Oct 26th, 12:00 AM

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Eighteenth International Specialty Conference on Cold-Formed Steel Structures
Orlando, Florida, U.S.A, October 26 & 27, 2006

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Abstract

Currently, design guidelines for laterally loaded (wind & seismic) light gauge steel frame / wood panel shear walls are not available in Canadian codes. It is anticipated that the construction of buildings which incorporate these shear walls as primary lateral load resisting elements will increase across Canada in coming years. This includes sites that have a relatively high seismic hazard, such as found along the West Coast of British Columbia and in the Ottawa and St. Lawrence River Valleys. An increase in the probability that a light gauge steel frame structure will be subjected to the demands of a severe earthquake will likely accompany this rise in construction activity. A research program with the main objective of developing a shear wall design method for use in Canada has been underway since 2001. The most recent phase of this research was carried out on shear walls sheathed with 9 mm OSB panels. A series of 18 walls was tested to expand upon the previous phases of the research program in which design information was developed for shear walls constructed with thicker OSB and plywood panels. The equivalent energy elastic-plastic (EEEP) analysis technique was implemented in the calculation of design parameters, which include; nominal shear strength, elastic stiffness and system ductility. The calibration of a resistance factor (ϕ) for use with the limit states design philosophy consistent with the 2005 National Building Code of Canada (NBCC) was carried out. It was determined that a resistance factor of 0.7 provided

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sufficient reliability and a reasonable factor of safety under the NBCC wind loading case. This paper describes the test program, EEEP analysis approach and calibration procedure. As well, nominal strength and unit elastic stiffness values for use in design are provided according to typical perimeter fastener schedules.

Introduction

Walls constructed of light gauge steel members are often used to carry gravity loads and can also be designed and used as shear walls. Shear walls transmit in-plane lateral forces due to wind or earthquake loads from the upper storey(s) of a building to the foundation. In order to develop a resistance to these lateral forces, the steel framing can be sheathed with a structural member such as oriented strand board (OSB) or plywood panels. The wood panels are fixed to the light gauge steel frame by means of screws, the size and number of which typically dictate the stiffness and the shear resistance of the wall. It is also necessary to attach the wall, by means of shear anchors and hold-downs, to the supporting foundation or to the lower wall segments in a multi-storey building. The use of light gauge steel framing is becoming more popular; in Canada however, there presently are no standards or codes to design shear walls constructed of a steel frame and wood sheathing. It is for this reason that light gauge steel frame / wood panel shear walls have been the subject of a research program at McGill University since 2001. The overall goal of the research is to develop a design method for light gauge steel frame / wood panel shear walls that can be used in conjunction with the 2005 National Building Code of Canada (NBCC) *(NRCC, 2005)*. The research undertaken at McGill University in previous years consisted mainly of the physical testing of single-storey shear walls under monotonic and reversed cyclic loading *(Boudreault, 2005; Branston, 2004; Branston et al., 2006a; Chen, 2004; Chen et al., 2006)*. Different wall configurations were used for testing, for which the following were varied; fastener schedule, wall length, as well as sheathing type and thickness. A method to evaluate the test data, that relies on the equivalent energy elastic-plastic (EEEP) analysis approach *(Park, 1989; Foliente, 1996; ASTM E2126, 2005)*, was then developed to determine design parameters for wind and earthquake loadings *(Branston, 2004; Branston et al., 2006b)*. This method incorporates Canadian limit states design philosophy, as documented in the 2005 NBCC, and accounts for the use of Canadian construction products. To date, the shear wall research project has focused on walls constructed of 11 mm (7/16") OSB and 12.5 mm (1/2") plywood panels. In construction it is not uncommon to use thinner sheathing, *i.e.* 9 mm OSB and 9.5 mm (3/8") plywood panels, for which shear wall design information has yet to be made available. Research projects by Blais *(2006)* and Rokas *(2006)* were carried out to provide design information for shear walls sheathed with these thinner panels.

Blais *(2006)* was responsible for a research project in which the scope of study included the monotonic and reversed cyclic testing of eighteen single-storey light gauge steel frame / wood panel shear walls (three configurations). The test program was defined such that it would augment the database of test results for existing light gauge steel frame / wood panel shear walls subject to lateral earthquake and wind loading. The wall specimens were constructed with 1.09 mm (0.043") thick light gauge steel frames and 9 mm (3/8") thick oriented strand board (OSB) sheathing, which was attached with screws at a spacing of 75, 100 and 152 mm (3", 4" and 6") over the panel perimeter. The resulting test data was then used to establish design parameters following the EEEP analysis approach. The results presented and values proposed in this paper are limited to individual 1220 × 2440 mm (4′ × 8′) light gauge steel frame / wood panel shear walls designed to resist lateral in-plane loading.

Test Program

All wall specimens were built and tested in a similar fashion to the shear walls included in previous studies by Branston *(2004)*, Chen *(2004)* and Boudreault *(2005)*. The test matrix included wall specimens that were 2440 mm (8′) in height and 1220 mm (4′) in length (Table 1). Each wall was composed of light gauge steel studs (92.1 × 41.3 × 12.7 mm (3-5/8" × 1-5/8" × 1/2")), spaced at 610 mm (24") o.c., and light gauge steel tracks (92.1 × 31.8 mm (3-5/8" ×1-1/4")), which were rolled from 1.09 mm (0.043") ASTM A653 *(2002)* Grade 230 (33 ksi) steel. No. 8 × 1/2" (12.7 mm) long wafer head self-drilling screws were used to connect the framing members. The 9 mm (3/8") OSB sheathing *(CSA O325, 1992)* was attached to one side of the steel frame with No. 8 × 1-1/2" (38.1 mm) long Grabber SuperDrive bugle head sheathing screws at 75 mm (3"), 100 mm (4") and 152 mm (6") spacing around the panel perimeter. A screw spacing of 305 mm (12") was used to connect the sheathing to the inner stud. Simpson Strong-Tie S/HD10 hold-downs connected the back-to-back chord studs to the test frame. Each of the three wall configurations consisted of six specimens, three of which were tested monotonically and three cyclically using the CUREE protocol for ordinary ground motions *(Krawinkler et al., 2000; ASTM E2126, 2005)*. More detailed information on the test program can be found in Blais *(2006)*.

Table 1 : Matrix of shear wall tests

Specimen	Protocol	Wall Length (mm)	Wall Height (mm)	Fastener[1] Schedule (mm)
41– A,B,C	Monotonic	1220	2440	152/305
42– A,B,C	CUREE	1220	2440	152/305
43– A,B,C	Monotonic	1220	2440	100/305
44– A,B,C	CUREE	1220	2440	100/305
45– A,B,C	Monotonic	1220	2440	75/305
46– A,B,C	CUREE	1220	2440	75/305

[1]Fastener schedule (e.g. 75/305) refers to the approx. spacing in mm between the sheathing to framing screws on the panel perimeter and along the intermediate studs (field spacing), respectively.

A specially constructed reaction frame, with a 250 kN (55 kip) capacity dynamic actuator having a displacement range of ± 125 mm (5O), was used to test each shear wall under stroke control while measuring the resistance and relevant deformations. A schematic of the test frame and photographs of typical wall specimens are provided in Figures 1 & 2. The behaviour of each shear wall was monitored throughout testing by means of measured loads, displacements and accelerations. In all, eleven transducers (LVDTs) were directly connected to the wall specimen measuring the uplift (2 LVDTs) and slip (2 LVDTs) at bottom corners, the in-plane lateral wall displacement (1 LVDT) and the displacement of the wood panel relative to the wall frame (4 LVDTs). In addition, two LVDTs were installed to measure the shear deformation of the wood sheathing. An accelerometer and three load cells were also used to monitor the wall. Two of the load cells were installed at the hold-down rods while the other was positioned in the loading beam assembly. The accelerometer, which was attached to of the main load cell, was relied on to measure the acceleration at the top of the wall during reversed cyclic tests. All of the measuring devices were connected to Vishay Model 5100B scanners to record data. Vishay System 5000 StrainSmart software was used to control the data acquisition system.

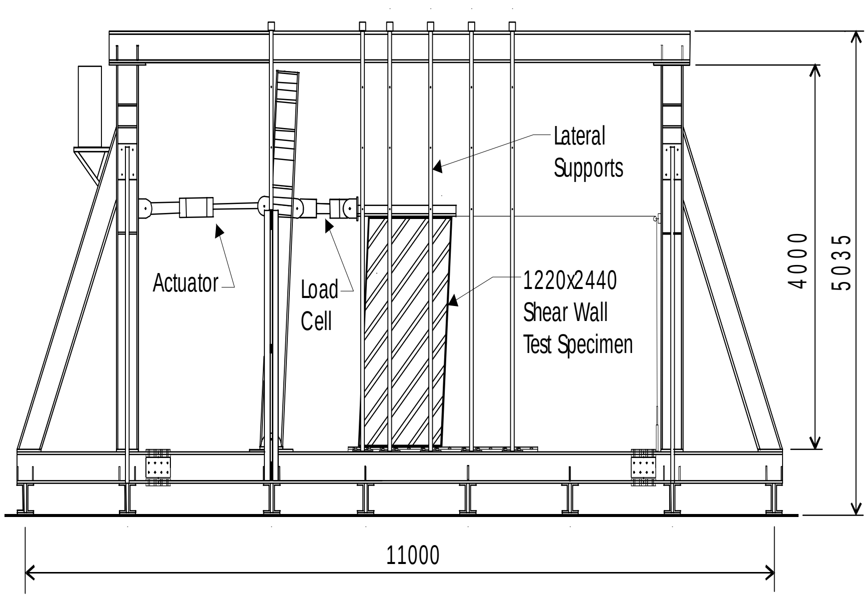


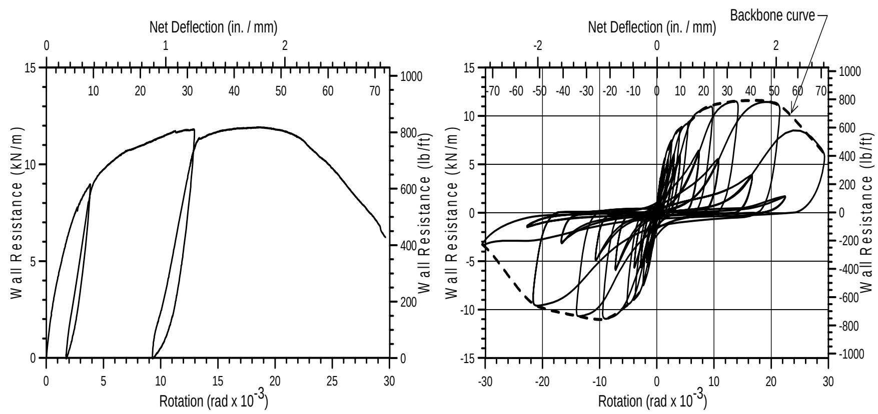
Figure 1 : Schematic of test frame with 1220 × 2440 mm (4′ × 8′) wall specimen

Figure 2 : Photographs of test frame and shear wall specimen

General Test Results

Typical monotonic and reversed cyclic test curves are illustrated in Figure 3. A backbone curve, which was based on the maximum force level, and in some cases deformation level, recorded during each of the displacement cycles, is also shown for the cyclic test. The average test results for the monotonic tests, as well as for the positive and negative cycles of the reversed cyclic tests have been provided in Tables 2 & 3, respectively. The following data is listed for each wall configuration; maximum wall resistance (S_u), displacement at $0.4S_u$ ($\Delta_{net,0.4u}$), displacement at S_u ($\Delta_{net,u}$), displacement at $0.8S_u$ ($\Delta_{net,0.8u}$), rotation at S_u ($\theta_{net,u}$), rotation at $0.8S_u$ ($\theta_{net,0.8u}$) and energy dissipation (E_r). All displacement measurements and wall resistance values (cyclic tests only) have been modified for slip and uplift of the test wall as well as accelerations *(Branston, 2004)*. Note, that for the reversed cyclic tests, this data was obtained from the backbone curve. A more comprehensive description of the shear wall test results, including force *vs.* deformation graphs, test data sheets and test observations can be found in Blais *(2006)*.

The general test results reveal that the ultimate shear wall resistance measured for the cyclic tests is lower than that obtained for the monotonic tests. This decrease in strength is due to the repetitive motion of the reversed cyclic protocols which caused an accumulation of damage at the sheathing connection locations. Walls with a screw schedule of 75/305 mm (3"/12") tested cyclically exhibited an ultimate strength that was approximately 11.6 % lower than measured for walls tested monotonically. Walls with screws spaced at a greater distance, *i.e.* 152/305 mm (6"/12"), exhibited only a 4.2 % decrease in ultimate strength. The decrease in S_u that was observed for these 18 tests is in the same range as that obtained in previous testing of shear walls using the CUREE reversed cyclic protocol *(Chen, 2004; Chen et al., 2006).*



(a) Monotonic test curve

(b) Cyclic test and backbone curves

Figure 3 : Typical monotonic and reversed cyclic (with backbone) test curves

Table 2 : General monotonic test results (average values)

Specimen	**S_u (kN/m)**	**$\Delta_{net,0.4u}$ (mm)**	**$\Delta_{net,u}$ (mm)**	**$\Delta_{net,0.8u}$ (mm)**	**$\theta_{net,u}$ (rad x 10^{-3})**	**$\theta_{net,0.8u}$ (rad x 10^{-3})**	**Energy (E_r) (joules)**
41–A,B,C	12.0	2.7	41.6	55.6	17.1	22.8	701
43–A,B,C	18.4	4.0	41.1	50.4	16.9	20.6	910
45–A,B,C	24.2	5.0	41.5	48.4	17.0	19.8	1130

Table 3 : General reversed cyclic test results (average values)

Specimen	S_u (kN/m)	$\Delta_{net,0.4u}$ (mm)	$\Delta_{net,u}$ (mm)	$\Delta_{net,0.8u}$ (mm)	$\theta_{net,u}$ (rad x 10^{-3})	$\theta_{net,0.8u}$ (rad x 10^{-3})	Energy (E_r) (joules)
42– A,B,C (Pos.)	11.5	3.1	33.0	57.7	13.5	23.7	3321
42– A,B,C (Neg.)	-11.2	-3.1	-22.0	-49.1	-9.0	-20.1	
44– A,B,C (Pos.)	17.2	4.2	45.8	54.7	18.8	22.4	4489
44– A,B,C (Neg.)	-16.1	-3.4	-30.5	-51.9	-12.5	-21.3	
46– A,B,C (Pos.)	21.4	4.0	34.0	48.1	14.0	19.7	4687
46– A,B,C (Neg.)	-20.3	-3.5	-30.6	-42.1	-12.5	-17.3	

As the walls were loaded cyclically some cumulative wood crushing occurred at the connections, which decreased the wall resistance of the successive cycle. The same phenomenon explains the lower shear resistance of the walls during the negative cycles since the positive cycles were executed first in the protocol. Both the shear resistance and energy values increased as the screw spacing distance decreased, which was expected given the information provided by Chen *(2004)* and Chen *et al. (2006)*. Another important observation is that the energy dissipation values obtained for the two testing protocols are very different. In terms of the monotonic tests the energy is equal to the area underneath the resistance *vs.* displacement curve, while for the cyclic tests the energy is determined based on the area enclosed by every loop in the protocol. Therefore the energy computed for a cyclic test is cumulative, and hence is much larger than found for a monotonic test since the loops are partially superimposed. The energy dissipation could only be directly compared if the backbone curve were used for the evaluation of cyclic test data.

Ancillary Materials Tests

The light gauge steel studs and tracks were tested to determine their material properties following the ASTM A370 Standard *(2002)*. The studs and tracks were rolled from the same coil of steel, hence, only one set of static material properties is provided. Table 4 lists the base metal thickness, the yield stress (F_y) the ultimate stress (F_u) and the modulus of elasticity (E), as well as the percent elongation over a 50 mm gauge length and the ratio of F_u to F_y. The material property requirements of the North American Specification for the Design of Cold-Formed Steel Structural Members *(CSA S136, 2001; AISI, 2001)* were met; this includes the ratio $F_u / F_y \geq 1.08$ and the minimum 10 % elongation over a 50 mm gauge length.

Table 4 : Measured material properties of the steel framing

Specimen	Member	Base Metal Thickness (mm)	F_y (MPa)	F_u (MPa)	F_u / F_y	E (GPa)	% Elong. 50 mm Gauge
1.09 mm 230 MPa	Stud & Track	1.12	264	345	1.30	199	31.5

Ancillary tests for the wood sheathing were carried out following the edgewise shear test prescribed in ASTM Standard D1037 *(1999)* Sections 130 to 136. Six OSB specimens of 254 × 90 mm (10O × 3.5O) in size, three of which were aligned parallel to the grain of the outermost strands and three of which were perpendicular to the strands, were used for the tests. Information regarding the ultimate shear strength, the shear modulus and the rigidity of the 9 mm OSB wood panels is provided in Table 5. The values shown are based on the average of the results for the parallel and perpendicular experimental data. This approach was taken because the results were similar for the OSB specimens in the two directions.

Table 5 : Measured material properties of the wood sheathing

Specimen	Thickness (mm)	Ultimate Shear Strength (MPa)	Shear Modulus (MPa)	Rigidity (N/mm)
9 mm OSB	9.27	4.52	1096	10148

Development of a Limit States Design Procedure

Based on a review of existing design methods for shear walls, as well as data interpretation procedures for non-linear testing, a choice was made to incorporate the Equivalent Energy Elastic-Plastic (EEEP) bilinear model *(Park, 1989; Foliente, 1996)* into the evaluation of all monotonic and reversed cyclic test data *(Branston, 2004; Branston et al., 2006b)*. A codified version of the EEEP approach to calculating the design parameters of light framed shear walls can also be found in ASTM E2126 *(2005)*.This data interpretation method was selected because it provides basic strength and stiffness information that can be used for design, it gives a measure of the ductility inherent in the shear wall, which is needed to define a test based force modification factor for seismic design, it can be applied irrespective of the loading protocol implemented, and because it has historically been used for the analysis of other structural systems that exhibit a non-linear resistance *vs.* deflection behaviour *(Branston, 2004)*. It was also necessary for the resulting design method to provide a simplification of the typical non-linear response demonstrated by light framed shear wall systems under lateral loading. The model results in an idealized load-deflection curve, of a simple bilinear shape, that can be easily defined and constructed, yet still provides a realistic depiction of the data obtained from laboratory testing based on a dissipated energy perspective. In the case of each reversed cyclic test a backbone curve was first constructed for both the positive and negative displacement ranges of the resistance *vs.* deflection hysteresis. The resistance *vs.* deflection curve for monotonic specimens and the backbone curves for cyclic tests were then used to create EEEP curves based on an equivalent energy approach, as illustrated in Figure 4.

The EEEP curve for each specimen was constructed by first determining three main parameters from the monotonic/backbone curve, including resistances: S_u, $S_{0.4u}$ and $S_{0.8u}$ (post-peak), and all matching displacements: $\Delta_{net,u}$, $\Delta_{net,0.4u}$, and $\Delta_{net,0.8u}$ (post-peak) (Figure 4). Due to the non-linear behaviour of the walls, a straight line passing through the origin and the $S_{0.4u}$ - $\Delta_{net,0.4u}$ position was relied on to define the elastic portion (K_e) of the bilinear EEEP curve. The 40% resistance level was considered to be a reasonable estimate of the maximum service load level. The area (energy) under the backbone curve was then calculated up to the post-peak displacement that corresponds to a wall resistance of $S_{0.8u}$, as described in ASTM E2126. This load level was considered to be the limit of the useful capacity of each shear wall and represents the failure point of a specimen. A horizontal line depicting the plastic portion of the EEEP curve was then positioned so that the area bounded by the EEEP curve, the x-axis, and the limiting displacement ($\Delta_{net,0.8u}$) was equal to the area below the observed test curve, or similarly, $A_1 = A_2$ as is indicated in Figure 4. The plastic portion of the bilinear curve was set as the wall nominal shear strength (S_y). This procedure was also followed for the negative displacement region of the reversed cyclic tests.

The 2005 National Building Code of Canada requires that for seismic design inelastic lateral deflections be limited to 2.5% of the storey height for buildings of normal importance. For a 2440 mm (8′) high shear wall this translates into an inelastic inter-storey drift limit of 61 mm (2.4O). There are two cases where the EEEP analysis of a light gauge steel frame / wood panel shear wall could be influenced by the inelastic drift limit: Case I: 61 mm < $\Delta_{net,u}$ and Case II: $\Delta_{net,u}$ < 61 mm < $\Delta_{net,0.8u}$. For Case I the equivalent energy up to the drift limit was considered, whereas for Case II the energy up to the displacement corresponding to $S_{0.8u}$ (post-peak) was evaluated. In the Case II situation, a restriction on the design capacity was not necessary and no modification to the EEEP curve procedure detailed above was utilized. Based on the data recorded for the 18 tests described in this research, the average lateral displacement $\Delta_{net,0.8u}$ for all monotonic and cyclic tests was found to be below the 2.5% drift limit (Tables 2 & 3). Therefore, Case I and Case II did not apply, and hence, the EEEP general procedure was implemented to obtain the final design parameters. EEEP bilinear curves for a representative monotonic and reversed cyclic test are illustrated in Figures 5 and 6, respectively. A typical family of test, backbone and EEEP curves for the six tests within a particular wall configuration is provided in Figure 7.

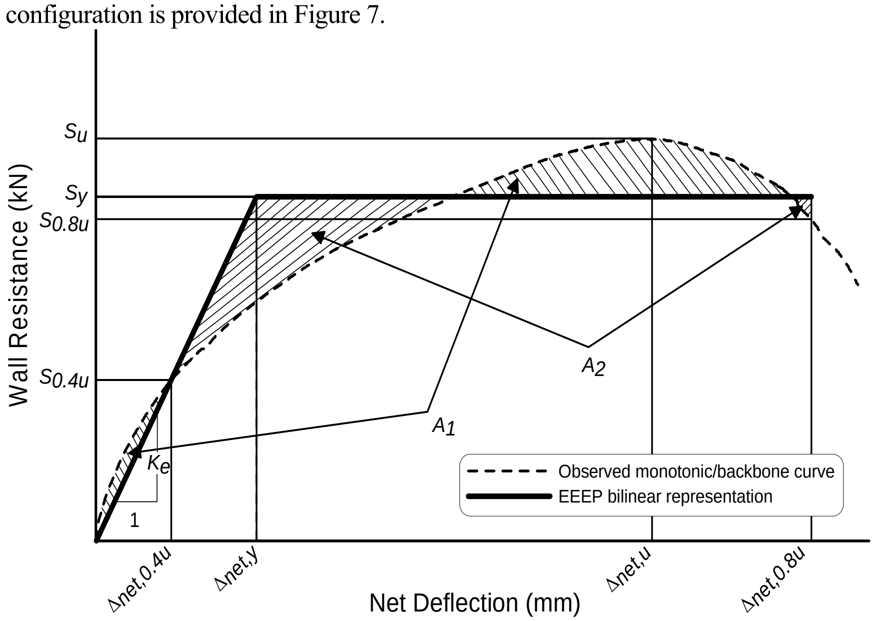


Figure 4 : Equivalent energy elastic-plastic (EEEP) analysis model *(Park, 1989; Foliente, 1996; ASTM E2126, 2005)*

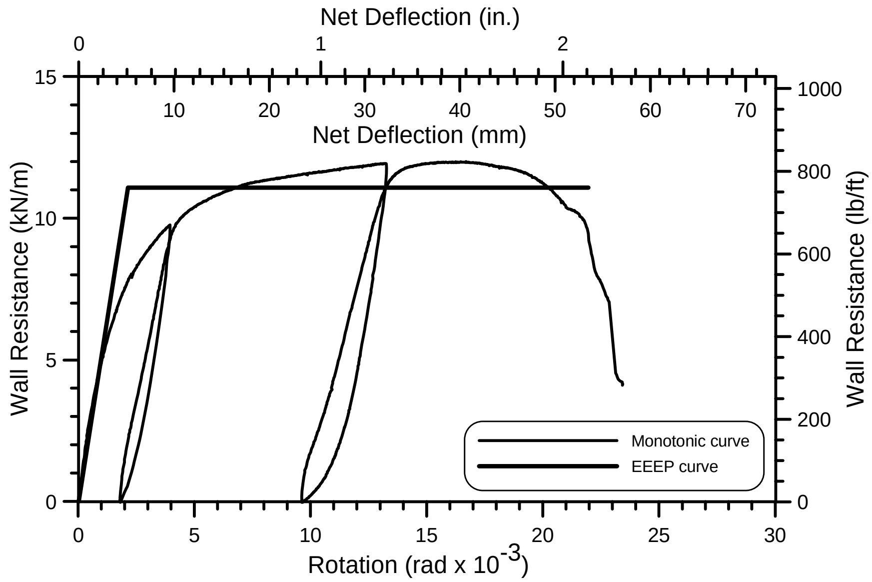


Figure 5 : Typical shear resistance *vs.* rotation of monotonic test wall with EEEP curve

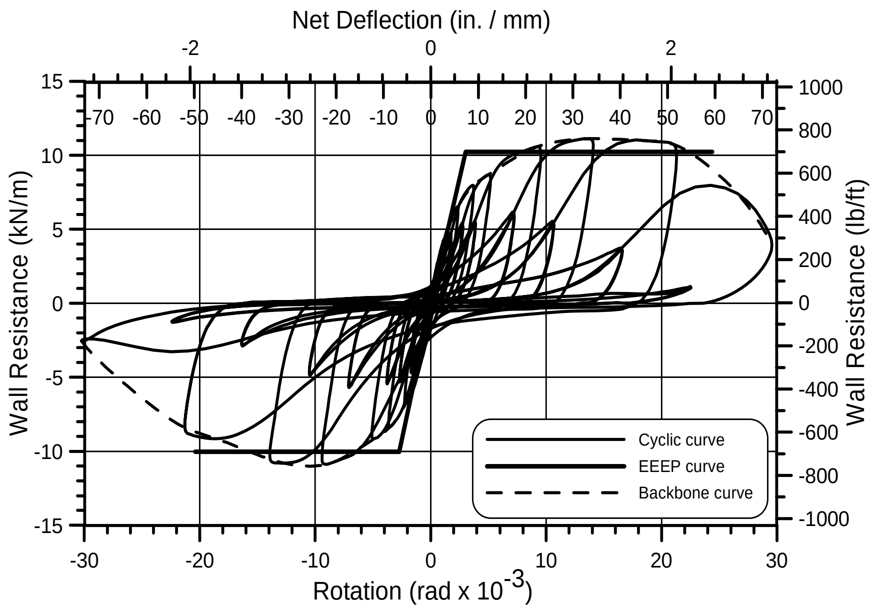


Figure 6 : Typical shear resistance *vs.* rotation of reversed cyclic test wall with EEEP curve

Based on the EEEP analysis approach nominal shear strength (S_y) and elastic stiffness (K_e) parameters were obtained for each test specimen, as well as ductility (μ) and energy (E_b) measures. Average values for the monotonic tests, as well as for the positive and negative segments of the cyclic tests are listed in Table 6. Recommended values for design were determined based on an average of the positive and negative reversed cyclic test results, which were then averaged with the monotonic results for each connection / sheathing type configuration. Since the CUREE reversed cyclic protocol for ordinary ground motions produces results that are very similar to those revealed by a monotonic test for an identical wall configuration *(Chen, 2004; Chen et al., 2006)*, it was decided that the results for the monotonic tests and the reversed cyclic tests would be combined to produce a minimum of six nominal shear values for each wall configuration.

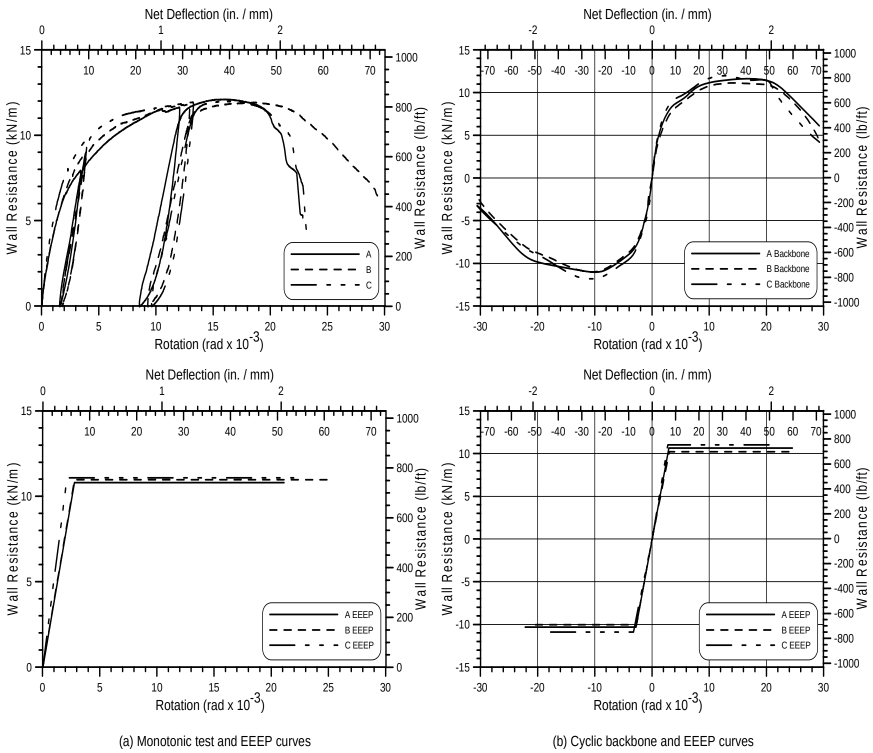


Figure 7 : Typical family of test and EEEP curves for monotonic and reversed cyclic tests

Table 6 : EEEP average test values

Specimen	S_y (kN/m)	$\Delta_{net,y}$ (mm)	K_e (kN/mm)	$\theta_{net,y}$ (rad x 10^{-3})	μ	Energy (E_b) (joules)
41– A,B,C	11.0	6.2	2.18	2.6	9.05	701
42– A,B,C (Pos.)	10.6	7.1	1.84	2.9	8.16	702
42– A,B,C (Neg.)	10.4	7.2	1.78	2.9	6.94	576
43– A,B,C	16.3	8.9	2.26	3.6	5.75	910
44– A,B,C (Pos.)	16.0	9.8	2.04	4.0	5.75	973
44– A,B,C (Neg.)	15.1	7.9	2.34	3.2	6.62	881
45– A,B,C	21.7	11.3	2.35	4.6	4.30	1130
46– A,B,C (Pos.)	19.6	9.1	2.62	3.8	5.23	1052
46– A,B,C (Neg.)	18.6	7.9	2.86	3.2	5.32	864

Design Procedure

At this time it is recommended that the shear resistance of a given structure made of light gauge steel frame / wood panel shear walls be obtained by the summation of the shear resistances of each shear wall segment of a storey (Eq. 1), assuming that the aspect ratio of each segment is less that 2:1 (height : length). The shear resistance of a wall segment is computed using the resistance factor (ϕ) the nominal shear resistance (S_y) (Table 7), the load duration factor (K'_D) and the length of the wall segment (L) (Eq.2).

$$S_r = \sum S_{rs}$$

where,

S_{rs} = $\phi\, S_y\, K'_D\, L$
S_r = Factored shear resistance, [kN]
S_{rs} = Factored shear resistance of shear wall segment, [kN]
ϕ = 0.7
S_y = Nominal shear strength (Table 7)
K'_D = Load duration factor
= 1.0 for short term loading
= 0.56 for permanent loading
= 0.87 for standard loading
L = Length of shear wall segment [m]

Previous studies have indicated that 2440 mm (8′) and 1220 mm (4′) long walls reach their maximum shear capacity at approximately the same deflection level, whereas the deflection for 610 mm (2′) long walls (4:1 aspect ratio) is almost double at the ultimate load position *(Chen, 2004; Chen et al., 2006)*. In a design situation 610 H 2440 mm (2′ H 8′) walls should not be expected to develop their full capacity together with either a 2440 H 2440 mm (8′ H 8′) or a 1220 H 2440 mm (4′ H 8′) wall. There exists in the AISI lateral design standard for cold-formed steel framing *(2004)* a method to determine the reduction of shear wall capacity dependent on the ratio of wall length to height. This method may be applicable for use with the shorter walls; however, this has yet to be confirmed using the results of the shear wall tests described herein. A more comprehensive study of short shear wall segments, *i.e.* less than 1220 mm (4′) in length, constructed of Canadian wood sheathing and framing is required before the AISI method to account for shear wall length is suggested for use in Canada. Hence, at this time it is recommended that a limit of 2:1 (height : length) be placed on the aspect ratio of shear wall segments.

Table 7 : Nominal shear strength, S_y (kN/m), and unit elastic stiffness, k_e (kN/mm/m)

Minimum nominal Panel thickness (mm) & Grade	**Screw spacing at panel edges (mm)**					
	75		100		152	
	S_y (kN/m)	k_e kN/mm/m)	S_y (kN/m)	k_e (kN/mm/m)	S_y (kN/m)	k_e (kN/mm/m)
9 mm OSB CSA O325 2R24/W24	20.4	2.09 (1.88)[10]	15.9	1.82 (1.75)	10.7	1.64 (1.38)

Notes:

(1) ϕ = 0.7 to obtain factored resistance for design.

(2) Full-height shear wall segments of maximum aspect ratio 2:1 shall be included in resistance calculations. Increases of nominal strength for sheathing installed on both sides of the wall shall not be permitted.

(3) Tabulated values are applicable for dry service conditions (sheathing panels) and short-term load duration (K'_D = 1.0) such as wind or earthquake loading. For shear walls under permanent loading, tabulated values must be multiplied by 0.56; and under standard term loads, tabulated values must be multiplied by 0.87.

(4) Back-to-back chord studs connected by two No. 10-16 x 3/4O (19.1 mm) screws at 12O (305 mm) o.c. equipped with industry standard hold-downs must be used for all shear wall segments with intermediate studs spaced at a maximum of 24O (610 mm) o.c. For 8′ (2440 mm) long shear walls, back-to-back studs are also used at the centre of the wall to facilitate the use of a 1/2O (12.7 mm) edge spacing.

(5) All panel edges shall be fully blocked with edge fasteners installed at not less than 1/2O (12.7 mm) from the panel edge and fasteners along intermediate supports shall be spaced at 305 mm o.c. Sheathing panels must be installed vertically with strength axis parallel to framing members.

(6) Minimum No.8 x 1/2O (12.7 mm) framing and No. 8 x 1-1/2O (38.1 mm) sheathing screws shall be used.

(7) ASTM A653 Grade 33 ksi (230 MPa) of minimum uncoated base metal thickness 0.043O (1.09 mm) steel shall be used throughout.

(8) Studs: 3-5/8O (92.1 mm) web, 1-5/8O (41.3 mm) flange, 1/2O (12.7 mm) return lip.
Tracks: 3-5/8O (92.1 mm) web, 1-1/4O (31.8 mm) flange.

(9) The above values are for lateral loading only. It must be noted that the compression chord failure mode must be accounted for in design, including the effects of gravity loads.

(10) Stiffness values obtained for shear walls sheathed with 11 mm (7/16O) OSB panels *(Branston, 2004; Branston et al. 2006b)*.

Note that in Table 7 a secondary set of stiffness values (k_e) have been presented for the three wall configurations. These values originate from the work of Branston *(2004)* and Branston *et al. (2006b)*, which covered the development of design parameters for shear walls constructed with thicker wood sheathing. In the case of walls with 11 mm (7/16O) OSB, the bracketed k_e values were obtained. The analysis results show that the shear walls with the thinner sheathing posses a higher initial stiffness than those with 11 mm (7/16O) OSB panels. A similar finding was made by Rokas *(2006)*, who developed design parameters for walls sheathed with 9.5 mm (3/8O) plywood panels. This secondary set of stiffness values have been presented because it seems counterintuitive that a shear wall with thinner wood panels would be able to provide higher k_e values, yet lower shear strengths. A number of possible explanations for this finding exist, including; a change in the material properties or species type from the thicker to thinner wood panels because of different mills or companies of manufacture. Or perhaps, less tilting of the sheathing screws in the initial stages of loading due to the thinner wood panels may result in elevated initial stiffness values. At this time a definitive reason as to why these k_e values are higher for the 9 mm (3/8O) OSB has not been formulated; hence, the second set of stiffness values have been presented such that a designer may select a more conservative approach to determine shear wall deflections.

Further to this design approach, it is recommended that a factor (K'_D) be included to account for the influence of the duration of the applied load on wood strength. In general, wood products exhibit a decreased resistance to long-term loads. Furthermore, the nominal values listed in Table 7 are for dry conditions only. Wet conditions, *i.e.* an increase in equilibrium moisture content, may present a durability problem for the steel members, and may also lead to a reduction in the capacity of wood members.

Limit States Design Calibration

In order to determine a factored shear resistance for use in design it was necessary to calibrate a resistance factor with respect to the 2005 NBCC wind loads. The calibration procedure was adopted from the North American Specification for the Design of Cold-Formed Steel Structural Members *(CSA S136, 2001; AISI, 2001)*. The CSA Guidelines for the Development of Limit States Design *(CSA S408, 1981)* also present the derivation of the calibration equation (Eq. 3), which was based on the work of Ravindra and Galambos *(1978)*. A detailed description of the calibration procedure can be found in the work of Branston *(2004)*, Branston *et al. (2006b)* and Blais *(2006)*. Calibration of the resulting design values with the 2005 NBCC wind loads, having a 1 in 50 year reference velocity pressure, and a reliability factor of $\beta = 2.5$ resulted in a resistance factor for limit states design of $\phi = 0.7$. This resistance factor is also recommended for use in seismic design.

$$\phi = C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{V_m^2 + V_F^2 + C_P V_P^2 + V_S^2}}$$

where,

C_ϕ = Calibration coefficient

M_m = Mean value of material factor for type of component involved

F_m = Mean value of fabrication factor for type of component involved
P_m = Mean value of professional factor for tested component
β_o = Reliability/safety index
V_m = Coefficient of variation of material factor
V_F = Coefficient of variation of fabrication factor
V_P = Coefficient of variation of professional factor
C_P = Correction factor for sample size = $(1+1/n)m/(m-2)$ for $n \geq 4$, and 5.7 for $n=3$.
V_S = Coefficient of variation of the load effect
m = Degree of freedom = n-1
n = Number of tests

Factor of Safety

The resistance factor and the nominal shear strength values recommended for design were used to calculate the factor of safety associated with light gauge steel frame / wood panel shear walls. Two different calculation methods were implemented; the first is associated with the limit states design (LSD) approach, whereby a simple comparison of the measured ultimate shear resistance with the nominal shear capacity was carried out (Figure 8). The second approach is in terms of an allowable stress design (ASD) format where the factor for wind load is taken into account. Thus the 1.4 wind load factor defined by the 2005 NBCC was utilized.

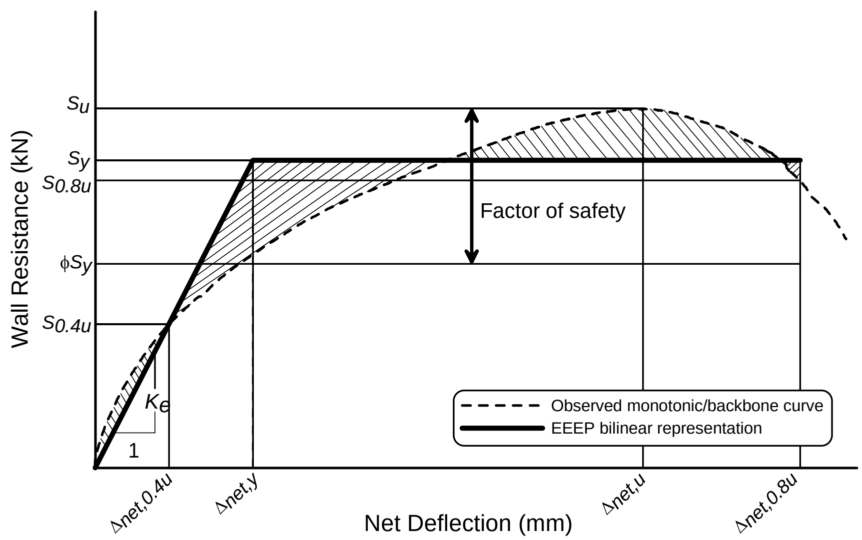


Figure 8 : Factor of safety inherent in limit states design

According to Branston *(2004)*, the factor of safety for allowable stress design (ASD) of light gauge steel frame / wood panel shear walls should fall between 2.0 and 2.5. These values are suggested by the 2000 IBC *(ICC, 2000)* for light gauge steel frame shear walls and by the IBC 2000 Handbook *(Ghosh and Chittenden, 2001)* for wood shear walls, respectively. When monotonic test values are considered, the ASD factor of safety ranges from 2.22 – 2.40 with an average of 2.31 (SD of 0.09 & CoV of 3.9%). For reversed cyclic tests, the ASD values range from 2.01 – 2.29 with an average of 2.14 (SD of 0.09 & CoV of 4.1%. Although these results are somewhat lower than those described by Branston *(2004)*, where an ASD value of approximately 2.4 was calculated, the values fall within the suggested range of 2.0 – 2.5. Furthermore, wind loads according to the 2005 NBCC are now based on a return period of 50 years, which provides for an added factor of safety when compared to wind loads based on the previous version of the NBCC *(NRCC, 1995)* (1 in 30 year return period).

Overstrength

As is discussed by Boudreault *et al. (2006)* force modification factors greater than unity, for both ductility and overstrength, are recommended for use in the calculation of seismic loads for this shear wall type according to the 2005 NBCC. Hence, in terms of capacity based seismic design requirements, if these walls were selected to form the fuse element in the seismic force resisting system (SFRS), they would be expected to dissipate energy by failing in a ductile manner. In design it would be anticipated that the sheathing to framing connections alone fail, to ensure that the steel frame is available to carry gravity loads after a design level earthquake. Blais *(2006)* has shown that the failure of all test walls was due to the deterioration or the complete loss of the connection between the wood structural panel and the light-gauge steel-frame. The failure modes for the wood-to-steel connections involved combinations of pull-through of the screws in the wood sheathing, tearing out of the sheathing edge and wood bearing / plug shear failure. In no case were the chord studs damaged due to high axial loads. However, in testing no gravity loads were applied to the shear walls in addition to the lateral in-plane loads. Hence, in a design situation the chord studs would need to be selected such that the compression forces due to gravity loads in combination with the forces associated with the ultimate shear capacity of the wall, as controlled by sheathing connection failure, could be resisted. This presents the engineer with the problem of selecting the other components in the SFRS such that they have a capacity that exceeds the probable resistance of the shear wall. Components such as the chord studs, tracks, hold-downs, anchors rods, shear anchors, foundation, *etc.* are included in the SFRS. In order to estimate this capacity, the nominal shear strength (S_y) of the wall must be multiplied by the overstrength factor (Figure 9). This factor can be obtained by dividing the ultimate shear wall resistance (S_u) by the nominal shear wall strength (S_y).

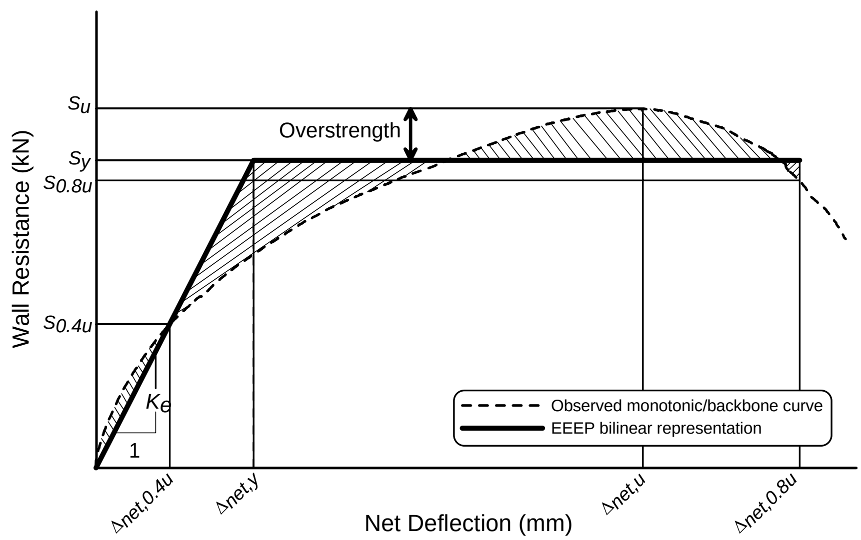


Figure 9 : Overstrength inherent in limit states design

The overstrength factors for the monotonic tests fall between 1.11 – 1.23, with an average of 1.15 (SD of 0.043 & CoV of 3.74%). The same factors for the reversed cyclic tests fall between 1.00 – 1.11, with an average of 1.07 (SD of 0.046 & CoV of 4.26%). Both averages are within the range of overstrength factors obtained from the previous shear wall tests completed at McGill University; which were 1.08 – 1.57 and 1.04 – 1.44 for monotonic and cyclic tests, respectively *(Branston, 2004)*. To validate the overstrength value of 1.2 suggested by Branston *(2004)*, the data of this present research were integrated with those of the previous studies. Average values of 1.22 (SD of 0.104 and CoV of 8.53%) and 1.17 (SD of 0.111 and CoV of 9.46%) were obtained for the monotonic and cyclic tests, respectively (based on 96 tests), which show that the previously suggested value of 1.2 for overstrength is appropriate.

Conclusions

A research program comprising eighteen (three configurations) light gauge steel frame / 9 mm OSB wood panel shear wall tests has led to the development of recommended design parameters for use with the 2005 NBCC loading provisions. These tests are an addition to the database of sixteen wall configurations established by Boudreault *(2005)*, Branston *(2004)* and Chen *(2004)*, and the three configurations tested by Rokas *(2006)*. The data obtained from the tests were used in combination with the equivalent energy elastic-plastic (EEEP) analysis approach to derive design values for the walls, including: shear strength, shear stiffness and a resistance factor (ϕ = 0.7). Nominal shear strength (S_y) and elastic stiffness (k_e) values have been recommended for use with shear walls constructed as per the test specimens. Additional design information, including an overstrength factor and a safety factor has been presented. A factor of safety associated with the 2005 NBCC wind load and proposed resistance factor has been found to fall within the range expected of shear walls. It must be noted, however, that the tabulated resistances do not account for gravity loading in combination with lateral in-plane loading. The designer must be aware that compression buckling / local buckling failure may occur in the chord studs, and therefore these studs must be designed to resist the expected forces due to the combined gravity and lateral effects in order to preserve the overall structural integrity of the building.

Acknowledgements

The authors would like to acknowledge the support provided by the Canada Foundation for Innovation (CFI), Le Fonds québécois de la recherche sur la nature et les technologies (FQRNT), the Natural Sciences and Engineering Research Council of Canada (NSERC), the Canadian Sheet Steel Building Institute, Simpson Strong-Tie Co. Inc., the Structural Board Association, Grant Forest Products Inc. and the Canam Group Inc.. Assistance with the procurement of steel framing was provided by Bailey Metal Products Limited.

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