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DEVELOPMENT OF CANADIAN SEISMIC DESIGN PROVISIONS FOR STEEL SHEATHED CFS FRAMED SHEAR WALLS

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ABSTRACT

Seismic design provisions for cold-formed steel (CFS) framed shear walls sheathed with steel panels are currently not available in the NBCC or in the CSA S136 CFS Specification. To address this lack of information a shear wall design method was developed that could be used in conjunction with the NBCC and incorporated into the Canadian section of the North American Lateral Design Standard for CFS structures. A total of 54 single-storey shear walls of various configurations were tested under monotonic and reversed cyclic loading to establish a database of information. The walls varied in terms of framing and sheathing thickness, detailing and aspect ratio. The equivalent energy elastic-plastic (EEEP) analysis approach was chosen to derive key design parameters for the shear walls including; nominal shear resistance, resistance factor, elastic stiffness, overstrength, ductility and test-based seismic force modification factors. Presented herein is a description of the test program, the development of the proposed design method, the resulting design values according to typical perimeter fastener schedules and sheathing type, as well as the calibration of a resistance factor. To augment the database, results of shear wall tests from the US were also incorporated in the study. This allowed for a broader range of wall configurations to be included in the recommended design provisions.

Introduction

The use of thin steel sheathing in place of wood panel products for the construction of shear walls is relatively new; as such there are no guidelines available for the design of steel sheathed / cold-formed steel (CFS) framed shear walls in the 2005 National Building Code of Canada (NBCC) (NRCC 2005) or in the CSA S136 Specification for the design of CFS structural members (2007). The North American Lateral Design Standard for cold-formed steel (AISI S213 2007) provides Canadian provisions for the design of wood sheathed shear walls and strap braced walls. Similarly, design provisions for steel sheathed shear walls for use in the US and Mexico are available in the S213 Standard; however, to date no Canadian design values have been made available for shear walls of this type. The US resistance values listed for steel sheathed shear walls

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in the 2007 AISI S213 Standard are based on a total of 14 tests carried out by Serrette (1997). The test walls were sheathed with 0.46 mm and 0.68 mm thick steel panels supported on 0.84 mm thick framing members. The current US design values were obtained from the peak load measurements of these specimens. In contrast, the equivalent energy elastic-plastic (EEEP) analysis method (Branston et al. 2006) has been used in the development of Canadian shear resistance values for the wood sheathed shear walls listed in AISI S213. Recently, Yu et al. (2007) completed a series of tests to verify Serrette's data and to increase the number of wall configurations available for an engineer in the US to choose from. Tests on walls with 0.76 mm and 0.84 mm thick sheathing supported on 1.09 mm framing were completed. Wall specimens similar to those used by Serrette (0.46 and 0.68 mm sheathing) were also tested. Furthermore, Ellis (2007) completed six shear wall tests constructed with 0.68 mm thick sheathing to validate the results obtained by Serrette and Yu et al. These existing shear wall data could have been used for the development of Canadian design shear resistance values and other associated parameters. It was nevertheless important to carry out additional tests on walls longer than 1.22 m because these had not been tested by Serrette, Yu et al. or Ellis; in addition, some of the configurations previously tested were included in the scope of the research project to further validate the existing test results. In this fashion a combined database of US and Canadian test results was relied on in the development of the steel sheathed CFS shear wall design provisions described in this paper.

Test Program

As part of the research program a total of 54 steel sheathed single-storey shear wall tests (18 configurations) were carried out at McGill University in 2008 by Balh (2010) and Ong-Tone (2009) (Table 1). The walls, which consisted of framing elements and a sheathing panel on one side (ASTM A653 Grade 230 MPa steel (2002)), measured 610 x 2440 mm, 1220 x 2440 mm, 1630 x 2440 mm and 2440 x 2440 mm in size. The frame was constructed using tracks (92.1 mm web, 38.1 mm flange) and studs (92.1 mm web, 41.3 mm flange and 12.7 mm lip) screw connected together (No. 8 x 12.7 mm wafer head self-drilling / self tapping screws). The studs were placed at a distance of 610 mm on centre except at the wall ends where back-to-back chord studs were installed. In addition, a Simpson Strong-Tie S/HD10S hold-down device was placed at the bottom of each chord stud to anchor the shear wall to the test frame (Figure 1). The shear walls varied in framing thickness, sheathing thickness and detailing of the fasteners. The framing thickness was either 0.84 mm or 1.09 mm. The sheathing panels were available in two sizes; 1220 x 2440 mm and 610 x 2440 mm and were either 0.46 mm or 0.76 mm in thickness. Each panel was attached to the frame with No. 8 x 19 mm self drilling / self tapping pan head screws spaced at 50 mm, 100 mm or 150 mm over the perimeter and at 300 mm along the field stud(s). The 1830 mm and 2440 mm long walls required the use of two sheathing panels butted together at a single internal stud.

The shear walls were tested using two loading protocols; monotonic and CUREE reversed-cyclic protocol (ASTM E2126 2005; Krawinkler et al. 2000). The monotonic loading protocol simulates a wind loading scenario where the shear wall is displaced laterally until the 0.8 post peak shear resistance is reached. The CUREE reversed-cyclic protocol represents the displacement based demand expected during a design level earthquake. The protocol is based on the yield displacement reached by a nominally identical shear wall under monotonic loading. The complete cyclic loading history for a particular wall configuration is then based upon multiples of the reference deformation (Balh 2010). The frequency of the reversed cyclic tests was 0.5 Hz, except toward the end of the protocol where 0.25 Hz was used.

A specially constructed reaction frame (Figure 1), with a 250 kN capacity dynamic actuator having a displacement range of ± 125 mm, was used to test each shear wall under stroke control while measuring the resistance and relevant deformations. The displacements of and forces on each shear wall specimen was monitored by LVDTs and load cells, respectively. 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. Detailed information on the test program can be found in the reports by Balh (2010) and Ong-Tone (2009).

Table 1. Matrix of shear wall tests.

Configuration	Sheathing Thickness (mm)	Wall Length (mm)	Wall Height (mm)	Fastener Spacing (mm)	Framing Thickness (mm)	Number of Tests and Protocol[1]
1	0.46	1220	2440	150/300	1.09	3M & 2C
2	0.46	1220	2440	50/300	1.09	2M & 2C
3	0.46	1220	2440	150/300	0.84	2M & 3C
4	0.76	1220	2440	150/300	1.09	2M & 2C
5	0.76	1220	2440	100/300	1.09	3M & 2C
6	0.76	1220	2440	50/300	1.09	3M & 2C
7	0.76	1220	2440	100/300	0.84	1M
8	0.76	610	2440	100	1.09	2M & 2C
9	0.76	610	2440	50	1.09	3M[2] & 2C
10	0.76	610	2440	100	0.84	1M
11	0.76	2440	2440	100/300	1.09	2M & 2C
12	0.76	1830	2440	100/300	1.09	1M
13	0.76	1830	2440	50/300	1.09	1M
14[3]	0.76	1220	2440	50/300	0.84	4M
15[4]	0.76	1220	2440	100/300	1.09	1M
16[5]	0.76	1830	2440	100/-	1.09	1M
17	0.46	1220	2440	-/300	1.09	2M
18	0.46	1220	2440	75/300	1.09	1M

[1] M-Monotonic, C-Cyclic [2]Addition of bridging to Test 9M-c [3]Various reinforcement schemes
[4]Raised hold-downs [5]Wall with window opening

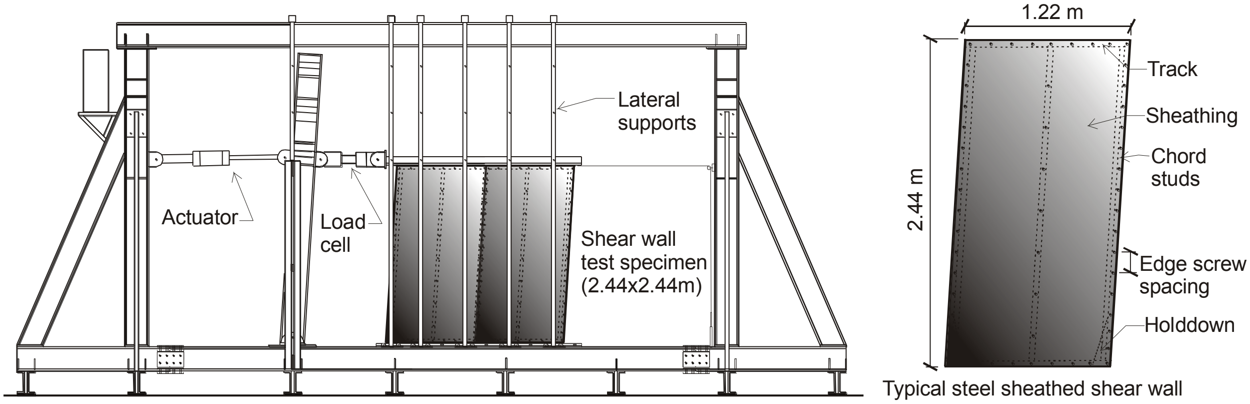

Figure 1. Schematic drawing of test frame and steel sheathed shear wall.

General Test Results

During loading of both the monotonic and reversed cyclic test specimens elastic shear buckling of the sheathing panels was first observed followed by the development of a tension field (Figure 2). At ultimate load the majority of failures occurred at the sheathing-to-frame connections where steel bearing (Figure 3), fastener pull-out, fastener pull-through and tear out were observed. Unzipping of the fasteners along the perimeter of the shear wall was also common. In some cases deformations were observed in the framing elements, especially when the thicker sheathing was combined with a screw spacing of 50 mm. Local twisting of the chord studs (Figure 3) occurred due to the horizontal component of the tension field that developed in the sheathing. In addition, uplift of the track elements took place due to the vertical component of the tension field.

Figure 2. Shear wall test specimens showing tension field and elastic shear buckling deformations in steel sheathing.

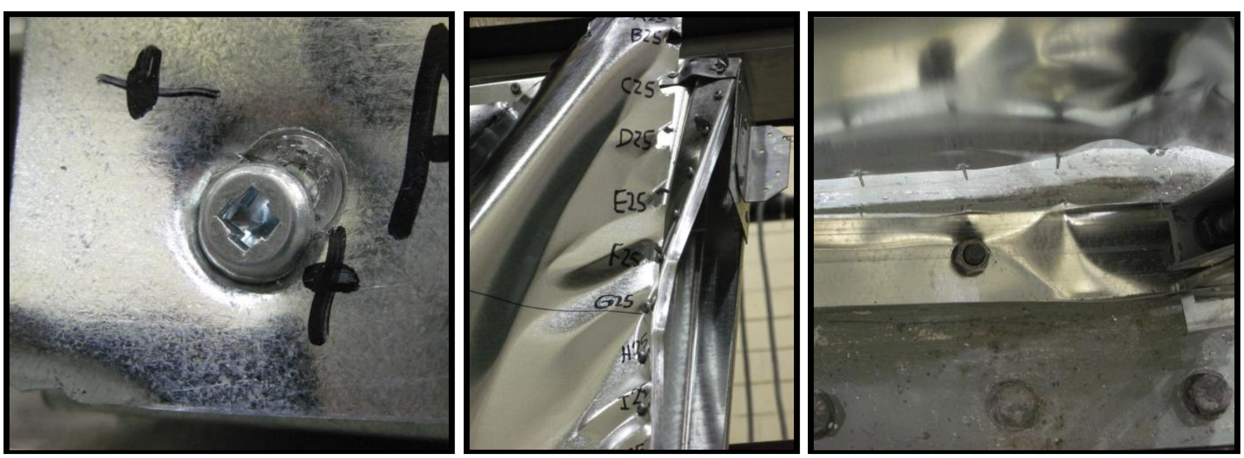

Figure 3. Shear wall test specimens showing bearing damage at screw connections, as well as chord stud twisting and track uplift caused by tension field forces.

As expected, the shear walls with a fastener spacing of 50 mm reached a higher capacity than walls with a fastener spacing of 150 mm along the perimeter. The observation can be attributed to the fact that there are more fasteners available to resist the applied lateral load. The shear walls that were tested with the thicker sheathing of 0.76 mm attained an ultimate resistance which exceeded that of walls sheathed with the 0.46 mm thick panels. Shear buckling of the panels occurred at low load levels; hence, the ultimate shear resistance is essentially a function of the tension field in the sheathing and the ability to anchor this force to the framing members. A range of wall lengths was tested to determine whether the shear resistance per unit wall length was consistent given a set construction configuration. It was observed that the walls with aspect ratios of 2:1, 1.33:1 and 1:1 were able to provide similar shear resistance per metre, whereas the 610 mm long walls (4:1 aspect ratio) did not attain the same resistance. This behaviour is attributed to the flexibility of the short length walls, which resulted in high shear rotations, and the inability to develop a comparable tension field in the sheathing.

Due to the twisting that was observed in the chord studs during the tests, supplemental specimens of configurations 9 and 14 were constructed with bridging channels placed through the web knock-out holes to provide rotational restraint. The results of these tests illustrated that the additional bridging reduced deformations of the chord studs and increased the shear resistance that could be reached (Ong-Tone 2009). However, the bridging elements were too slender which resulted in their lateral torsional buckling. Further study of a means to provide adequate stud bracing is suggested; the design method described herein was based only on the walls in which the stud members were not braced and can be considered a lower bound design situation.

Ancillary Testing of Materials

The framing and sheathing components were tested according to ASTM A370 (2006) to determine the material properties (Table 2). The measured yield and tensile stresses of the wall components were higher than the nominal values of 230 MPa and 310 MPa, respectively. The zinc coating was removed from the coupons such that the base thickness of the metal could be measured.

Table 2. Measured material properties of the steel framing and sheathing

Nominal Material Thickness (mm)	**Member**	**Base Metal Thickness (mm)**	**F_y (MPa)**	**F_u (MPa)**	**F_u / F_y**	**% Elong. 50 mm Gauge**
0.84	Stud & track	0.87	342	391	1.14	31
1.09	Stud & track	1.14	346	496	1.43	31
0.46	Sheathing	0.46	300	395	1.32	26
0.76	Sheathing	0.76	284	373	1.32	35

Development of a Limit States Design Procedure

Similar to wood sheathed CFS shear walls, the shear resistance vs. deformation behaviour of steel sheathed shear walls under lateral loading is highly nonlinear. The Equivalent Energy Elastic Plastic (EEEP) method (Park 1989; Foliente 1996) was relied on to provide a bilinear elastic-plastic curve, which was then used to evaluate the performance of each shear wall. This approach is consistent with that used by Branston et al. (2006) in the development of Canadian shear wall resistance values and other related design values for the wood sheathed shear walls that are listed in the AISI S213 Standard. The EEEP method is valid for the analysis of results of both monotonic and reversed-cyclic tests. A backbone curve must be constructed for the positive and negative displacement ranges of the resistance vs. deflection hysteresis of each reversed cyclic test. The typical nonlinear force vs. displacement response of a CFS shear wall is simplified for design by means of a bi-linear curve for which the plateau portion is set as the nominal shear resistance, S_y. This plateau section is established by equating the energy dissipated in testing (area under the backbone or monotonic curve) with the area under the bilinear curve, or as shown in Figure 4 by setting A1 = A2. The bilinear curve is limited by the ultimate failure displacement, assumed to occur at the 0.8 post-peak resistance, and the initial slope which is defined using the secant stiffness positioned through the position defined by 40% of the ultimate load. Velchev (2009) also used the EEEP method to analyze the tests by Yu et al. (2007) and Ellis (2007).

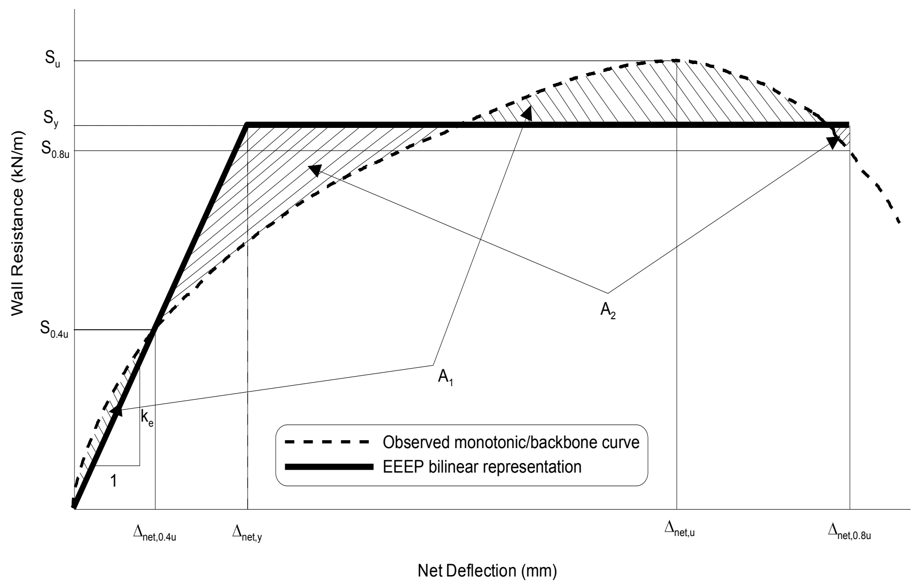

Figure 4. EEEP model *(Branston, 2004)*

The EEEP results for the McGill and the US shear wall tests were incorporated to obtain uniform values for resistance, ductility and stiffness. The shear walls were grouped based on measured values of sheathing thickness, framing thickness, and fastener spacing. The material properties tests (Table 2) showed that the thickness and tensile stress of the various wall components differed from the nominal values; therefore, the yield resistance of each test wall was reduced such that it represents the expected resistance if a wall had been constructed of framing and sheathing that did not exceed the minimum specified thickness and material stress (see Balh (2010)). The proposed nominal yield resistance values, S_y, of steel sheathed shear walls based on sheathing thickness, framing thickness, and fastener spacing are presented in Table 3. Note, these values were obtained through consideration of the test results for walls with an aspect ratio of at least 2:1; the shorter 4:1 (610 mm long) walls were not included.

A resistance factor, ϕ, is used to obtain the factored resistance values used in limit states design. CSA S136 defines a method for determining the ultimate limit state resistance factor to be used with any proposed expression for the design of cold-formed steel members, assemblies or connections. The resistance factor, ϕ, was determined based on the most common failure modes that occurred during the shear wall tests, namely failure in the connections due to shear and bearing, chord stud failure and uplift failure of the tracks. An average material resistance factor, ϕ, of 0.7 was proposed for use with steel sheathed shear walls (Balh 2010).

Short walls measuring 610 x 2440 mm were tested to determine if shear walls with higher aspect ratios can be used. The AISI S213 Standard specifies that the resistance of high aspect ratio walls be multiplied by 2w/h, where w and h are the length and height of the wall, respectively. Based on the test results, the EEEP derived S_y resistance values (kN / m) of the 610 x 2440 mm walls were higher than the recommended values listed in Table 3 (after being adjusted); therefore, the use of the 2w/h shear strength reduction factor for higher aspect ratio shear walls is appropriate (Balh 2010).

Table 3. Proposed nominal shear resistance, S_y, for CFS frame / steel sheathed shear walls[1,2,8]

Assembly Description	Max. Aspect Ratio (h/w)[3]	Fastener Spacing[4] at Panel Edges (mm(in))				Designation Thickness[5,6] of Stud, Track, and Blocking (mm) (mils)	Required Sheathing Screw Size[7]
		150(6)	100(4)	75(3)	50(2)		
0.46 mm (0.018") steel sheet, one side	2:1	4.13	-	-	-	0.84 (33)	8
		4.53	6.03	6.78	7.53	1.09 (43)	8
0.68 mm (0.027") steel sheet, one side	2:1	6.48	7.17	7.94	8.69	0.84 (33)	8
0.76 mm (0.030") steel sheet, one side	4:1	8.89	10.58	11.56	12.54	1.09 (43)	8
0.84 mm (0.033") steel sheet, one side	4:1	10.69	12.01	12.97	13.94	1.09 (43)	8

1 Nominal resistance (kN/m) is to be multiplied by the resistance factor, ϕ, to obtain the factored shear resistance
2 Sheathing is to be connected vertically to the steel frame
3 Nominal resistances are to be multiplied by 2w/h for aspect ratios greater than 2:1 but no greater than 4:1
4 Field screws to be spaced at 300 mm on centre
5 Wall stud and track shall be of ASTM A653 grade 230 MPa with a minimum uncoated base thickness of 0.84mm (0.033”) for members with a designation thickness of 33 mils, and ASTM A653 grade 230 MPA with a minimum uncoated base thickness of 1.09 mm (0.043”) for members with a designation thickness of 43 mils
6 Substitution of wall stud or track is not permitted
7 Minimum No. 8x12.7mm (1/2”) self drilling / self tapping screws shall be used
8 Tabulated nominal resistances are applicable for lateral loading only

Factor of Safety

The resistance factor and the nominal shear resistance values recommended for design were used to calculate the associated factor of safety. Two different calculation methods were implemented; the first applies to the limit states design (LSD) approach, whereby a simple comparison of the measured ultimate shear resistance with the nominal shear resistance was carried out. The second approach is in terms of an allowable stress design (ASD) format where the factor for wind load is taken into account. The average LSD factor of safety was 2.0, while the ASD factor was 2.8; these are both similar to the factor of safety obtained for wood sheathed shear walls (Branston et al. 2006). The factor of safety is only applicable to lateral loading and does not take into account the effects of gravity loading.

Overstrength

The overstrength factor was calculated in a similar manner to the factor of safety. It was determined by using the ratio of the measured ultimate resistance for each individual test to the nominal shear resistance, S_y. The factor is used for capacity based seismic design where the energy dissipating element in the seismic force resisting system (SFRS) is expected to reach its ultimate resistance. The connections between the frame and the sheathing in the shear wall are defined as the fuse element; and all other elements in the SFRS such as the chord studs are designed using the overstrength factor to avoid failure. Based on the shear wall test results, an overstrength factor of 1.4 is recommended for steel sheathed shear walls.

Seismic Force Modification Factors

There are two force modification factors used in seismic design according to the NBCC; ductility-related and overstrength-related. The ductility-related force modification factor, R_d, is a measure of the fuse element's ability to dissipate energy through inelastic deformation. Newmark and Hall (1982) determined a relationship between the ductility-related force modification factor and the ductility of the shear walls based on the natural period of vibration of structures.

$$R_d = \sqrt{2\mu - 1} \text{ for } 0.1\text{s} < T < 0.5\text{s} \qquad (1)$$

where,

R_d = Ductility-related force modification factor
μ = Ductility of shear wall
T= Natural period of structure

Boudreault (2005) found that many light framed structures have a natural period less than 0.5 seconds. Therefore, the same assumption for low natural periods was used to determine the R_d value for steel sheathed shear walls (Eq. 1). The average value for R_d from test results was 2.87 (Balh 2010). It is suggested that a value of 2.5 be considered to be consistent with the R_d value currently specified in AISI S213 for wood sheathed shear walls.

A second force modification factor, R_o, represents the reserve of strength within the structure. Its calculation was based on the recommendation of Mitchell et al. (2003) who proposed a formula for calculating the overstrength-related force modification factor (Eq. (2)).

$$R_o = R_{size} R_{\phi} R_{yield} R_{sh} R_{mech} \qquad (2)$$

where,

R_{size} = overstrength due to restricted choices for sizes of components
$R_{\phi} = 1/\phi$, (ϕ=0.7)
R_{yield} = ratio of test yield resistance to minimum specified yield resistance
R_{sh} = overstrength due to development of strain hardening
R_{mech} = overstrength due to collapse mechanism

The values for each component of R_o are described by Balh (2010) and Ong-Tone (2009). The average R_o value based on test results is 2.1; a value of 1.7 is suggested, again because of the R_o value for wood sheathed shear walls in AISI S213. These ‘test-based’ R-values require further verification by means of non-linear time history dynamic analyses of multi-storey CFS framed structures following an approach adopted from FEMA P695 on the quantification of building seismic performance factors. Element ductility may be amplified compared to system ductility; hence the results of single storey tests are not sufficient to make final recommendations regarding R-values. Dynamic modeling of multi-storey structures, as well as dynamic tests of multi-storey steel sheathed CFS structures, would allow for the evaluation of failure probability and aid in the development of recommended R-values for design.

Conclusions

To address the lack of Canadian design provisions for steel sheathed / CFS framed shear walls a shear wall test program was carried out comprising 54 single-storey steel sheathed shear walls. The walls were tested using two loading protocols; monotonic and CUREE reversed-cyclic. Shear wall tests completed by Yu et al. and Ellis were incorporated with the tests at McGill University to obtain a larger database from which design values were derived. The majority of failure modes were observed in the connection between the sheathing and frame with some deformation in the frame. The tests were analyzed using the EEEP method which provided a bilinear elastic-plastic curve that can be considered equivalent to the nonlinear behaviour exhibited by the shear walls. Based on the test results a resistance factor, ϕ, of 0.7 is recommended and nominal resistance values, S_y, are proposed. It should be noted that the nominal resistance values are only applicable for lateral loading and do not take into account the effects of gravity loads. An overstrength factor of 1.4 is recommended for capacity based seismic design of elements in the SFRS. ‘Test-based’ R_d and R_o values of 2.5 and 1.7, respectively, are suggested for steel sheathed shear walls. It is further recommended, however, that representative buildings be designed using the seismic design resistances and R-values described herein and then evaluated in terms of the probability of collapse as per the FEMA P695 analysis procedure.

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