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Comparison of structural foam sheathing and oriented strand board panels of shear walls under lateral load

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Abstract. This study performed lateral load testing on seventeen wood wall frames in two sections. Section one included eight tests studying structural foam sheathing of shear walls subjected to monotonic loads following the ASTM E564 test method. In this section, the wood frame was sheathed with four different types of structural foam sheathing on one side and gypsum wallboard (GWB) on the opposite side of the wall frame, with Simpson HDQ8 hold down anchors at the terminal studs. Section two included nine tests studying wall constructed with oriented strand board (OSB) only on one side of the wall frame subjected to gradually applied monotonic loads. Three of the OSB walls were tied to the baseplate with Simpson LSTA 9 tie on each stud. From the test results for Section one; the monotonic tests showed an 11 to 27 percent reduction in capacity from the published design values and for Section two; doubling baseplates, reducing anchor bolt spacing, using bearing plate washers and LSTA 9 ties effectively improved the OSB wall capacity. In comparison of sections one and two, it is expected the walls with structural foam sheathing without hold downs and GWB have a lower wall capacity as hold down and GWB improved the capacity.

Keywords: capacity; design values; shear wall; oriented strand board; structural foam sheathing; monotonic loads, gypsum wallboard

1. Introduction

Most of the wood frame construction in the United States has been sheathed with oriented strand board (OSB) or plywood. OSB and plywood are engineered products with multiple uses such as subflooring, web of wood I-joists, mezzanine decks, and furniture (APA 2018). These panels can be used as structural members for shear walls if they are qualified as premium grade or Structure 1.

There have been extensive experimental, theoretical, and analytical studies on shear walls sheathed with OSB, plywood, and gypsum wallboard (GWB) subjected to static and dynamic loads (e.g. Foschi 1974, Tissel and Elliott 1977, Tuomi and McCutcheon 1978, Foschi 1982, Atherton 1983, Wolfe 1983, Itani and Cheung 1984, Gupta and Kuo 1985, Zacher and Gray 1985,

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Falk and Itani 1987, Adams 1987, Cheung *et al.* 1988, Dolan 1989, Filiatrault 1990, Oliva 1990, Polensek and Schimel 1991, Dolan and Madsen 1992, Karacabeyli and Ceccotti 1996, Karacabeyli *et al.* 1999, McMullin and Merrick 2002, Seaders *et al.* 2009, Memari and Solnosky 2014, Chen *et al.* 2016, Lafontaine *et al.* 2017).

Dolen and Madison (1992) determined that the different type of sheathing, like plywood and waferboard, increased the working stress and ultimate capacity of the shear walls. According to Karacabeyli and Ceccotti (1996), sheathing the second side of the wood frame walls with GWB increased the wall strength and stiffness compared to the walls sheathed with OSB or plywood only on one side of the wall frame. Likewise, Sinha and Gupta (2009) studied wood shear walls sheathed at both sides of the wood frame with OSB and GWB subjected to monotonic loadings. It was found that the GWB fails first, at 60% of the ultimate load capacity of the wall. Then, the OSB panel resisted the load. Similarly, Zhou and He (2011) found that GWB improved the ultimate load, elastic stiffness and energy dissipated of the tested walls. According to Plesnik *et al.* (2016), using intermediate GWB reduced the capacity and stiffness of shear walls. Lafontaine *et. al* (2017) investigated the effect of fastener type, fastener panel edge spacing, and fastener panel edge distance on shear wall capacity sheathed with Type-X GWB and subjected to reversed cyclic loadings. The results indicated the fastener panel edge distance between 9-19 mm (3/8 -3/4 in.) did not influence shear wall response. In addition, wall capacity improved with reduced fastener spacing and the failure mode changed to a brittle failure mode.

Lam *et al.* (1997) showed that oversized panel walls with the conventional nail spacing had a higher load capacity, stiffness, and ductility, but lower deformation than the walls with regular OSB panels. Likewise, Durham *et al.* (2001) indicated walls with large size OSB panels had 26% higher shear capacity and 25% lower maximum deflection than the standard size OSB panels.

Wanyama *et al.* (2012) found the timber-plywood-timber joints showed a higher elasticity than the stiffer control timber-timber joints. Likewise, reducing the nail spacing fastening the plywood panels to the main timber members improved the elasticity of the tested walls.

Dinehart and Shenton III (1998) determined the resistance of walls sheathed with plywood or OSB to lateral loading following ASTM E-564 for monotonic tests and the Structural Engineering Association of Southern California (SEAOSC) fully reversed cyclic test. This study found that the ultimate load capacity of the wall could be closely predicted from monotonic tests. In addition, the design values used in design were much higher than the actual load on a shear wall during an earthquake. This study also investigated sensitivity of wall capacity as a function of wall length. Results indicated the lateral load capacity of walls increased nonlinearly with increasing wall length which supports a conclusion made by Casagrande *et al.* (2016). Other studies on effect of length of a shear wall on wall capacity support that the racking strength and stiffness of a shear wall has an approximately linearly proportional relationship with wall length (Patton-Mollory and Wolfe 1985, Flak and Itani 1987, He *et al.* 1999).

Varoglu *et al.* (2007) investigated a new system in shear wall design. They used the midply shear wall constructed with a sheathing at the center of the wall between pairs of studs. The studs were oriented with a 90° rotation in relation to the studs in a standard shear wall. The midply wall system was compared to the standard shear wall under monotonic and cyclic load using a shake table. In this study, the midply shear walls showed over two and a half times higher resilient capabilities than a standard shear wall when subjected to the applied load.

The Consortium of Universities for Research in Earthquake Engineering (CUREE 1999) workshop conducted experimental studies by testing of full-scale two and three-story buildings, an analytical study referred to as Cyclic Analysis of Shear Walls (CASHEW), and two-dimensional modeling developed by the Seismic Analysis of Woodframe Structures (SAWS) program (Pardoen *et al.* 2000, Folz and Filiatrault 2001, Folz and Filiatrault 2004). In addition, different methods of analysis and models were established to predict wood-frame and shear wall lateral load capacity and provide design equations for engineers to use in the design (Van de Lindt 2004, Van de Lindt *et al.* 2004, Christovasilis *et al.* 2008, Van de Lindt *et al.* 2010, Pei *et al.* 2010, Pang *et al.* 2010, Pei and Van de Lindt 2011, Pang *et al.* 2007, Tomasi and Sartor 2013).

Seaders *et al.* (2008) followed ASTM E564 and CUREE testing methods to determine wall capacity tested with and without hold downs. The walls with hold downs were referred to as fully anchored wood-frame shear walls (Pardoen *et al.* 2000, Gatto and Uang 2002). The walls without hold downs, which are used in residential buildings, were referred to as partially anchored walls. This study showed different failure modes for the fully and partially anchored walls. In addition, the partially anchored walls showed a lower coefficient of variation in cyclic tests compared to the monotonic tests. Likewise, Shipp *et al.* (2000) found that hold down anchors had a minor effect on shear wall capacity subjected to reversed-cyclic loading.

In Atherton (1983), the nail spacing was more effective for increasing the wall capacity than the other variables like sheathing thickness, blocking, and nail patterns, used in this study. Kamiya *et al.* (1996) showed that bolt fasteners had a smaller impact on shear wall response than nail fasteners. Likewise, nail fasteners showed a larger influence than the bolt fasteners on increasing the displacement response of the wall.

According to Rezazadah *et al.* (2016), sill plate failures were the main cause of failure of wood frame buildings during tornados like Moore, 2013. Therefore, using a larger washer size or reducing bolt spacing transforms the failure of the sill plate due to the bending moment on the bottom face to the edge failure. Similarly, Shadravan and Ramseyer (2018) show that wall strength improves with doubling the baseplate, decreasing anchor bolt spacing, and increasing washer size. There have been a number of studies on wall-to-foundation connection (e.g. Marshall 2003, Canfield *et al.* 1991, Vilasineekul 2014, Caprolu *et al.* 2015) indicating metal strapping produces better performance than a typical nailed connection and can be as effective as anchor bolts in connecting the sill to the foundation.

Showalter (2017) provided wood shear wall design examples subjected to wind loads. This article analyzed and compared a wood shear wall per American Wood Council's 2015 Wood Frame Construction Manual (WFCM) and 2015 Special Design Provision for Wind and Seismic (SDPWS). It was found that the 2015 WFCM is an adequate source and time saver for calculating loads and designing wood frame shear walls.

Structural Foam Sheathing has recently been used in place of OSB and plywood in residential construction in the United States. The manufacturers claim that structural foam sheathing provides "four benefits in one product" (OX Engineered Products 2018). These four benefits of structural foam sheathings are: (a) high capacity to resist lateral loads due to wind and seismic loads (b) water resistive-barrier (c) air barrier and (d) high R-values that make them energy-efficient and cost-effective panels.

The manufacturers have computed and published design values for the foam sheathing that indicate the ultimate capacity of the foam panels in a wood frame shear wall structure subjected to wind and seismic forces. It is expected the published design values are to be used in design by engineers. However, there have been numerous studies on wood shear walls paneled with OSB or plywood, and GWB, and no study comparing wood shear walls sheathed with structural foam sheathing to the shear walls with OSB sheathing. Therefore, this research included seventeen tests of shear walls with structural foam sheathing and oriented strand board (OSB) subjected to monotonic (static) loads. Eight shear walls sheathed with four types of structural foam sheathing and nine shear walls sheathed with OSB were tested.

In this study, the methods of ASTM E564 were followed to test the shear walls sheathed with structural foam. The OSB walls were subjected to consistent and gradually applied monotonic (static) loads to the ultimate wall capacity. The basic wall configurations for the structural foam walls and OSB walls followed the manufacturers' wall details and Moore, Oklahoma Adopted New Building Code (2014), respectively.

2. Test protocal

2.1 Test details

Two types of test wall configurations are represented in this study; Section 1: shear walls constructed with structural foam sheathing and GWB and Section 2: shear walls constructed with OSB.

2.1.1 Section 1: Shear walls constructed with structural foam sheathing and gypsum wallboard

The test specimens were 2.4 × 2.4 m (8 × 8 ft) walls constructed with 2 × 4 dimension grade Fir lumber (38 × 88 mm [1.5 × 3.5 in.]) used for the studs, top, and baseplates. The walls were framed with 40 cm (16 in.) stud spacing, with double terminal studs, double top plates, and a single baseplate. The walls were fully sheathed on both sides of the stud wall frame: the structural foam sheathing on one side and GWB on the opposite side of the stud frame. The foam sheathing types studied included: 12.5 mm (1/2 in.) SI-Strong (SIS), 25 mm (1 in.) SI-Strong (SIS), 12.5 mm (1/2 in.) R-Max Thermasheath-SI, and 2 mm (0.078 in.) ThermoPly Green. The GWB was 12.5 mm (1/2 in.) thick. The foam sheathing and GWB were fastened to the stud wall frame with an edge distance of 10 mm (3/8 in.) following the manufacturers' test detail. Table 1 indicates the test details for wall tests in this section.

Two 16d box nails (3.3 × 75 mm [0.131 × 3 in.]) were used to fasten the top plate and baseplate to the studs through end-nailing. One 10d box nail (3.2 × 75 mm [0.128 × 3 in.]) was used to fasten the outer top plate to the lower top plate with 600 mm (24 in.) spacing on center. Likewise, the 10d box nails were used to fasten the doubled terminal studs at each end of the stud wall with 150 mm (6 in.) spacing on center. The GWB was fastened to the wall studs with #6 Type W 32 mm (1-1/4 in.) long screws. All fasteners of a given type were from the same box according to ASTM F1667, as stated by the nail manufacturer. The structural foam sheathing was stapled to the wall stud using 16-gauge staples with crown and leg and spacing matching the manufacturers' recommendations and are provided in Table 1. Two walls were tested for each wall configuration in this section.

The wall frames were anchored to a steel beam (W10×39), bolted to the test floor, with (3) 16 mm (5/8 in.) anchor bolts with 1.0 m (3.5 ft) nominal spacing and 75 × 75 × 6.3 mm (3 × 3 × 0.25 in.) bearing plate washers. Wall frames were tied down to the base with Simpson HDQ8 tie down anchors using 16 mm (5/8 in.) anchor bolts at each end of the wall frame. The Simpson tie downs were also screwed to the terminal studs with (14) 6.3 × 75 mm (1/4 × 3 in.) SDS screws.

2.1.2 Section 2: Shear walls constructed with oriented strand board (OSB)

Moore, Oklahoma is one of the cities that has been struck by many tornados over the years since 1893 (National Weather Center record from 1893-2015). The most massive and powerful

Table 1 Test details- structural foam sheathing

Section 1	Sheathing type, fastener type, and spacing			
Test No.	Type of sheathing (thickness - mm)	Type of staple (mm)	Panel fastener spacing edge/field (mm/mm)	GWB screw spacing edge/field (mm/mm)
1 2	SIS/SI-Strong (12.5 mm)	Crown:25 Leg: 38	75/75	400/400
3 4	SIS/SI-Strong (25 mm)	Crown:25 Leg: 50	75/75	200/200
5 6	R-Max Thermasheath-SI (12.5 mm)	Crown:12.5 Leg: 38	75/150	100/400
7 8	Thermoply Green (2 mm)	Crown:25 Leg: 38	75/75	200/200

EF5 tornado hit this city in May 2013 (Insurance Journal-2013) causing loss of multiple lives and billions of dollars in damage. After this deadly tornado, the Moore City Council adopted the New Building Codes based on research and a proposal by Ramseyer *et al.* (2015). According to the New Building Code adoption, the shear wall configuration consists of a structural sheathing panel (OSB or plywood), studs spacing of 40 cm (16 in) on center, panel fasteners of 8d (60 × 2.85 mm [2-3/8 × 0.113 in.]) ring shank nails, and 100 mm/150 mm (4 in./6 in. - edge/field) nail patterns (City Adopts New Building Code, 2014).

In this section, the test wall configurations followed the Moore New Building Code adoption (2014). The results from this section (OSB walls) are compared to Section 1 (structural foam sheathing walls) in this study. In this section, nine test specimens were subjected to monotonic (static) load. The wall specimens were 3.6 × 2.4 m (12 × 8 ft) and constructed with 2×4 dimension grade Fir lumber (38 × 88 mm [1.5 × 3.5 in.]) for the studs, top and baseplates. Two walls were tested for each wall configuration for the walls without tie (strap). All the walls were constructed with single terminal studs, double top plates, and single or double baseplates. The wall frames were fully sheathed on one side with 11.0 mm (7/16 in.) thick and 1.2 × 2.4 m (4 × 8 ft) OSB panels. The panels were fastened to the studs with 8d, 60 × 2.85 mm (2-3/8 × 0.113 in.) ring shank nails, at a 100 mm/150 mm (4 in./6 in.) edge to field nail patterns following the Moore New Building Code adoption (2014). The edge distance of the bottom row nails was kept 19 mm (3/4 in.) from the bottom edge of the panel for the minimum required edge distance based on NDS-SDPWS (2014), Sec. 4.2.7.1.3. A staggered nail pattern was used to fasten the panel to the double baseplates while one row of nails was used to fasten the panel to the doubled top plate. The studs were end nailed to the top and bottom baseplates with (2) 16d, (3.3 × 75 mm [0.131 × 3 in.]), smooth shank nails. The second top and baseplate (for double baseplates) was fastened to the first plate with (2) 16d, (3.3 × 75 mm [0.131 × 3 in.]), smooth shank nails with 40 cm (16 in.) spacing, at the location of studs. The 12.5 mm (1/2 in.) anchor bolts were used to anchor the walls to the steel beam (W10×39), which was bolted to the lab floor. The Simpson LSTA 9 ties (straps) were fastened to the studs and baseplates with N8 Simpson nails.

Table 2 Test details- OSB sheathing

Section 2		Strap	Sheathing fastener type & spacing		12.5 mm Anchor bolt		Base plate
Test No.	Wall length (m)	Type of tie (strap)	Panel fastener spacing edge/field (mm/mm)	Type of nail	Nominal spacing (m)	Washer type	Single/ Double S/D
9	3.6	---	100/150	8d Ringshank (60 × 2.85 mm)	1.8	Std.	S
10							
11					1.8	BP	D
12							
13					0.9	BP	D
14							
15		LSTA9 Tie	100/150	8d Ringshank (60 × 2.85 mm)	1.8	Std.	S
16						BP	S
17						BP	D

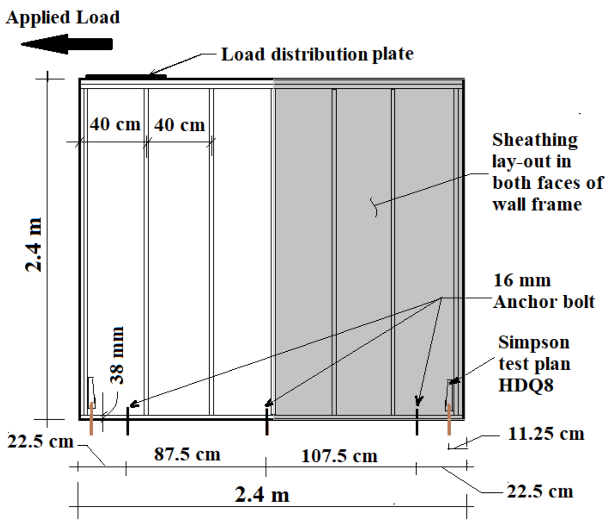
Table 2 indicates test variables for the nine tests in this section. The variables in this section included: (a) number of baseplates: single or double (b) size of washer: standard cut round washer with 35 mm (1.375 in.) diameter and 76 × 76 × 6.4 mm (3 × 3 × 0.25 in.) bearing plate square washer referred to as BP in Table 2 (c) anchor bolt spacing and (d) using tie (strap). From prior testing at Fears Structural Engineering Laboratory, Simpson LSTA 9 tie improved the wall lateral load capacity more than other ties tested. Therefore, LSTA 9 tie (strap) has been used in this study to compare with hold down strong tie used in Section 1.

3. Test setup

3.1 Section 1: Structural foam sheathing and gypsum wallboard wall tests

Fig. 1(a) shows the schematics of a typical shear wall with 40 cm (16 in.) stud spacing and sheathing lay-out on both faces of the wall. An 89 kN (20 kips) hydraulic cylinder was mounted on a reaction frame and used to apply lateral loading in plane of the walls and along the top of the walls through a 60 cm (2.4 ft) load distribution plate. The load distribution plate was bolted to the top plates of the wall with (4) 8 mm (5/16 in.) bolts. An HSS 4×2 steel column was used to keep the walls in-plane with the applied load (Fig. 1(b)).

The test method followed ASTM E564, Section 7. In this method, the wall is loaded and unloaded in a series of increasing increments until the ultimate wall capacity is reached. Approximately 10%, 30%, 60% of the estimated ultimate load, was applied for each loading level. The load was removed at each level, and the wall was remained unloaded for 5 minutes to allow for wall recovery. Then, the wall was reloaded to the next level of loading. After loading and unloading the 60% of ultimate load level, the loading continued until the ultimate capacity of the wall was reached. The lateral load was measured with a load cell located in-line with the hydraulic cylinder. Displacement of the top of the wall was measured using a string potentiometer. Data acquisition was used to simultaneously gather data for the racking load and displacement of the top of the wall.



(a) Schematic of typical wall and sheathing lay-out

(b) Loading mechanism and out-of-plane support frame

Fig. 1 Schematics of a typical shear wall

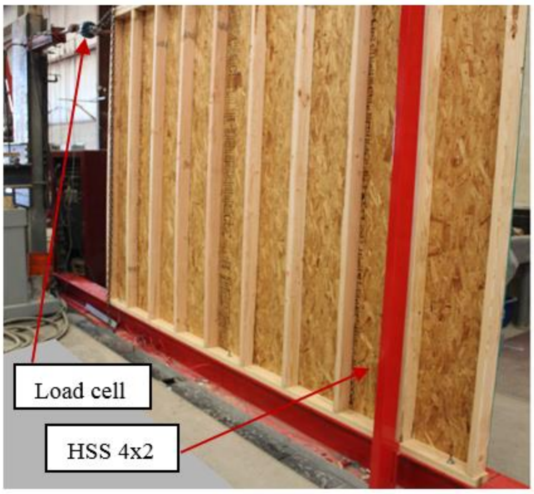
3.2 Section 2: OSB panel wall tests

Figs. 2(a)-(b) shows a typical wall specimen without and with LSTA 9 tie (strap). The walls were fully covered with OSB only on one side of the wood frame wall with 40 cm (16 in.) stud spacing. The walls were subjected to lateral loading by an 89 kN (20 kip) hydraulic cylinder mounted onto a reaction frame (Fig. 2). The hydraulic cylinder pulled along the top of the wall and in the plane of the wall through a 60 cm (2.4 ft) load distribution plate bolted with (4) 8 mm (5/16 in.) bolts to the top plates of the wall to distribute the lateral load, similar to the walls in Section 1. An HSS 4×2 steel column was used to keep the walls in-plane with the applied load (Fig. 2).

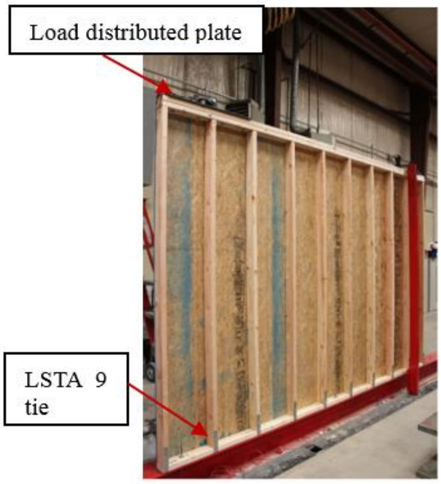
Each wall was subjected to a gradual and in-plane lateral load applied continuously until reaching to the ultimate capacity of the wall referred to as the wall failure point or the peak load. Then, loading was continued until the displacement at the top of the wall exceeded the allowable story drift, which was computed based on ASCE 7 (Eq. 1). After displacement reached about 75 mm (3 in.) at the top of the wall, the loading was stopped, and the wall was unloaded to zero. The applied load and displacement on the top of the wall (deflection) were measured by a load cell located in-line with the hydraulic cylinder and a string potentiometer that was mounted onto the reaction frame and connected to the top of the wall, respectively. The data from these instruments were collected in a data acquisition program.

According to ASCE 7-10, Section 12.12, for Risk Categories I or II; the allowable drift for light-frame shear wall buildings is calculated using (Eq. 1)

$$0.02\ h = 0.020 \times (2.4\ m \times 1000\ mm/m) = 48\ mm\ (1.9\ in.) \tag{1}$$



(a) OSB wall without tie



(b) OSB wall with LSTA 9 tie on each stud

Fig. 2 Typical OSB wall

From ASCE 7-10, Section 12.12: "The design story drift (Δ) as determined in Sections 12.8.6, 12.9.2, or 16.1, and shall not exceed the allowable story drift (Δ_a) as obtained from Table 12.12-1 for any story."

4. Test results

According to the prior studies described in the introduction, wall capacity improves with increasing wall length. As the wall length is 2.4 m [8 ft] in Section 1 and 3.6 m (12 ft) in Section 2, unit shear capacity is considered for comparison in test results due to the linear relationship between the wall capacity and its length (Patton-Mollory and Wolfe 1985, Flak and Itani 1987, He *et al.* 1999).

4.1 Section 1: Shear walls constructed with structural foam sheathing and gypsum wallboard

Table 3 indicates the test results for wall tests 1-8, structural foam sheathing walls. Two tests were conducted for each wall configuration following ASTM E564. Note that the load-displacement relationship is not applicable for this section due to the method of load application. Averages of the test results for each pair of tests were computed and compared to the published design values computed by manufacturers. The average monotonic test values for a pair of tests are 11 to 27 percent below the manufacturers published design values. Fig. 3 shows the variation between the average test values and published design values.

In addition, Table 3 includes the displacement of the top wall (deflection) and stiffness calculations at peak load. The average displacement at peak loads are less than the allowable drift (48 mm) computed in section 3.2. this manuscript following ASCE 7 Sec. 12.12. The stiffness is computed as ratio of average peak load to the average displacement at peak load. It is noticeable that the higher the wall capacity was, the higher the wall stiffness at peak load was.

Table 3 Summary of results for Section 1, structural foam sheathing walls

Test No.	Test value peak load (kN)	Average test value (kN)	Displacement at peak load (kN)	Average displacement at peak load (mm)	Stiffness at peak load (kN/mm)	Average test value- unit shear capacity (kN/m)	Published design value- ultimate unit shear (kN/m)	Difference between avg. test value and published value (%)
1	20.5	21.8	60.2	45.4	0.48	9.1	10.8	-16
2	23.2		30.7					
3	25.5	26.8	29.0	30.5	0.88	11.2	13.9	-20
4	28.0		31.9					
5	30.2	30.5	31.6	32.3	0.94	12.7	14.1	-11
6	30.8		32.9					
7	21.0	20.3	41.4	44.9	0.45	8.4	11.5	-27
8	19.5		48.3					

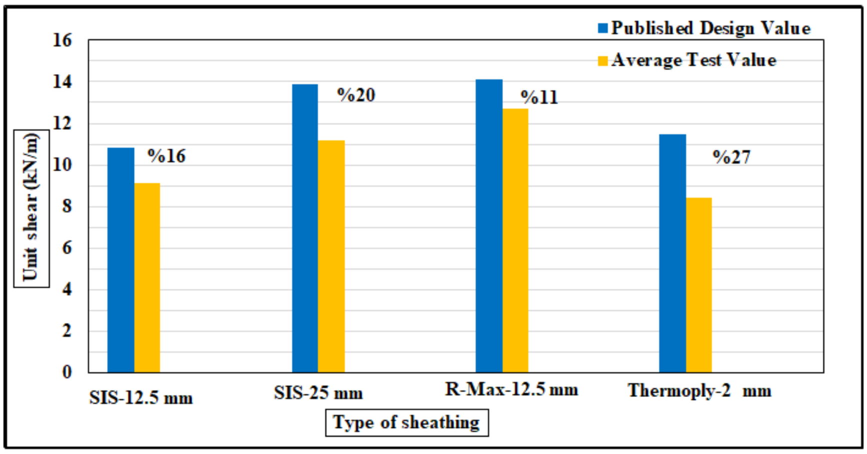


Fig. 3 Published design value vs average test value for structural foam sheathing walls

4.2 Section 2: Shear walls constructed with OSB

Table 4 summarizes the test results for Section 2 in this study (tests 9-17). This table includes peak load, displacement of top of the wall (deflection) at peak load and stiffness at peak load. As two walls were tested for each configuration of the walls without LSTA 9 tie, the average monotonic (static) test values, deflection and stiffness have been indicated for a pair of tests in Table 4. The stiffness is defined as ratio of the peak load to the corresponding deflection at peak load. In addition, Table 4 indicates that deflection at peak loads are less than the allowable drift (48 mm) limits which has been computed in section 3.2. this manuscript, following ASCE 7 Sec. 12.12. However, the average deflection of wall 13 and 14 exceeded the allowable drift at peak load. Note that walls 13 and 14 with doubled base plates, reduced anchor bolt spacing, and with bearing plate washer, had the highest wall capacity in tested OSB walls without tie. Likewise, the tied OSB wall 17 with doubled base plates and bearing plate washer showed the highest wall capacity for tested OSB walls in Section 2. Furthermore, the walls with doubles base plates and bearing plate washer showed a higher stiffness at peak load than the other wall configuration in this section.

Table 4 Summary of results for Section 2, OSB walls

Test No.	Nominal anchor bolt spacing (m)	Washer type	Single/ Double S/D	Peak load (kN)	Avg. peak load (kN)	Deflection at peak load (mm)	Avg. deflection at peak (mm)	Stiffness at peak load (kN/mm)	Unit shear (kN/m)
9	1.8	Std.	S	14.2	14.8	29.5	29.2	0.50	4.1
10				15.4		29.0			
11	1.8	BP	D	26.8	25.8	47.0	35.5	0.72	7.2
12				24.7		24.0			
13	0.9	BP	D	37.9	35.1	43.7	48.9	0.71	9.8
14				32.4		54.1			
15	1.8 & LSTA9 Tie	Std.	S	23.7	23.7	35.3	35.3	0.67	6.6
16		BP	S	26.0	26.0	42.4	42.4	0.61	7.2
17		BP	D	39.0	39.0	45.2	45.2	0.86	10.8

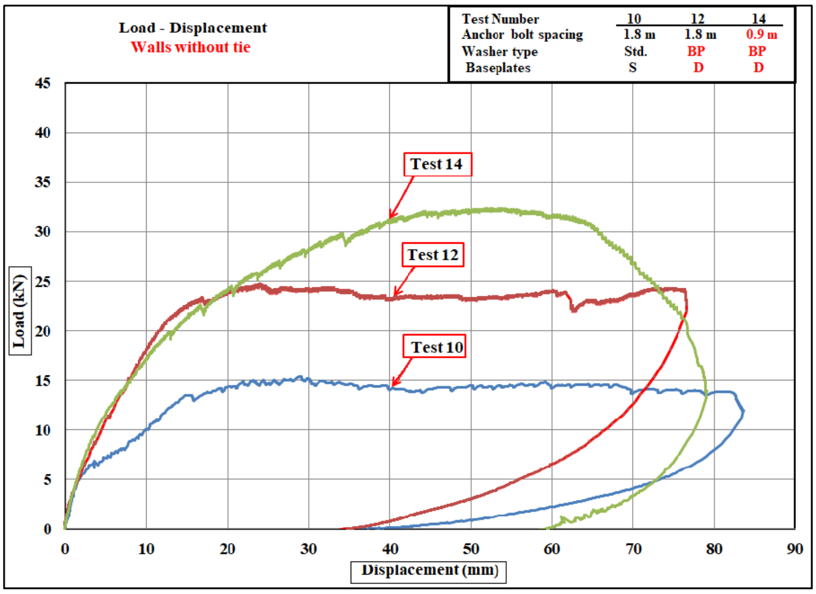


Fig. 4 Effect of bearing plate washer, doubled baseplate and decreased anchor bolt spacing

Fig. 4 shows load-displacement graph and effect of bearing plate washer (BP), double baseplate (D), and decreased anchor bolt spacing on unit shear capacity of the walls without tie (LSTA9 tie). However, two walls were tested for each wall configuration without tie, providing average data for the entire graphs (results) was not applicable. Therefore, one wall was graphed from each series in this section; walls 10, 12, and 14. It is noticeable that doubling base plate and using bearing plate washer improved the wall capacity (Test 10 vs 12). Likewise, reducing anchor bolt spacing enhance the wall capacity (Test 12 vs 14).

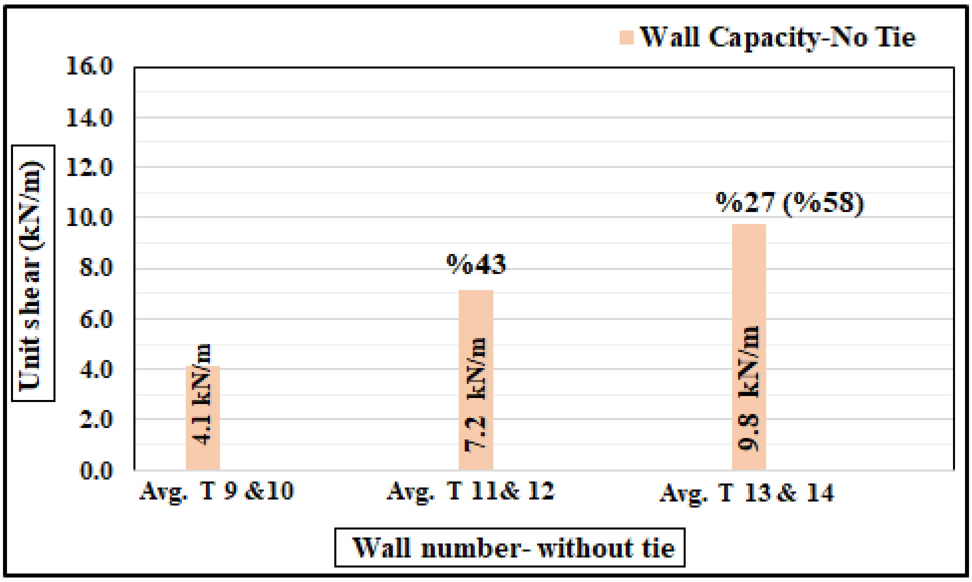


Fig. 5 Average wall peak loads: effect of bearing plate washer, doubled baseplate and decreased anchor bolt spacing

Fig. 5 also indicates the influence of bearing plate washer, doubled baseplate and decreased anchor bolt spacing on unit shear capacity for the walls without ties considering the average unit shear at peak loads for each series of the test walls. Note that "Test" is referred to as "T" in the following figures with column chart. Fig. 5 shows that doubled baseplate with bearing plate washers improved the wall capacity by 43% (3.1 kN/m [212 Ibs/ft]) compared to the base case of a single baseplate and standard washers (Avg. Tests 9 and 10). Likewise, reducing anchor bolt spacing improved the wall capacity by 27% (2.6 kN/m [178 lbs/ft])- compared to just doubling the baseplate and using bearing plate washers. Note that doubled baseplates, bearing baseplate washers and decreased anchor bolt spacing improved unit wall shear capacity by 58% (5.7 kN/m [374 lbs/ft]) compared to the base case.

Figs. 6-7 show the effect of Simpson LSTA9 (23 cm [9 in.] long) ties (straps) on unit shear wall capacity. Fig. 6 shows Test 10 in comparison to the strapped walls and Fig.7 indicates average unit shear walls without tie (strap) at peak loads in comparison to the strapped walls. The results indicate adding LSTA 9 tie (strap) enhanced the wall unit shear capacity by 38% (2.5 kN/m [171lbs/ft]) compared to the base case (Test 15 vs Avg. Tests 9 and10). In addition, adding bearing plate washers to the wall with straps improved the wall unit shear capacity by 8.9% (0.6 kN/m [41 lbs/ ft]) (Test 15 vs 16). Similarly, doubling the base plate for the wall with straps with bearing plate washers improved the wall unit shear capacity by approximately 32% (3.6 kN/m [247 lbs/ft]), (Test 16 vs 17). These results support the findings from Section 2 shown in Figs. 4-5.

Fig. 8 indicates a comparison between the test results for OSB walls without and with LSTA 9 ties. Walls with a doubled baseplate, bearing plate washers, and decreased anchor bolt spacing without and with LSTA 9 ties (Avg. Tests 11&12 and Test 17), showed the highest wall capacity for the walls in Section 2.

Test wall 16 with LSTA 9 ties and single baseplate had a 7.2 kN/m (493 lbs/ft) wall unit shear capacity, which was equal to the average unit shear wall capacity for Tests 11&12 with doubled

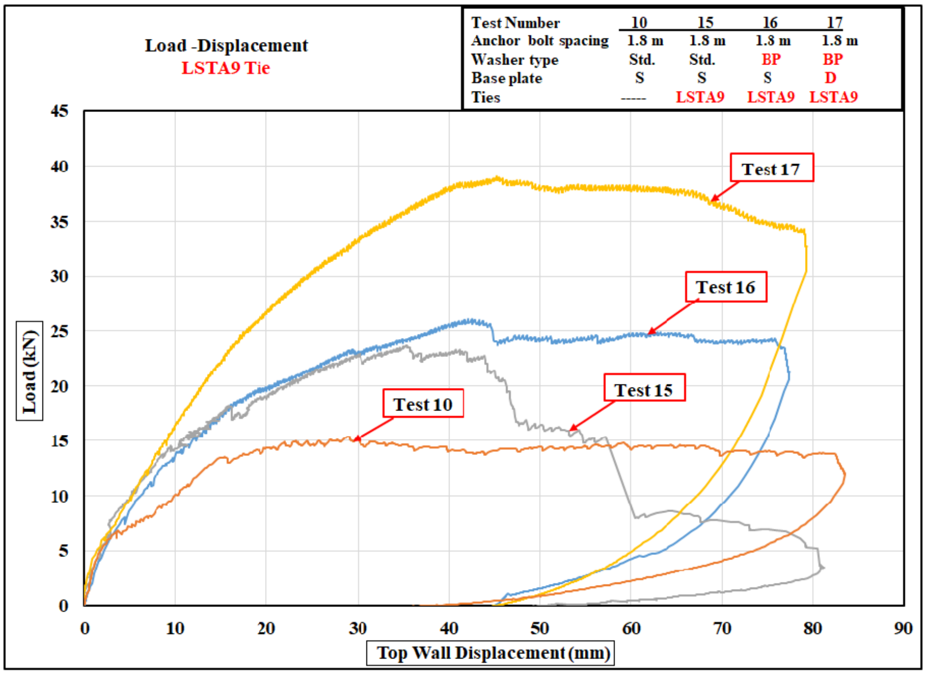


Fig. 6 Effects of adding tie, bearing plate washer, and doubled baseplate on shear wall capacity for walls with LSTA9 ties

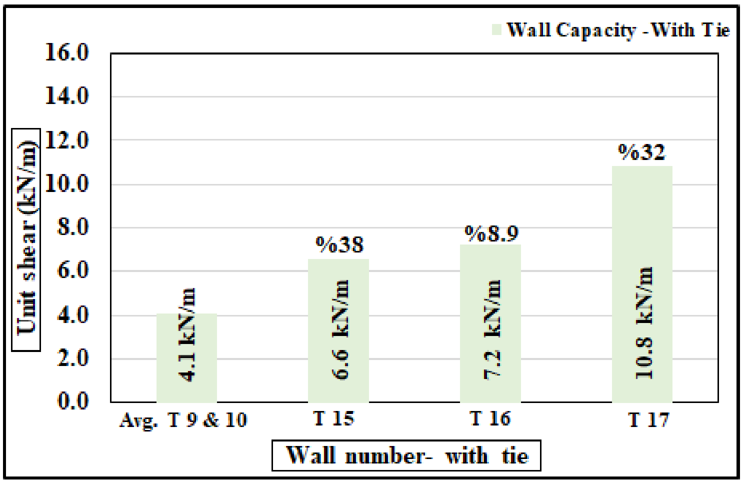


Fig. 7 Effects of adding tie, bearing plate washer, and doubled baseplate on shear wall capacity for walls with LSTA9 ties using Avg. Tests 9 and 10

baseplate. Average wall unit shear capacity for Tests 13 &14 with increased number of anchor bolts had a 9.8 kN/m (672 lbs/ft) unit shear capacity, which was higher than strapped wall Test 16 and very close to the strapped wall Test 17. Consequently, doubling baseplates with bearing plate washers and increasing number of anchor bolts showed an effective method to increase unit shear wall capacity for the tested walls in this study.

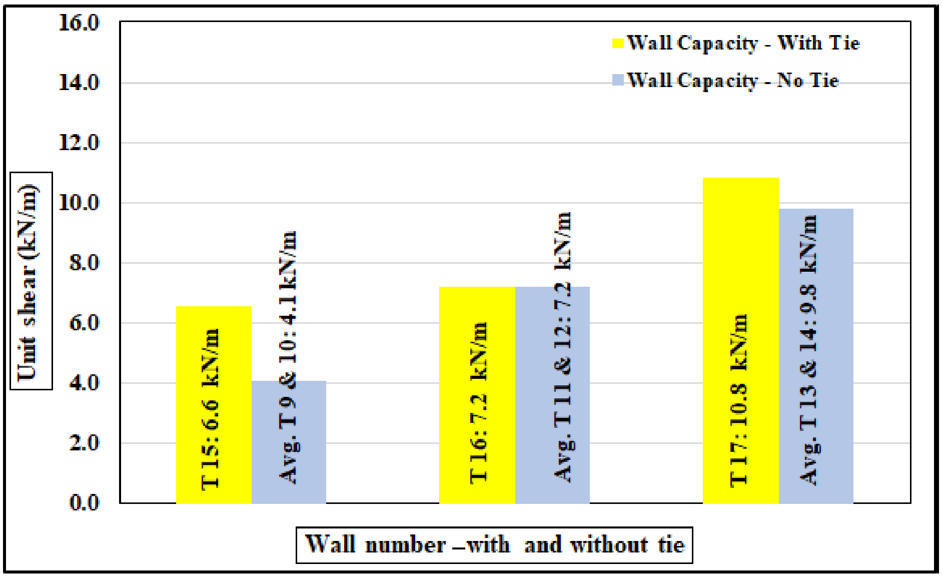


Fig. 8 Comparison wall without and with LSTA 9 tie

4.3 Analytical results

4.3.1 Code design values computation

Nominal and allowable wall capacities were determined for Section 2 for a base wall (Tests 9 &10) following NDS, Special Design Provisions (SDPWS 2015) for wind, Section 4.3., Table 4.3.A. The tested shear walls were unblocked. Therefore, an adjustment factor, C_{ub} from Table 4.3.3.2. (SDPWS 2015) was applied to the values in Table 4.3A which is applicable for blocked wood frame (SDPWS 2015- Equation 4.3.3.2), Eq. 2:

$$V_{ub} = V_b\, C_{ub} \qquad \text{(SDPWS 2015- Equation 4.3.3.2)} \qquad (2)$$

Where V_b is represented nominal unit shear capacity (Ibs/ft) for blocked shear walls obtained from Table 4.3A, C_{ub} is unblocked shear wall adjustment factor based on Table 4.3.3.2, and V_{ub} is nominal unit shear capacity for unblocked shear wall (Ibs/ft).

Example of computation (NDS SDPWS 2015):

Tabulated nominal unit shear capacities for wind design (V_w) is provided in Column B of Table 4.3A, NDS SDPWS (2015) and is referred to (V_b) in Eq. 2. Based on Column B of Table 4.3A, the nominal unit shear capacity for shear wall subjected to wind (V_w) equals to 14.3 kN/m (980 Ibs/ft) which is specified for 11.0 mm (7/16 in.) OSB thickness, 8d nail fasteners, and 100 mm (4 in.) panel edge fattener spacing.

From Table 4.3.3.2 NDS SDPWS, the unblocked adjustment factor (C_{ub}) approximately equals to 0.8 ($C_{ub} \approx 0.8$) for 150/150 mm (6 in./6 in.) panel nail spacing. Therefore, the nominal unit shear capacity for unblocked shear wall is calculated, Eq. 2:

$V_{ub} = 14.3 \times 0.8 = 11.44$ kN/m

According to Section 4.3.3 NDS SDPWS, the Allowable Strength Design (ASD) reduction factor of 2 shall be applied to the tabulated nominal unit shear capacity. Therefore, the unit shear capacity of walls is obtained from Eq. 3:

Unit shear capacity of wall (Code design value); 11.44 ÷ 2= 5.72 kN/m (3)

It is noticeable that the unit shear capacity of the test walls from Table 4 exceeded the computed code design values (5.72 kN/m) within a 13 to 38 percent, except unit shear capacity for average Tests 9 &10 which was the weakest wall in this study. Likewise, Fig. 9, which will be discussed in Section 4.3. this manscrip, indicates the ultimate load test value for the tested walls exceeded the code design values, except the base wall (Avg. Tests 9 &10). Note that the OSB walls do not include corner tie downs in this study.

4.3.2 Code Deflection Values Computations

In this section, maximum shear wall deflection (δ_{sw}) is computed based on elastic analysis following SDPWS 2015, Section 4.3.2, Equation 4.3-1. As this equation is based on U.S. unit, it is computed as U.S. unit originally (Eq. 4). Then, it is converted to SI unit in this manuscript.

$$\delta_{sw}=\frac{8vh^3}{EAb}+\frac{vh}{1000G_a}+\frac{h\Delta_a}{b} \text{ (SDPWS, Equation 4.3-1)} \tag{4}$$

Where G_a is represented shear stiffness and obtained from NDS SDPWS 2015, Table 4.3A-Column A which is 22 kips/in. (3.85 kN/mm) for wall with panel edge fastener spacing of 4 in. (100 mm). Δ_a is represented total vertical elongation of wall anchorage system and assumed 1/8 in. (3 mm) according to Design of Wood Structures ASD/LRFD (7th Edition), Chapter 10. v is represented induced unit shear at peak, Ibs/ft. The induced unit shear shall be divided by adjustment factor, $C_{ub} \approx 0.8$ (SDPWS 2015, Table 4.3.3.2.) for unblocked shear wall.

h and b are represented height and length of the wall, respectively, ft. E and A are represented modulus of elasticity of end posts and area of end post cross-section for 2×4 chords, respectively. δ_{sw} is maximum shear wall deflection (in.) determined by elastic analysis from Eq. 4.

A deflection amplification factor C_d from ASCE 7, Table 12.2-1 is multiplied by computed δ_{sw} (Eq. 5). This factor for wood-frame walls is 4 (Cd = 4), and the resulting deflection is:

$$\Delta= C_d\delta_{sw} \tag{5}$$

Here is an example of calculations for Test 9 in Table 5, using Eq. 4:
Where G_a: 22 kips/in. (3.85 kN/mm), Δ_a: 1/8 in (3 mm), peak load: 3182 Ibs (14.2 kN (Table 4), unit shear (load/ wall length): 3182 ÷ 12 = 265.15 Ibs/ft (3.87 kN/m), and v induced unit shear at peak load: 265.15 ÷ 0.8 = 313.46 Ibs/ft (4.57 kN/m) (SDPWS 2015, Section 4.3.2.2 and Table 4.3.3.2.).

In addition, h is 8 ft (2.4 m), b is 12 ft (3.6 m), E is 1,600,000 psi (11.03 $\frac{kN}{mm^2}$), and A is 5.25 in.^{2} (1.5 × 3.5 = 5.25 in.^{2} [3281 mm^2]) in Eq. 4.

From using the above quantities in Eq. 4, δ_{sw} is obtained as

$$\delta_{sw} = 0.2173\ in.$$

From Eq. 5; $\Delta = C_d\delta_{sw}$ = (4) (0.2173) = 0.869 in. (21.73 mm)

Table 5 indicates shear wall deflection (Δ), test load at computed deflection, unit shear and stiffness at computed deflection. Stiffness is defined as ratio of load (kN) at computed deflection to the computed deflection (Δ). It is noticeable that the computed maximum shear wall deflection determined by elastic analysis (Δ) was less than the allowable drift 48 mm (1.9 in.) for shear wall buildings in this study- Section 2. Likewise, deflection at peak load for the tested walls in Table 4 exceeded the computed shear wall deflection from Table 5.

Table 5 Computed deflection (Δ) value - SDPWS 2015, Section 4.3.2.

0	Computed shear wall deflection Δ (mm)	Avg. computed shear wall deflection (mm)	Load at computed deflection (kN)	Avg. load at computed deflection (kN)	Unit shear at computed deflection (kN/m)	Stiffness (kN/mm)
9	21.73	22.32	10.92	12.90	3.58	0.58
10	22.91		14.88			
11	33.72	32.71	25.06	24.61	6.84	0.75
12	31.71		24.16			
13	44.17	41.59	36.60	33.84	9.40	0.81
14	39.00		31.09			
15	30.78	30.78	22.41	22.41	6.22	0.73
16	32.98	32.98	23.86	23.86	6.63	0.72
17	45.27	45.27	38.37	38.37	10.66	0.85

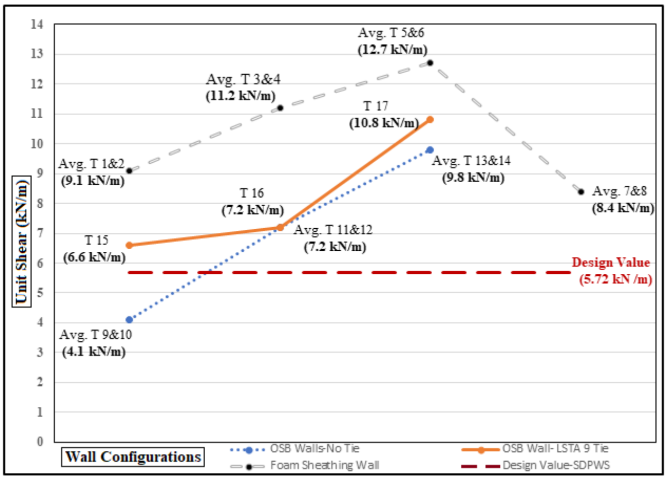


Fig. 9 Comparison OSB walls with structural foam wall (Test results: Section 1 vs Section 2)

4.4 Section 1 vs Section 2: Shear walls constructed with OSB compared to structural foam sheathing

Figure 9 indicates the results for shear walls sheathed with structural foam sheathing in comparison with the shear walls sheathed with OSB with and without LASTA9 ties. Note that hold down anchors were used for the wall tests in Section 1, strongly anchored the wall frame at the base. Therefore, the LSTA9 tie was selected as a strong tie, determined by prior testing at the Fears Lab, for wall tests in Section 2. From Fig. 9, Test 17 with LSTA 9 tie shows higher unit shear wall capacity than the walls with 12.5 mm (1/2 in.) SIS and Thermoply Green, and very close to 25 mm (1 in.) SIS. Test 17 and Avg. Tests 13 & 14 (no tie) showed higher unit shear wall capacity than 2 mm (0.078 in.) Thermoply Green sheathing. Likewise, all unit shear capacity of the tested walls in this study exceeded the code deign values (5.72 kN/m [392 Ibs/ft]), except Avg. Tests 9 &10 which was the weakest wall in this study.

5. Discussion: failure modes

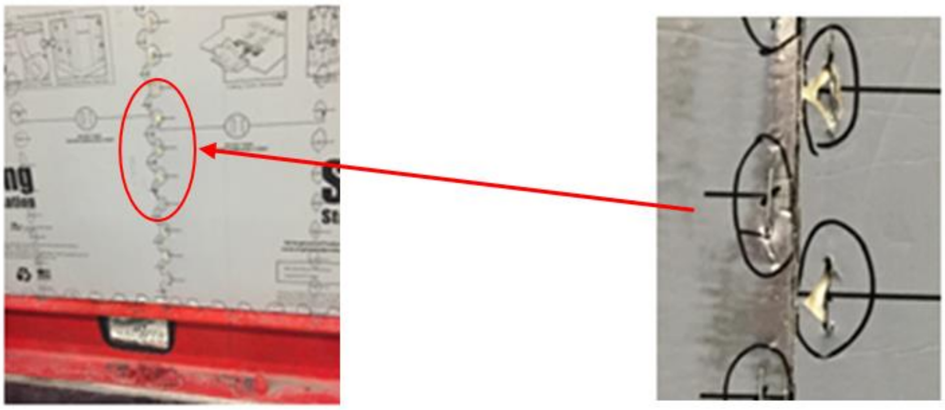
5.1 Section 1: Shear walls constructed with structural foam sheathing and gypsum wallboard

The failure mode for walls in Section 1 can be categorized as: (a) shear panel failure of staples bearing through the structural foam sheathing, (b) shear panel failure of screws bearing through the GWB, or (c) combination of these failure modes (Figs. 10-11). As hold downs strongly anchored the wall frame at the base, none of the walls failed due to withdrawn nails at the frame connection between the end studs and baseplate at far end from the load, baseplate fracture or bending, anchor bolt failure, or hold downs failure. Fig. 10(a) shows the shear wall after test; the GWB was cut to see the frame, which supports this discussion.

(a) Bottom of wall frame after testing

(b) Failure of GWB due to bearing of screws through the GWB

Fig. 10 Failure of test walls- Section 1



(a) Failure of structural foam panel due to bearing of staples through the pane

(b) Failure due to bearing of staplels at the edges

Fig. 11 Failure of test walls- Section 1

5.2 Section 2: Shear walls constructed with OSB

The failures of the test walls in Section 2 were due to: (a) panel tearing out, (b) shear panel failure due to bearing of the nails through the panel, (c) baseplate shearing, (d) bending in the baseplate, (e) failing of the frame connection between the end studs and baseplate at far end from the load, (f) withdrawing the tie (strap) nails, or (g) a combination of several of these failure modes (Figs.12- 13). Note that the baseplates did not bend in the wall with decreased anchor bolt spacing (Tests 13 and 14) and none of the walls failed due to anchor bolt failure. The failure modes (c, d, e, and f) that occurred for the walls in this section did not happen for the walls in Section 1, which supports results of previous studies comparing OSB walls with and without hold downs (Pardoen *et al.* 2000, Seaders 2008, Gatto and Uang 2002).

(a) Bent baseplates

(b) Failed frame connection and LSTA 9 tie fastener nail withdrawn

Fig. 12 Failure modes – Section 2

(a) Torn out OSB, withdrawn nails and combination of several failure modes

(b) Torn out OSB, fractured baseplate and combination of several failure modes

Fig. 13 Failure modes – Section 2

6. Conclusion

Overall, the experimental and analytical results in this study concluded:

• The experimental test values for the walls with structural foam sheathing showed an 11 to 27 percent smaller capacity compared to the manufacturers published design values for monotonic tests.

• Therefore, the published design values are not accurate to be used by engineers in design equations for wind loads for the tested walls in this study.

• The OSB wall results showed a 13 to 38 percent increase in unit shear wall for most of the tested walls in this study in comparison to the code design values computed based on SDPWS-2015.

According to the prior studies, sheathing the second side of the wood frame walls with GWB increases the wall strength and stiffness of the wall in comparison to the walls sheathed with OSB only on one side of the wall frame (e.g. Karacabeyli and Ceccotti 1996, Filiatrault *et al.* 2002, Sinha and Gupta 2009, Sinha and Gupta 2009, Zhou and He 2011). Therefore, it is expected that:

• The walls using structural foam sheathing without GWB have a lower unit shear load resistance than the walls tested with GWB in Section 1 of the current study.

Likewise, the hold downs used in the test improved unit shear capacity in prior testing at Fears Lab changed the failure modes when compared to typical residential buildings without hold downs (Pardoen *et al.* 2000, Gatto and Uang 2002). Therefore, it is expected that:

• The walls sheathed with structural foam sheathing without hold downs have a lower unit shear capacity than the walls tested with hold downs in this study.

In comparison of the foam sheathing and OSB walls, the OSB test walls showed:

• Using doubled baseplates with bearing plate washers and increased number of anchor bolts are the most effective method to increase capacity of shear walls sheathed with OSB for walls tested in this study.

• Adding LSTA 9 ties improved the unit shear capacity for the tested walls in this study.

• Unit shear capacity of the OSB test walls were higher than the computed code design values.

• Computed deflection based on SDPWS-2015 and test wall deflection at peak loads were less than ASCE 7 limits for the test walls in both sections (1 and 2) in this study.

Further research is recommended to investigate structural foam sheathed wood shear walls with and without hold downs and GWB under monotonic and cyclic loadings.

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