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Investigation of design values computation of wood shear walls constructed with structural foam sheathing

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Abstract. This study investigated the ultimate lateral load capacity of shear walls constructed with several types of structural foam sheathing. Sixteen tests were conducted and the results were compared to the published design values commutated by the manufactures for each test series. The sheathing products included 12.7 mm (½ in) SI-Strong, 25.4 mm (1 in) SI-Strong, 12.7 mm (½ in) R-Max Thermasheath, and 2 mm (0.078 in) ThermoPly Green. The structural foam sheathing was attached per the manufacturers' specification to one side of the wood frame for each wall tested. Standard 12.7 mm (½ in) gypsum wallboard was screwed to the opposite side of the frame. Simpson HDQ8 tie-down anchors were screwed to the terminal studs at each end of the wall and anchored to the base of the testing apparatus. Both monotonic and cyclic testing following ASTM E564 and ASTM E2126, respectively, were considered. Results from the monotonic tests showed an 11 to 27 percent smaller capacity when compared to the published design values. Likewise, the test results from the cyclic tests showed a 24 to 45 percent smaller capacity than the published design values and did not meet the seismic performance design criteria computation.

Keywords: capacity; design value; shear wall; sheathing; structural foam sheathing; design criteria computation

1. Introduction

The use of Oriented Strand Board (OSB) and plywood is slowly being replaced with structural foam sheathing in a growing number of residential markets in the United States. Oriented Strand Board (OSB) and plywood are engineered wood products which have been extensively used as structural panel for sheathing wood-frame structures in the United States since 1963 (US Patent 1963). OSB and plywood have multiple uses such as subflooring, wall and roof sheathing, ceiling/deck sheathing, Structural Insulated Panels (SIP), webs of wood I-joists, industrial containers, mezzanine decks, and furniture (APA 1997, 2013 and 2018). Likewise, the premium grade of OSB and plywood, referred to as Structural 1 with highest-strength characteristics, is used for flooring, beams, and wood shear wall construction to resist lateral loads. As an example, International Residential Code (IRC 2015) has qualified the 11 mm (7/16 in) OSB panel to be used as Facing for SIP wall construction.

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There have been extensive studies on OSB and plywood walls including experimental and analytical methods. The goal is to understand the behavior of the walls, improve the walls lateral load strength, and provide design equations and computation technics for design engineers to use it in design. According to Dolen and Madsen (1992) study, sheathing type influenced working stress and ultimate capacity of shear walls. Karacabeyli and Ceccotti (1996) found that gypsum wallboard improved wall strength and stiffness effectively when compared to the walls sheathed only with OSB or plywood on one side of the wall frame. However, using gypsum on one side of the frame reduced the ductility of the walls. Lam *et al.* (1997) investigated the performance and behavior of wood shear walls sheathed with regular (1.2 x 2.4 m [4 x 8 ft]) and nonstandard large dimension (2.4 x 7.3 m [8 x 24 ft]) OSB panels subjected to lateral forces. This study showed the conventional nail spacing and the oversized panel walls had a higher load carrying capacity, stiffness, and ductility, but lower deformation capacity in comparison to the walls with regular OSB panels. In Durham *et al.* (2001) study, large size (2.4 x 2.4 m [8 x 8 ft]) OSB panels were tested and compared to the standard size (1.2 x 2.4 m [4 ft x 8 ft]) OSB panels from previous studies on the seismic resistance of large and standard size OSB sheathing panels. The results showed that the large size OSB had 26% higher shear capacity and 25% lower maximum drift in comparison to the standard size OSB.

There have been few studies specifically focusing on shear walls sheathed with only gypsum wallboard since 1983 (e.g. Wolfe 1983, Zacher and Gray 1985, Olive 1990, Karacabeyli and Ceccotti 1996, McMullin and Merrick 2002, Memari and Solnosky 2014, Chen *et al.* 2016). In Plesnik *et al*. (2016) study, using intermediate layer of gypsum board between the sheathing of a frame showed a reduction in the capacity and stiffness of wood shear walls. This study also found a good agreement between the analytical and experimental methods used in this research. Lafontaine *et al.* (2017) studied on shear wall sheathed with Type-X gypsum wallboard subjected to reversed cyclic loading. It was found that the fastener panel edge distance between 9-19 mm (3/8 -3/4 in.) did not have effect on the shear wall response. Likewise, reducing fastener spacing increased the wall capacity and changed the failure mode to a brittle failure mode.

Sinha and Gupta (2009) studied the load sharing abilities between oriented strand board (OSB) and gypsum wallboard in a shear wall assembly during a racking load event. The test results showed that the gypsum wallboard fails first at 60% of the ultimate load capacity of the wall, and the load resistance shifts to the OSB panel. Likewise, Zhou and He (2011) found that gypsum wallboard had a positive impact on the ultimate load, elastic stiffness, and energy dissipated of the tested walls.

In Patton-Mollory and Wolfe (1985) study, a linearly proportional relationship was found between the racking strength of a shear wall and the wall length. However, the stiffness of the walls was reduced linearly and proportional to the wall openings (Falk and Itani 1987). Therefore, openings such as doors and windows caused a reduction in strength and stiffness of shear walls (He *et al.* 1999). Results indicate that the lateral load capacity increases are nonlinear with wall length which supports a conclusion made by Casagrande *et al.* (2016). Consequently, the effect of openings on shear wall capacity must be counted in design equations for the wall strength computation by design engineers.

Tomasi and Sartor (2013) developed a simple model to determine the load path and its mechanism of load transmission through the connectors. Dinehart and Shenton III (1998 a & b) used ASTM E-564 for monotonic tests and Structural Engineering Association of Southern California (SEAOSC) fully reversed cyclic test. The test walls were sheathed with plywood or OSB panels. The results indicated that the monotonic tests could closely predict the ultimate load capacity of the wall. It was also found that the actual load of a shear wall during an earthquake was much less than the design values used in design at that time.

When the Northridge earthquake struck the San Fernando area in Los Angeles (1994), the old residential wood frame buildings showed good performance as they had a regular plan with small openings and acted as a simple structural box (Filiatrault 1990, Hamburger and McCormick 1997). While the Northridge earthquake caused many structural and non-structural damages to some of the newer wood frame buildings (Hamburger and McCormick 1997). According to Zacher (1999), there was a lack of information on the actual behavior of wood shear walls to determine the capacity of wood frame construction when subjected to strength level loads in service.

In order to address this lack of information, the Consortium of Universities for Research in Earthquake Engineering (CUREE 1999) held a workshop in 1999. This workshop gathered the ideas from the experts in the field of seismic testing, analysis and design of wood buildings (Seible *et al.* (1999). CUREE (1999) conducted experimental testing of full-scale two and three-story wood-frame buildings. In addition, CUREE used an analytical method called Cyclic Analysis of Shear Walls (CASHEW) to predict the capacity of the buildings (Folz and Filiatrault 2001, Pardoen *et al.* 2000). Furthermore, many other analytical methods were developed to analyze and predict the capacity of wood-frame buildings and wood shear walls and provide possibly design equations for the purpose of design (Folz and Filiatrault 2004, Van de Lindt 2004, Van de Lindt *et al.* 2004, NEES 2015, Christovasilis 2008, Van de Lindt *et al*. 2010, Pei *et al.* 2010, Pang *et al.* 2010, Pei and Van de Lindt 2011). Likewise, Pang *et al.* (2007) validated test results from experimental studies on shear walls with different wall configurations and loading protocols using the Evolutionary Parameter Hysteretic Model (EPHM). This study found a good agreement between experimental test results and the analytical EPHM program.

According to Atherton (1983), wall capacity was influenced by nail spacing more than other variables were examined: sheathing thickness, nail size and spacing, blocking, and nail patterns. Wanyama *et al.* (2012) examined timber-plywood-timber joints connected with nails. It was found that the elasticity of the timber-plywood-timber joints is higher than the stiffer control timber-timber joints, and it is improved by increasing the number of nails used to attach the plywood panels to the main timber members.

Zheng *et al.* (2015) studied the effect of different variables on double-shear-nail (DSN) connections used in midply wood shear walls. The results indicated that sheathing thickness and nail edge distance influence the DSN failure mode. Likewise, increasing sheathing thickness and nail edge distance improved the ultimate wall capacity and ductility, but it barley had effect on initial stiffness.

Kamiya *et al.* (1996) tested shear walls using a shake table to determine the effect of the type of fasteners on shear wall response. This study found bolt fasteners had a low influence on wall response in comparison to the nail fasteners. However, in high acceleration, nail fasteners increased the displacement response of the wall extensively when compared to the bolt fasteners. In Shipp *et al.* (2000) study, the hold-down anchors showed a minor effect on the load capacity of a shear wall under reversed-cyclic loading.

Seaders *et al.* (2008) conducted experimental tests following ASTM E564 and CUREE testing methods for the monotonic and cyclic loadings, respectively. The walls were tested with fully and partially anchored wood-frame shear walls. The fully anchored walls were referred to as “the walls with hold-downs at the wall chords” (Pardoen 2000, Gatto and Uang 2002). The walls with no hold-downs were considered partially anchored walls, which were similar to typical residential construction. This study found that the cyclic tests on partially anchored walls showed a lower coefficient of variation than monotonic tests. Also, fully and partially anchored walls showed different failure modes.

Rezazadeh *et al.* (2016) studied sill plates failures of the wood frames during tornadoes like Moore, 2013. This study found that using larger washer size or decreasing bolt spacing prevent sill plate failures caused by bending moment on the bottom face. Likewise, Shadravan and Ramseyer (2018) investigated the effect of the type, number, and spacing of connections on shear wall capacity to resist lateral load, without significant changes to construction practices. The results indicated that doubling the base plate, decreasing anchor bolt spacing, using bearing plate washers, and changing the fastener type and spacing have a significant impact on improving the shear capacity of the shear wall, without significant changes to common wall framing methodologies.

Hao *et al.* (2018) studied seismic behavior of Chuan-Dou type timber which is used in residential building of three stories or less in southern China. This study found that the Chuan-Dou type timber has low ultimate load capacity and stiffness that can be improved by stiffening the frame by infilling wattle and paneling the frame. The Chuan-Dou showed an exceptional deformation and energy dissipation capacity.

Structural foam sheathing is a new wall sheathing system which is recently being used in place of OSB and plywood in a growing number of residential buildings in the United States. According to the manufacturers, the structural foam sheathing products provide multiple benefits as one product. They claim that structural foam sheathing is a strong structural panel with a high capacity to resist lateral loads. In addition, the manufacturers market structural foam sheathing as a water resistive-barrier and air barrier with high R-values, making them energy-efficient and cost-effective (OX Engineered Products 2015 and 2018). The ultimate capacity of the foam panels has been commutated and published by manufactures. These capacities can be used in design equations by design engineers as design values for the structural foam sheathing in a wood frame shear wall structure subjected to wind and seismic forces.

There have been extensive studies on wood shear walls with significant verification of the OSB and plywood lateral load resistance capacity. However, an extensive literature search did not find any research concerning the lateral load resistance of wood frame walls sheathed with structural foam sheathing. Consequently, independent verification of the manufactured published design values is not available. Therefore, this study examined the accuracy of the manufactured published ultimate lateral load capacity using structural foam sheathing in shear walls. Sixteen tests were conducted on shear walls with four different types of structural foam sheathing subjected to monotonic and cyclic loads following ASTM E564 and ASTM E2126 (test method B- ISO 16670 Protocol), respectively. A minimum of two tests was performed with each type of sheathing detail following ASTM test method. The experimental test results were compared to the manufacturers published design values for each test series. In addition, the equivalent curve and seismic performance of structural foam shear walls were commutated based on ASTM E2126 and ASCE 7 for the purpose of strength evaluation of wood shear walls constructed with structural foam sheathing.

2. Test protocol

2.1 Test objective and details

The sixteen test shear walls in this study were 2.4 x 2.4 m (8 x 8 ft) in dimension using 2x4 dimension grade Fir lumber (38.1 x 88.9 mm [1.5 x 3.5 in.]) for framing members. The walls were

Table 1 Test Details

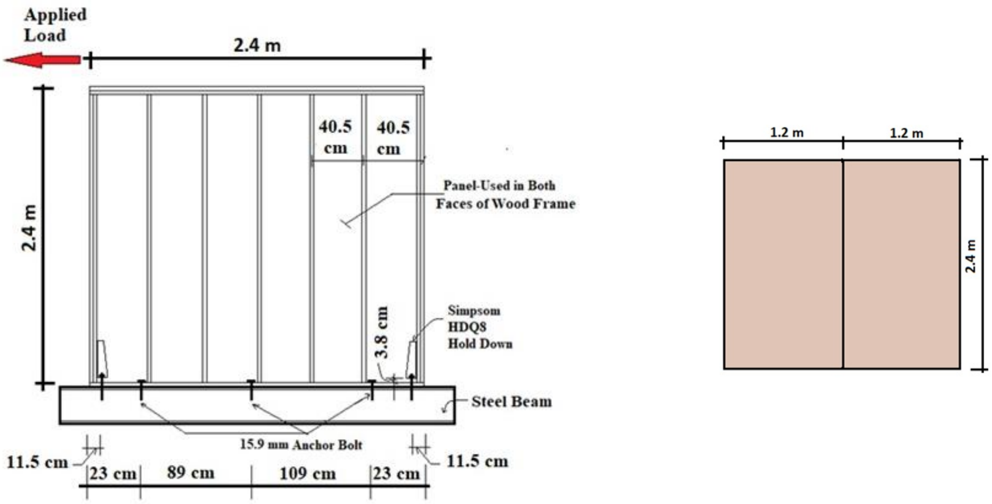
Test Details			Type of Sheathing	Type of Staple	Staple Spacing (Edge/Field) (mm/mm)	Gypsum Wallboard Screw Spacing (Edge/Field) (mm/mm)
Monotonic Tests	Series 1	2 Tests (1M & 2M)	12.7 mm SIS	25.4 mm crown 38 mm leg	76/76	405/405
	Series 3	2 Tests (3M & 4M)	25.4 mm SIS	25.4 mm crown 51 mm leg	76/76	203/203
	Series 5	2 Tests (5M & 6M)	12.7 mm R-Max Thermasheath-SI	12.7 mm crown 32 mm leg	76/150	100/405
	Series 7	2 Tests (7M & 8M)	2 mm ThermoPly Green	25.4 mm crown 38 mm leg	76/76	203/203
Cyclic Tests	Series 2	2 Tests (1C & 2C)	12.7 mm SIS	25.4 mm crown 38 mm leg	76/76	405/405
	Series 4	2 Tests (3C & 4C)	25.4 mm SIS	25.4 mm crown 51 mm leg	76/76	405/405
	Series 6	2 Tests (5C & 6C)	12.7 mm R-Max Thermasheath-SI	12.7 mm crown, 32 mm leg	76/150	203/203
	Series 8	2 Tests (7C & 8C)	2 mm ThermoPly Green	25.4 mm crown 38 mm leg	76/76	405/405

framed with 40.5 cm (16 in.) or 60 cm (24 in.) stud spacing, with double terminal studs, double top plates, and a single base plate. The top plates and baseplate were end-nailed into the studs with (2) 16d box (3.4 x 82.55 mm [0.135 x 3-1/4 in.]) nails. The outer top plate was fastened to the lower top plate with 10d box (3.25 x 76 mm [0.128 x 3 in.]) nails spaced 600 mm (24 in.) on center. The doubled terminal studs at each end of the frame wall were stitched together with 10d box (3.25 x 76 mm [0.128 x 3 in.]) nails spaced at 150 mm (6 in.) on center.

The walls were fully sheathed on one side of the wood frame with the 1.2 x 2.4 m (4 x 8 ft) foam sheathing panels being studied. The foam sheathing types examined included: 12.7 mm (½ in.) SI-Strong (SIS), 25.4 mm (1 in.) SI-Strong (SIS), 12.7 mm (½ in.) R-Max Thermasheath-SI, and 2 mm (0.078 in.) ThermoPly Green. The foam sheathing was stapled to the stud wall using 16-gauge staples with crown and leg and spacing specified by the manufacturers' recommendation, wall details. In each wall, standard 12.7 mm (½ in.) gypsum wallboard was screwed to the opposite side of the wood frame with #6 Type W 32 mm (1-1/4 in.) long.

All the staples and screws were installed with an edge distance of 9.5 mm (3/8 in). Table 1 shows the test details for both the monotonic tests (Series 1,3,5,7) and cyclic tests (Series 2,4,6,8). Note that the walls were framed with 40.5 cm (16 in.) for all tests except tests C7 and C8 (Series 8) which were framed with 60 cm (24 in.) stud spacing.

The walls were anchored to a steel beam (W10 x39) with (3) 15.9 mm (5/8 in) anchor bolts nominally spaced at 1.07 m (3.5 ft) and 76 x 76 x 6 mm (3 x 3 x 0.25 in) bearing plate washers.



(a) Scheme of a typical wall (b) Sheathing and drywall layout

Fig. 1 Typical test wall configuration

The steel beam was bolted to the laboratory test floor in the load assembly frame, which applied an in-plane load. Simpson HDQ8 tie-down anchors were screwed to the terminal studs at each end of the wall with (14) 6 x 76 mm (1/4 x 3 in) SDS screws and anchored to the base using 15.9 mm (5/8 in) anchor bolts.

2.2 Test setup

2.2.1 Typical wall configurations- monotonic and cyclic tests

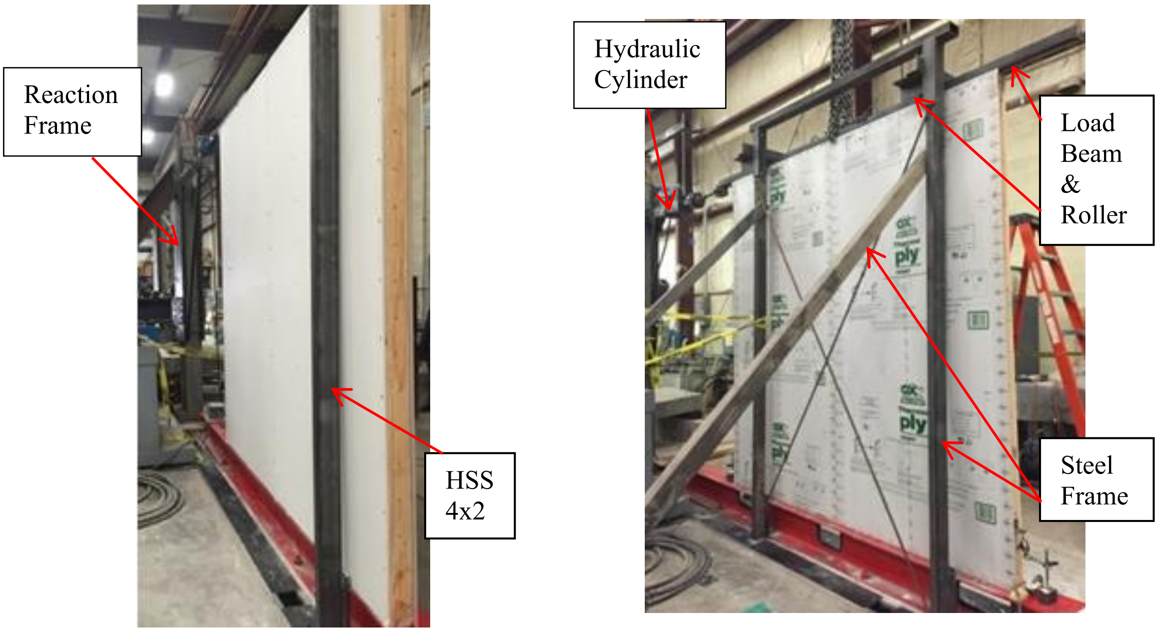
Two ASTM test methods were considered in this research; a monotonic test that followed the ASTM E564 standard and a cyclic test that followed the ASTM E2126 standard (test method B-ISO 16670 Protocol). A minimum of two tests was performed with each type of sheathing based on manufacturers wall details and ASTM test methods.

Fig. 1(a) illustrates a typical shear wall test and b) foam sheathing and gypsum wallboard layout for both monotonic and cyclic tests. The walls were subjected to in-plane lateral loading using a hydraulic cylinder along the top of the walls through a load distribution plate for monotonic tests and a load beam for the cyclic tests. The hydraulic cylinder was mounted onto a reaction frame with an in-line load cell to measure the lateral load (Figs. 2(a)-(b)).

A 60 cm (2.4 ft) long load distribution plate was bolted to the wall assembly with (4) 8 mm (5/16 in.) bolts extending through the top plates of the walls for the monotonic tests. The load beam used for the cyclic tests consisted of a built up 50 x 88 x 6 mm (2 x 3.5 x1/4 in) channel attached to the top of the wall with (8) 3.2 x 50 mm (3/8 x 2 in.) screws, meeting the requirements of ASTM E2126 (Fig. 2(b)).

2.2.2 Out-of-Plane Support Members (Monotonic and Cyclic Tests)

Two methods were used to keep the applied load in-plane with the test walls. For the monotonic tests; an HSS 4x2 steel column was used as an out-of-plane support frame, keeping the applied



(a) Monotonic test (b) Cyclic test

Fig. 2 Frame used to control out-of-plane deformation

load in-plane with the walls (Fig. 2(a)). For cyclic tests: a steel frame with bracing provided by two sets of rollers was built for out-of-plane support, keeping the applied load in-plane with the walls (Fig. 2(b)).

2.3 Test monitoring

For the monotonic tests, a 20 kip (89 kN) hydraulic cylinder was used to apply a tension load to the load plate, resulting in a lateral load applied to the top of the walls (Fig. 2(a)). The lateral load was measured with a load cell located in-line with the hydraulic cylinder (Fig. 2). The displacement of the top of the wall was measured using a string potentiometer. For the cyclic load tests an MTS dynamic control system with a 22 kip (100 kN), a double acting hydraulic cylinder was used to apply a varying tension-compression load to the load beam resulting in an alternating lateral load applied to the top of the walls.

For the monotonic tests, following Test Method ASTM E-564 (Sec. 7), the preload was applied approximately 10% of the estimated ultimate wall capacity. Then, the load was removed after 5 min holding the wall at the preloaded condition. Then the wall was unloaded, and all gages were measured after 5 min wait time. These readings were used as the initial readings. After initial readings, approximately one third and two third of the estimated ultimate load were applied. At each loading level, the wall was unloaded and recorded after 5 min resting. Finally, the wall was loaded until it failed at ultimate applied load. The final measurements were recorded at the same manner.

Cyclic tests followed Test Method B (ISO 16670 Protocol) from ASTM E2126 (Sec. 8.4). The ultimate displacement (Δ_m) from the result of testing a similar specimen in the monotonic test was used for the loading procedure of the cyclic tests. The ISO loading was designed in two

Table 2 Summary of Monotonic test results

Test Details: Type of Foam Sheathing	Test No.	Test Value Peak Load (kN)	Average Test Value (kN)	Average Test Value- Unit Shear Capacity (kN/m)	Published Design Value- Ultimate Unit shear (kN/m)	Ratio (Test/Pub.)	Variation between Avg. Test Value and Published Design Value (%)
SIS/SI-Strong (12.7 mm)	1M	20.5	21.8	9.1	10.8	0.84	-16
	2M	23.2					
SIS/SI-Strong (25.4 mm)	3M	25.5	26.8	11.2	13.9	0.8	-20
	4M	28					
R-Max Thermasheath-SI (12.7 mm)	5M	30.2	30.5	12.7	14.1	0.89	-11
	6M	30.8					
Thermoply Green (2 mm)	7M	21	20.3	8.4	11.5	0.73	-27
	8M	19.5					

displacement patterns; a) five single fully reversed cycles of 1.25%, 2.5%, 5%, 7.5 %, and 10% of the ultimate displacement (Δ_m) and b) three fully reversed cycles of equal amplitude at 20%, 40%, 60%, 80%, 100%, and 120% of the ultimate displacement (Δ_m).

A single National Instruments data acquisition system was used to collect the data which consisted of the lateral load from the load cell and lateral displacements at the top of the wall (deflection) from the string potentiometer for both monotonic and cyclic tests. All data were recorded using a PC laptop and Labview software. In addition, vertical displacements of the end studs and horizontal displacement of the bottom plate relative to the rigid base were measured for the cyclic tests using dial gauges, following ASTM E2126.

3. Test Results

3.1 Monotonic test results- ASTM E564

Table 2 provides a summary of the test results for the monotonic tests. Averages of the test results for each series of tests were computed and compared to the published design value. The average monotonic test values for a pair of tests are 11 to 27 percent below the manufacturers computed and published design values. Likewise, the ratios of average test value to the published design value are 0.73 to 0.89.

3.2 Cyclic test results- ASTM E2126

Table 3 provides the test results for the cyclic tests. Per ASTM E2126, the average test value is based on the absolute values of minimum ultimate loads selected from positive and negative peak loads. For the calculations in this section, the absolute values were of concern, and the smaller absolute value was used in computation as the ultimate load (peak load) for each test. The average test value unit shear capacity for each series of cyclic tests was compared to the published design values commutated by the manufactures. The average cyclic test values for a pair of tests

Table 3 Cyclic test results

Test Details: Type of Foam Sheathing	Test No.	Test Value Peak Load (kN)	Average Test Value (kN)	Average Test Value- Unit Shear Capacity (kN/m)	Published Design Value- Ultimate Unit shear (kN/m)	Ratio (Test/Pub.)	Variation between Avg. Test Value and Published Design Value (%)
SIS/SI-Strong (12.7 mm)	1C	18	19	7.9	10.8	0.73	-27
	2C	19.9					
SIS/SI-Strong (25.4 mm)	3C	17.4	17.7	7.4	11.7	0.63	-37
	4C	17.9					
R-Max Thermasheath-SI (12.7 mm)	5C	18.8	18.2	7.6	11.7	0.65	-35
	6C	17.6					
Thermoply Green (2 mm)	7C	10.9	11.1	4.6	8.4	0.55	-45
	8C	11.3					

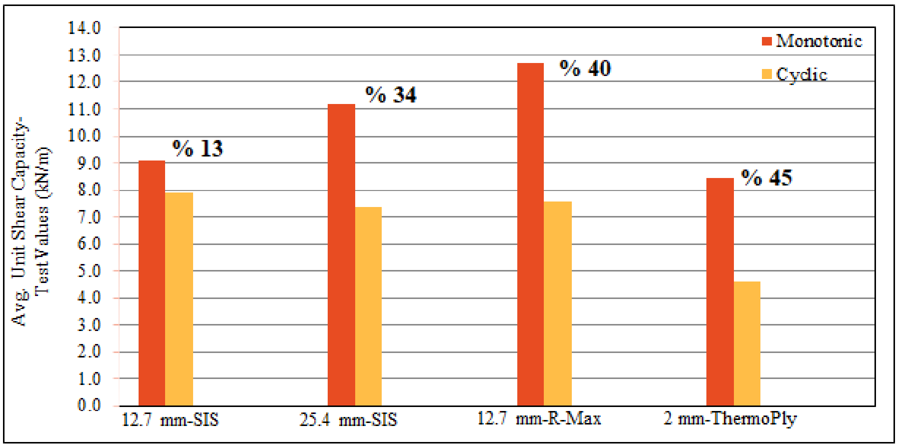


Fig. 3 Test results for monotonic tests in comparison to the cyclic tests

are 27 to 45 percent below the manufacturers published design values. Likewise, the ratios of average test values to the published design values are 0.55 to 0.73.

Fig. 3 shows the test results comparing monotonic and cyclic tests. The average experimental unit shear capacity for the monotonic test was 13 to 45 percent higher than the cyclic test for each series of the tests. Indeed, the load capacity of the tested walls decreased under cyclic load in comparison to the monotonic tests which were intended to represent dynamic and static forces, or seismic and wind loads, respectively. From the test details, Table 1, there is no differences in wall configurations for Series 1 and 2 that showed a 13 percent reduction in shear wall capacity for the cyclic loading (Series 2) in comparison to the monotonic loading (Series1), Fig. 3. While the screw spacing for the gypsum wallboard for the test series subjected to the monotonic loadings (Series 3,5,7) are mostly smaller than the similar series subjected to the cyclic loading (Series 4,6,8). As the gypsum wallboard fastener spacing was increased for the cyclic tests, it could have an effect on the results and reduce the shear wall capacity under cyclic loadings. In addition, the stud spacing

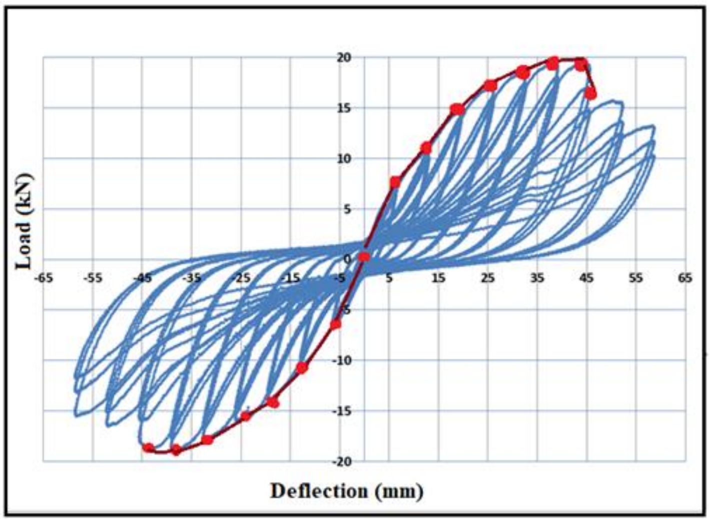


Fig. 4 The typical cyclic test result for Series 2, Test 1C: 12.7 mm (½ in.) SIS

was different for the walls in Series 7 and 8; 40.5 cm (16 in.) and 60 cm (24 in.), respectively. Therefore, besides increasing the gypsum screw spacing, increasing the stud spacing could have an influence on reducing the shear wall capacity within 45 percent (Fig. 3). Note that the reduction in shear wall capacity under cyclic loading has been expected in manufacturers' test details.

Fig. 4 shows a typical cyclic test result which is the observed hysteresis plot for Test 1C of Series 2, with 12.7 mm (½ in.) SI-Strong. There is positive and negative specimen displacement, which is based on the outward (positive) or inward (negative) movement of the hydraulic actuator.

4. Computations

Computations for envelop curves (positive, negative, and average) were conducted for each cyclic test specimen following ASTM E2126 (Sec. 9). The positive and negative (absolute value) envelope curves were graphed to be used in the computation. The average of the positive and absolute value of negative points was reported for the computations. Figs. 5 and 6 show the typical envelope curve and average envelope curve for Test 1C. The envelope curve is graphed by averaging the absolute values of load and displacement for the positive and negative envelope points. The ultimate displacement (Δ_u) is determined from the graph following ASTM E2126. Note that the envelope curve is generated based on the result of testing a similar specimen in the monotonic test (Δ_m), following ASTM E564.

Computations were conducted for seismic performance parameters, cyclic tests. Tables 4-7 show seismic performance parameters calculated for each structural foam panel. The $P_{peak.avg}$ is determined from the average envelope curve. P_{ASD} is the allowable design load based on manufacturers' specifications, wall details. The $\Delta_{ASD,avg}$ is commutated from the envelope curve as the value corresponding to P_{ASD} in the average envelope curve. According to the manufacturers wall details, ASCE 7 (chapters 11 and 12) was used for factor of safety of 2.5 to calculate allowable unit shear capacity, response modification factor (R), over-strength factor (Ω_0),

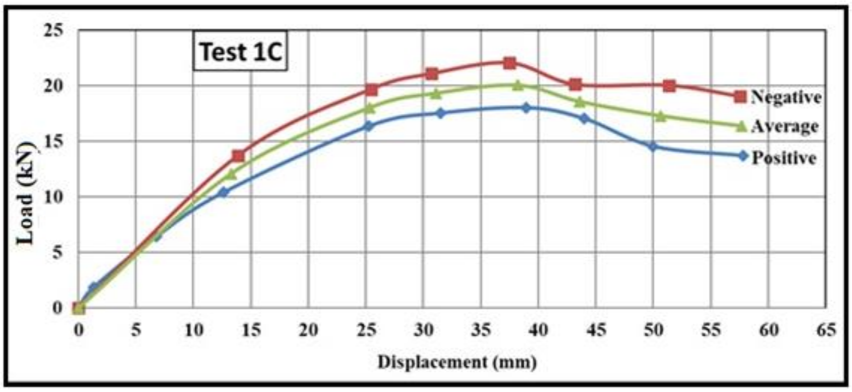


Fig. 5 The typical cyclic test result for Series 2, Test 1C: 12.7 mm (½ in.) SIS

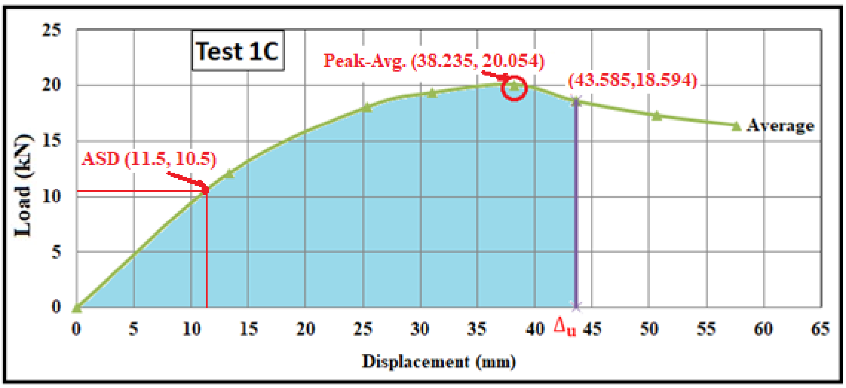


Fig. 6 Typical average envelope curve (Test 1C) used to determine ultimate displacement

deflection amplification factor (Cd: ASCE 7, Section 12.8.6, 12.8.7, and 12.9.2), and height of the structure (ASCE 7. Section 11.2). The values shown in Tables 4-7 indicate that all of the tested foam panels did not meet the criteria for over-strength, drift capacity, and ductility based on ASCE 7 and manufacturers wall details.

An example of the seismic performance parameter computations for Test 1C (Table 4) is followed:

Unit Shear $_{ASD}$: 4.305 kN/m (295 lbf/ft) (Manufacturers wall detail)

P_{ASD} : 4.305 kN/m (295 lbf/ft) x 2.4 m (8 ft)= 10.4978 kN (2360 lbf)= 10.5 kN

Δ_{ASD} : 11.5 mm (0.45 in.) (Envelope curve – Fig. 6)

$P_{peak\text{-}avg}$: 20.054 kN (4508.31 lbf) (Envelope curve – Fig. 6)

Δ_u : 43.585 mm (1.716 in.) (used 43.6 mm) (Envelope curve – Fig. 6)

Over-Strength: Ppeak-avg/ P_{ASD} = 20.054/10.5= 1.91≈ 1.9

Drift Capacity: h= 2.4 m= 2400 mm, $\Delta_u/h =$ 43.6/2400= 0.018, 0.018h

Ductility: Δ_u/Δ_{ASD} = 43.6/11.5= 3.791 ≈ 3.8

Table 4 Seismic performance parameters for 12.7 mm (1/2 in.) Styrofoam SIS

Wall ID	Over-Strength ($P_{peak,avg}$ / P_{ASD})(a)	Drift Capacity ($\Delta_{U,avg}$)(b)	Ductility ($\Delta_{U,avg}$ / $\Delta_{ASD,avg}$)(c)
Test 1C	1.9	0.018h	3.8
Test 2C	1.9	0.018h	3.7
Average	1.9	0.019h	3.7
Criteria	$2.5 \leq P_{Peak,avg} / P_{ASD} \leq 5.0$	$\Delta_{U,avg} \geq 0.028h$	$(\Delta_{U,avg} / \Delta_{ASD,avg}) \geq 11$
Pass/Fail	Fail	Fail	Fail

(a) Ppeak,avg is the average peak load and PASD is the allowable design load (4.3052 kN/m [295 lbf/ft] for the tested wall configuration based on Table 5 of TER-0804-1 [48]).
- See (b) and (c) bellow.

Table 5 Seismic performance parameters for 25.4 mm (1 in.) Styrofoam SIS

Wall ID	Over-Strength ($P_{peak,avg}$ / P_{ASD})(a)	Drift Capacity ($\Delta_{U,avg}$)(b)	Ductility ($\Delta_{U,avg}$ / $\Delta_{ASD,avg}$)(c)
Test 3C	1.7	0.018h	3.0
Test 4C	1.7	0.018h	3.4
Average	1.7	0.018h	3.2
Criteria	$2.5 \leq P_{Peak,avg} / P_{ASD} \leq 5.0$	$\Delta_{U,avg} \geq 0.028h$	$(\Delta_{U,avg} / \Delta_{ASD,avg}) \geq 11$
Pass/Fail	Fail	Fail	Fail

(a) Ppeak,avg is the average peak load and PASD is the allowable design load (4.67 kN/m [320 lbf/ft] for the tested wall configuration based on Table 5 of TER-0804-1 [50]).
- See (b) and (c) bellow.

Table 6 Seismic performance parameters for 12.7 mm (1/2 in.) Rmax Thermasheath-SI

Wall ID	Over-Strength ($P_{peak,avg}$ / P_{ASD})(a)	Drift Capacity ($\Delta_{U,avg}$)(b)	Ductility ($\Delta_{U,avg}$ / $\Delta_{ASD,avg}$)(c)
Test 5C	1.7	0.015h	4.3
Test 6C	1.7	0.016h	4.4
Average	1.7	0.015h	4.3
Criteria	$2.5 \leq P_{Peak,avg} / P_{ASD} \leq 5.0$	$\Delta_{U,avg} \geq 0.028h$	$(\Delta_{U,avg} / \Delta_{ASD,avg}) \geq 11$
Pass/Fail	Fail	Fail	Fail

(a) Ppeak,avg is the average peak load and PASD is the allowable design load ((4.67 kN/m [320 lbf/ft] for the tested wall configuration based on Table 5 of TER-1207-01 [52]).
- See (b) and (c) bellow.

5. Conclusions

Sixteen tests were conducted in this study to investigate the ultimate lateral load capacity of shear walls constructed with several types of structural foam sheathing considering both monotonic and cyclic loadings. The experimental test results were compared to the published design values computed by the manufactures for each test series. In addition, the computation was provided

Table 7 Seismic performance parameters for 2 mm (0.078 in.) Thermo-Ply Green

Wall ID	Over-Strength ($P_{peak,avg}$ / P_{ASD})[(a)]	Drift Capacity ($\Delta_{U,avg}$)[(b)]	Ductility ($\Delta_{U,avg}$ / $\Delta_{ASD,avg}$)[(c)]
Test 7C	1.4	0.024h	3.9
Test 8C	1.4	0.023h	4.0
Average	1.4	0.023h	3.7
Criteria	$2.5 \leq P_{Peak,avg} / P_{ASD} \leq 5.0$	$\Delta_{U,avg} \geq 0.028h$	$(\Delta_{U,avg} / \Delta_{ASD,avg}) \geq 11$
Pass/Fail	Fail	Fail	Fail

(a) Ppeak,avg is the average peak load and PASD is the allowable design load (3.356 kN/m [230 lbf/ft] for the tested wall configuration based on Table 4 of TER-1004-03 [54]).

- (b) and (c) for Tables 4-7:

  (b)$\Delta_{U,avg}$ is the average ultimate displacement and h is the height of the shear wall.

  (c) $\Delta_{ASD,avg}$ is the average displacement corresponding to the allowable design load.

calculating equivalent curve and seismic performance of structural foam shear walls following ASTM E2126 and ASCE 7. Overall, the experimental test values showed:

- Significantly smaller ultimate lateral load capacities for both monotonic and cyclic tests in comparison to the manufacturers commutated and published design values.
- The monotonic test values were 11 to 27 percent lower and the cyclic test values 27 to 45 percent lower than the published design values. Therefore, the published design values are not accurate for the purposes of design in design equations when used with the conditions examined in this study.
- The structural foam panels did not meet the criteria for the over-strength, drift capacity, and ductility for seismic performance.

It should be noted that hold-downs at each end of a wall section are not commonly used in residential building construction. Prior testing at Fears Structural Engineering Laboratory indicates that hold-downs have a positive impact on wall lateral load capacity. Prior research (Pardoen *et al.* 2000, Seaders 2008, Gatto and Uang 2002) has shown that walls with hold-downs at the wall chords, what were described as double terminal studs in this research, have different failure modes when compared to typical residential building construction. Therefore, it is expected that the walls using structural foam sheathing without hold-downs will have a lower lateral load or ultimate unit shear capacity than the walls tested in this investigation with hold-downs. Therefore, it is recommended that further research of structural foam sheathed wood shear walls be performed, with and without hold-downs.

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