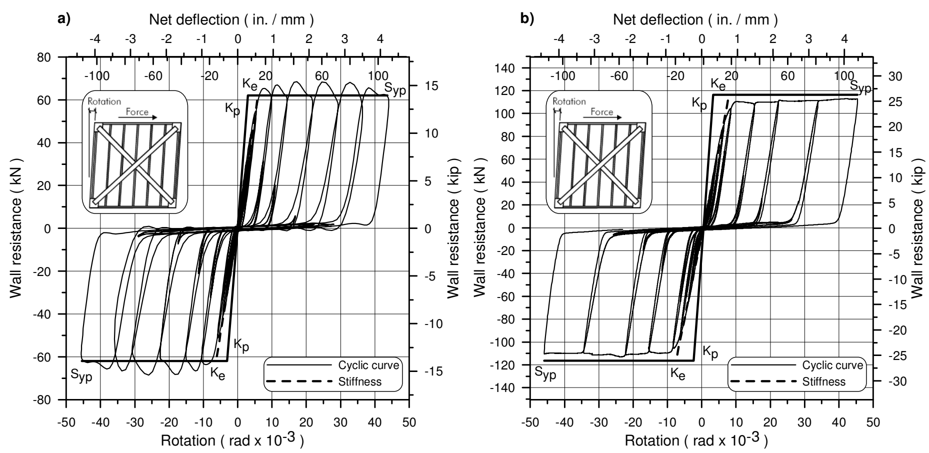

Figure 8. Cyclic resistance versus displacement curve of strap braced walls: a) 10B-C, b) 12D-C.

The higher ultimate tensile strength of the material allows for the steel along the length of the brace to yield while the steel at the net section strain hardens without fracturing. It may be more reasonable to use the F_u / $F_y \geq 1.20$ for brace material tested at a slow strain rate, which is within the ASTM A370 testing guidelines. Both the high and slow strain rate F_u / F_y ratios are substantially higher than that presently required by the North American Specification for the Design of Cold-Formed Steel Structural Members (CSA, 2001; AISI 2004b), where F_u / F_y need only exceed 1.08.

Conclusions

In general, strap braced walls do have the potential to reach and maintain their yield strength in the inelastic range of deformations if a capacity based design approach is implemented. Nonetheless, there are modifications that need to be made to the design approach to ensure that yielding failure of the brace members occurs. The cold-formed steel used for the straps showed to be strain rate sensitive. Brace yielding behaviour was observed for the monotonic wall tests with extended tracks, however, the 0.5 Hz reversed cyclic walls 8B-C, 12B-C and 12C-C experienced fracture of the braces, most likely due to the reduced F_u / F_y ratio that is associated with higher strain rates. Test 10B-C did not suffer from brace fracture, and hence, it is recommended that an F_u / $F_y \geq 1.2$ requirement for the strap material be implemented if yielding of screw connected braces under seismic loading is desired. The typical (non-extended) track detail was found to be inadequate if the designer wishes to limit the inelastic deformations to the brace members. The extended track detail should be used or a reinforcement scheme for the track would need to be implemented to improve its compression and bearing capacity. The estimate of the probable brace force using $R_y = 1.2$ is lower than what was obtained from the coupon tests. The F_y / F_{yn} ratios ranged from 1.30 to 1.57 depending on the strain rate used in testing and the steel thickness. It is recommended that a survey of the material properties of steel coils available across North America be carried out to better define the increase in probable F_y and F_u values compared with the minimum specified design values. The increased brace loads that can be achieved due to high strain rates should also be accounted for in the definition of R_y and R_t values.

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