3rd Specialty Conference on Material Engineering & Applied Mechanics
3e Conférence spécialisée sur le génie des matériaux et mécanique appliquée

Montréal, Québec
May 29 to June 1, 2013 / *29 mai au 1 juin 2013*

Comparative Structural Behavior of Insulated Sandwich Foam-OSB Walls and Floor with Conventional Timber Stud Panels in Residential Buildings

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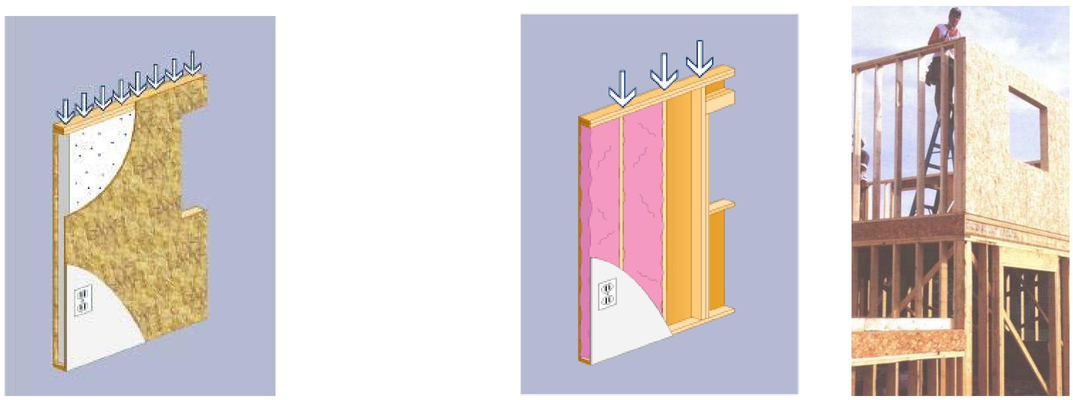
Abstract: This report presents the experimental testing on selected structural insulated foam-timber panels to investigate their structural behavior when used as walls in residential construction. The structural insulated panel considered in this study is a panel composed of expanded polystyrene insulation (EPS) board laminated between two oriented-strand boards (OSB). The results of axial load and flexural tests performed in this study established a database that can be used further to develop design tables of SIP wall subjected to either axial compressive loading or combined axial compressive loading and lateral bending moment from wind loading. A design table for walls subjected to axial compressive loading was developed too. Experimental findings proved that the tested SIPs are "as good as" the conventional wood-frame system.

1. Introduction

A Structural Insulated Panel (SIP) in this study is a panel composed of foam insulation core laminated between two oriented-strand boards (OSB). SIPs deliver building efficiencies by replacing several components of traditional residential and commercial construction, including: (i) studs; (ii) insulation; (iii) vapour barrier; and (iv) air barrier. A SIP-based structure offers superior insulation, exceptional strength, and fast installation. Figure 1 shows views of SIP panel and stud wall system under gravity loading. Besides those benefits, the total construction costs are less with SIPs compared to wood-framed homes, especially when considering speed of construction, less expensive HVAC equipment required, reduced site waste, reduction construction financing costs, more favourable energy-efficient mortgages available, and the lower cost of owning a home built with SIPs. In case of flexural loading, all of the elements of a SIP are stressed; the skins are in tension and compression, while the core resists shear and buckling. Under axial concentric in-plane loading, the facings of a SIP act as slender columns, and the core stabilizes the facings and resists forces that may cause local bucking of the facings. However, in the conventional stud wall system shown in Fig. 1, the studs transfer the load from the roof and floor down to the foundation, while the foam is installed between studs to provide insulation. SIPs panels can be used in industrial, commercial and residential construction as load bearing elements. The energy saving insulation, design capabilities, cost effectiveness, speed of construction and exceptional strength make SIPs the future material for high performance buildings. To determine the structural adequacy of the level of adhesion between the foam and the OSB boards and the level of composite action between them, it is felt necessary to conduct experimental testing to-collapse on the developed structural insulated sandwich timber panels. Clause 8.6 of the Canadian Standard for Engineering Design of Wood, CAN/CSA-O86-10, (2010) specifies the effective stiffness, bending resistance and shear resistance of stressed skin panels. These stressed skin panels have continuous or splice longitudinal web members and continuous or spliced panel flanges on one or both panel faces, with the flanges glued to the web members. These strength equations are not applicable to SIPs since they do not address the adequacy of the foam as the main shear carrying element near the supports and the connector between the facings at the maximum moment location. CAN/CSA-O86-10 compressive resistance equations for studs cannot be applied to SIPs as a result of their structural performance at failure. Since ultimate load carrying capacity of SIPs in compression or bending is as yet unavailable, evaluation of a given full-scale SIP panel a loading tester is required. The technical guide of Canadian Construction Materials Commission (CCMC) and National Research Council Canada (NRC) for stressed skin panels for walls and roof (Institute, 2007) formed the basis for the experimental testing conducted in this thesis for flexure, axial eccentric and axial concentric, with the ultimate goal of providing enough technical data for strength and serviceability of the developed structural insulated sandwich timber panels. With this database, design tables can be established. However, SIP panels can be structurally qualified if their performance is proven to be “as good as” conventional stud wall system currently used in housing construction. The main objective of this experimental study is to provide research information to examine whether SIPs are as good as stud wall system in resisting transverse and axial loading with eccentricity.



(a) SIP wall (b) Stud wall system

Figure 1. Comparison of SIP with stud wall system (Thermapan, 2007)

2. Experimental Program

SIPs considered in this experimental study (Thermapan, 2007) have a standard size of 1.2 m wide and are composed of thick layer of expanded polystyrene insulation (EPS) board laminated between two sheets of 11 mm (7/16”) thick. oriented strand board (OSB) as shown in Fig. 1. The tested specimens were divided into 4 groups based on the type of wall and loading conditions. Tables 1 and 2 summarize the tested specimens for axial compressive loading and flexural loading, respectively. The following sections describe the specimen geometry and material properties in addition to the test setup and test procedure.

2.1 Description of SIP and stud wall materials

Group C-SIP, listed in Table 1, consisted of 3 identical panels of 2.73 m (9’) length, 165 mm (6 ½”) total depth, and foam-spline connection. These panels are listed as W7, W8, and W9. It should be noted that the applied compressive load was eccentric to the mid-thickness of the panel. An eccentricity of t/6 was considered, where t is the thickness of the panel. Group G-SIP, listed in Table 2, consisted of 5 identical panels to those in group C-SIP except they were subjected to flexural loading. These panels are listed as WS19, WS20, WS21, WS22 and WS23. The OSB board fabricate panels had 1R24/EF16/W24 panel mark with 11 mm thickness construction sheathing. The expanded polystyrene (EPS) core material demonstrates the following characteristics: nominal density =16 kg/m^3, flexural strength = 172 kPa; ccompressive strength = 70 kPa; shear strength = 83 kPa; and shear modulus = 2758 kPa.All stud panels were manufactured for wall construction with 2.4 m wide and 11 mm thick OSB boards for the outside facing (i.e. loading face) and 12.7 mm drywall board for the inside facing. Group C-STUD, listed in Table 1, consisted of 3 identical panels of 2.73 m (9’) length. These panels are listed as SW-1, SW-2 and SW-3. 38x140 mm (2x6) timber studs were used between the OSB and drywall facings at spacing of 600 mm centre-to-centre. It should be noted that the applied compressive load was eccentric to the mid-thickness of the panel. An eccentricity of t/6 was considered, where t is the thickness of the panel. Group G-STUD, listed in Table 2, consisted of 3 identical panels to those in group C-STUD except they were subjected to flexural loading. These panels are listed as SF-1, SF-2 and SF-3. It should be noted that the OSB facings were nailed to the studs in three segments with a 2 mm gap at the facing horizontal joints. The drywall facing was nailed to the studs in two vertical segments. The studs were Spruce-Pine-Fir species combination with No. 2 grade (i.e. SPF No. 2). The following are the material properties of the tested studs: flexural strength = 11.8 MPa; shear strength = 1.5 MPa; compressive strength parallel to the grains = 11.5 MPa, and modulus of elasticity = 9500 MPa.

Table 1. Results from axial compressive load tests

Group	Test No.	Test type	Panel size: length× width× total thick.	Connection type	Experim. Ultimate jacking load (kN)	Resisting ultimate jacking load (kN)	Jacking load at 1/8" axial shortening limit (kN)
C-SIP	W7	Axial loading at t/6	4'×9'×6 ½"	Foam spline connection	199.58*	79.83(1)	91.72
	W8	Axial loading at t/6	4'×9'×6 ½"	Foam spline connection	172.36*	68.94(1)	59.91
	W9	Axial loading at t/6	4'×9'×6 ½"	Foam spline connection	268.23*	107.29(1)	92.94
C-STUD	SW-1	Axial loading at t/6	8'×9'×6 ½"	2x6 studs @ 2'-0" c/c	253.51**	101.40(1)	62.41
	SW-2	Axial loading at t/6	8'×9'×6 ½"	2x6 studs @ 2'-0" c/c	297.23**	118.89(1)	66.65
	SW-3	Axial loading at t/6	8'×9'×6 ½"	2x6 studs @ 2'-0" c/c	189.60**	75.84(1)	69.84

* Did not include 1.25 kN weight of the loading system.
** Did not include 3.34 kN weight of the loading system.
(1) Considering safety factor of 2.5

Table 2. Results from flexural load tests

Group	Test No.	Test type	Panel size: length× width× total thick.	Connection type	Experim. Ultimate jacking load (kN)	Resisting ultimate jacking load (kN)	Resisting Ultimate moment (kN.m)
G-SIP	WS19	Flexural loading	4'×9'×6 ½"	Foam spline connection	27.22*		
	WS20	Flexural loading	4'×9'×6 ½"	Foam spline connection	27.77*		
	WS21	Flexural loading	4'×9'×6 ½"	Foam spline connection	24.99*		
	WS22	Flexural loading	4'×9'×6 ½"	Foam spline connection	28.77*	9.03(1)	2.93
	WS23	Flexural loading	4'×9'×6 ½"	Foam spline connection	26.77*		
G-STUD	SF-1	Flexural loading	8'×9'×6 ½"	2x6 studs @ 2'-0" c/c	59.68**		
	SF-2	Flexural loading	8'×9'×6 ½"	2x6 studs @ 2'-0" c/c	67.68**	21.65(1)	7.01
	SF-3	Flexural loading	8'×9'×6 ½"	2x6 studs @ 2'-0" c/c	67.56**		

* Did not include 2 kN weight of the loading system.
** Did not include 3.82 kN weight of the loading system.
(1) Considering safety factor of 3

2.2 Test method for SIP Panels and Stud Walls under Axial Compressive Loading

For the purpose of structural qualifications of SIPs, the Canadian Construction Materials Commission (CCMC) produced a technical guide (IRC, 2007) in collaboration with the National Research Council Canada (NRC) to describe the technical requirements and performance criteria for the assessment of stressed skin panels (with lumber 1200 mm o.c. and EPS core) for walls and roofs. In this guide, The performance of the stressed skin panels for walls and roofs, have been evaluated, as an alternative solution, with respect to Part 4, Structural Design, and Part 9, Housing and Small Buildings, of the National Building Code of Canada (NBCC, 2010). A successful evaluation conforming to this Technical Guide will result in a published CCMC Evaluation Report. The published CCMC Evaluation Report is applicable only to products bearing the proper identification number of CCMC's evaluation number. This NRC/IRC/CCMC Technical Guide specifies test methods for SIPs which is similar to those specified in ASTM E72-10, *Standard Test Methods for Conducting Strength Tests of Panels for Building Construction*, (ASTM, 2002) as well as ICC AC04, *Acceptance Criteria for Sandwich Panels*, (2004). AC04 specifies that load bearing wall panels shall support an axial loading applied with an eccentricity on one-sixth the panel thickness to the interior or towards the weaker facing material of an interior panel. Also, AC04 specifies that the test setup shall be capable of accommodating rotation of the test panel at the top of the wall due to out-of-plane deflection with the load applied throughout the duration of the test with the required eccentricity. AC04 also specifies that the test panel shall have wall sill and cap plate details with connections matching the proposed field installations. Axial loads shall be applied uniformly or at the anticipated spacing of the floor or roof framing. To prepare for the test, the wall panel aligned vertically and supported directly over the laboratory's floor or over an elevated precast concrete slab units. A uniformly distributed line load was applied on the top side over the 1200 m width using a loading assembly. This loading assembly was composed of a 1200×350×12 mm steel base plate resting over the top side of the panel. A 125×125×12.7 mm HSS box beam of length 1200 mm was welded to the top side of the steel base plate to transfer the applied jacking load over the panel width. Two 70×70×9 mm steel angles of 1200 mm length were welded to the steel base plate, one on each side of the wall panel to stabilize the loading assembly during the test. The weight of the loading assembly was calculated as 1.25 kN. Figure 2 shows view of Wall W8 before testing. Four potentiometers (POTs) were installed vertically over the four corners on the top side of the panels to record axial shortening of the wall panel under load. The compressive load was applied through a jacking load system with a universal flat load cell of 222 kN capacity to measure the jacking load. During testing, the process for collecting and converting data captured by POTs and load cell were done using a test control software (TCS) with SYSTEM 5000 data acquisition unit which was adjusted to sample the data at rate of 10 reading per second during the test. ASTM E72-10 specifies that wall panels shall be loaded in increments to failure with deflections taken to obtain deflections and set characteristics. For each panel, the jacking load was continuously increasing at a slow rate. Visual inspection was continuously conducted during the test record any change in the structural integrity of the wall panel. Each test was terminated after the wall panel failure. Failure of the panel was considered when the recorded jacking load was not increasing or when the panel could not absorb more loads while recorded axial shortening was increasing by continuously pressing the pump handle. Mode of failure was recorded and test data was then used to draw the load-deflection and load-axial shortening relationships for each panel.

Identical test setup to that used for SIPs was utilized in stud wall testing except that the stud wall specimens were of 2400 mm (8') width. To prepare the test setup for compression testing, the stud walls were assembled and was then aligned vertically and supported directly over the laboratory's floor. A uniformly distributed line load was applied on the top side over the 2400 m width using a loading assembly. This loading assembly was composed of a 2400×350×12.7 mm steel base plate resting over the top side of the panel. A 150×150×12.7 mm HSS box beam of length 2400 mm was welded to the top side of the steel base plate to transfer the applied jacking load over the panel width. Two 125×125×12.7 mm steel angles of 2400 mm length were welded to the steel base plate, one on each side of the wall panel to stabilize the loading assembly during the test. The weight of the loading assembly was calculated as 3.34 kN. Figures 4(a) and 4(b) show views of Wall SW-1 from the drywall side and the OSB side, respectively, before testing. Four potentiometers (POTs) were installed vertically over the four corners on the top side of the specimen to record axial shortening of the wall panel under load.

2.3 Test method for SIP Panels and Stud Wall System under Flexural Load

Bending qualification tests on the panels were conducted as specified in ASTM E72-10 that specifies at least three identical specimens for each test group. Each tested panel was supported over two 25.4 mm steel rollers at each side in the short direction. Other similar-size steel plates were inserted between the supporting roller and the panel bottom facing. A 150×150×12.7 mm HSS beam of 2400 mm length used to transfer the applied jacking load to a 102×1020×6.4 mm HSS beam that was laid transversally over the top panel facing at the quarter points to spread the load over the panel width. The weight of this loading system is 2.0 kN. Figure 8 shows view of the test setup of WS19 wall panel. Mid-span deflection was measured using 4 Linear Variable Displacement Transducers (LVDTs). These LVDTs were positioned underneath the panel, with two LVDTs were located at 25 mm from the panel free edges and other two LVDTs located at the third points of the panel width. The load was applied through a jacking load system with a load cell of 222 kN capacity. As for flexural tests on stud wall system, similar test setup and test procedure were similar to those used for SIP testing was utilized except that the stud wall specimens were of 2400 mm (8') width. Figure 10 shows view of the flexural test setup for specimen SF-1. Each tested panel was supported over two 25.4 mm steel rollers at each side in the short direction. A 150×150×12.7 mm HSS beam was used to transfer the applied jacking load to two W150 steel beams that then transfer the load to a 125×125×12.7 mm HSS beam that was laid transversally over the top panel facing at the quarter points to spread the load over the specimen width. Steel roller and plate assembly similar to that used to support the panel over the steel pedestals was used to support the W150 steel beams over the two 2400 mm length HSS spread beams at the quarter points. The weight of this loading system is 3.82 kN. Mid-span deflection was measured using 3 Linear Variable Displacement Transducers (LVDTs). These LVDTs were positioned underneath the specimen, with one LVDT located at the centre of the specimen and two LVDTs located at the quarter points of the specimen width.

Figure 2. View of wall W8 before testing

Figure 3. Views of deformed shape of wall W8

(a) Dry wall side (b) OSB sheet side

Figure 4. Views of the test setup for stud wall SW-1 before testing

3. Experimental Results

This Section discusses the experimental results of testing to-collapse 14 actual-size timber panels according to ASTM standards to qualify them based on code requirements and test method criteria. The experimental results for all panels, in the form of load-axial shortening relationship, load-lateral deflection relationship, and failure pattern, are presented in sequence for each specimen group.. The structural adequacy of the tested sandwich panels for possible use in residential construction is presented.

Figure 5. SW-1 Stud wall views after failure showing OSB sheet joint widening near the top of wall

3.1 Code Requirements for the Structural Qualification of SIPs

The Structural qualifications of the SIPs have been assessed based on (i) the general design principles provided in CSA Standard CAN/CSA-O86.10, (ii) the evaluation criteria set forth in the NRC/CCMC Technical Guide which focuses on SIPs as being "as good as" the conventional wood-frame building with respect to strength and serviceability and (iii) the National Building Code of Canada (NBCC, 2010). In this research, the following loads and load factors are used to examine the structural adequacy of the panels for serviceability and ultimate limit states design: dead load factor = 1.25; live load factor = 1.50; dead load for roofs = 0.5 kPa; dead load for floors = 0.47 kPa; live load for residential construction = 1.9 kPa; snow load for residential construction = 1.9 kPa (for simplification of comparison); deflection limit for serviceability (live load effect) = span / 360; and deflection limit for serviceability (total load effect, dead and live) = span/180. The average deflection and ultimate load carrying capacity of each specimen group are basically the average of those for the three panels in each panel group (ICC-ES AC04, 2004). Further, when the results of one of the tested panel vary more than 15% from the average of the three panels, one of the following two actions was chosen: (i) the lowest test value may be used; or (ii) the average result based on a minimum of five tests may be used regardless of the variations. Factor of safety for ultimate load carrying capacity of SIPs is dependant on the followings: (i) consistency of materials, (ii) the range of test results, and (iii) the load-deformation characteristics of the panel. AC04 generally applies a factor of safety of 3 to the ultimate load based on the average of three tests which called in this research as panel group. However, AC04 provides the following factors of safety applicable to uniform transverse loads: F.S. = 3.0 for ultimate load at shear failure for all loading conditions; F.S. = 2.5 for ultimate reaction at failure for all loading conditions; F.S. = 2.5 for ultimate load determined by bending (facing buckling) failure under allowable snow loads; F.S. = 2 for ultimate load determined by bending (facing buckling) failure under allowable live loads up to 0.958 kPa. In case of wall panel axial load tests, AC04 specifies that wall panels shall support an axial loading applied with an eccentricity of 1/6 the panel thickness. Also, AC04 specifies that the factored design resisting axial load is determined from the experimental axial load at a net axial deformation of 3.18 mm (1/8") or the ultimate load divided by a factor of safety determined in accordance with those specified for transverse load testing mentioned above, whichever is lower.

3.2 Results of Panel Group C-SIP for Axial Loading

In this group, three identical panels were tested to complete collapse under uniformly distributed axial compression load with t/6 eccentricity. As an example of the test results, Fig. 2 shows view of panel W8 before testing, while Fig. 3 shows views of the permanent deformed shape of the panel after failure. Figure 6 depicts the load-axial shortening relationship for model W8. It was observed that the failure mode was OSB crushing near the top part of the wall on one facing and tensile fracture on the other facing at the same location. It was observed that the failure mode of OSB crushing occurred on one side of the panel near its top quarter point, associated with diagonal tensile fracture of the foam at the same location as well as at the connection with the top wall plate. The failure was abrupt causing a sudden drop in the applied jacking load. It was observed that linear elastic behaviour was maintained till failure as depicted in Fig. 3. Table 1 shows that the experimental ultimate jacking loads were 199.58, 172.36 and 268.23 kN for walls W7, W8 and W9, respectively. As per AC-04, the ultimate factored design axial resisting compressive load is the experimental ultimate load divided by 2.5. Thus, the factored design axial resisting load is 79.83, 68.94 and 107.29 kN for walls W7, W8 and W9, respectively. Since the obtained design values of two of the walls are more than 15% difference with the average value of the three panels, the design factored axial compressive load for group C-SIP is taken 68.94 kN, as the least value of the three values. This value is recorded in summary Table 3. By including the 1.25 kN self-weight of the loading beam, the design factored compressive resistance of the C-SIP becomes 70.19 kN as recorded in summary Table 4. Table 1 shows that the jacking loads at 1/8" axial shortening were 91.72, 59.91 and 92.94 kN for walls W7, W8 and W9, respectively. Since the obtained design values of two of the walls are more than 15% difference with the average value of the three panels, the design factored axial compressive resistance per the 3 mm (1/8") axial shortening criteria for group C-SIP is 59.91 kN, as the least value of the three. This value is reported in summary Tables 3 and 4.

3.3 Results of Stud Group C-STUD for Axial Loading

In this group, three identical stud wall specimens were tested to complete collapse under uniformly distributed axial compression load with t/6 eccentricity. Each specimen was of 2.4 m wide in contract to 1.2

m width of SIP specimens. Fig. 4 shows view of stud specimen SW-1 before testing. It was observed that the failure mode in stud wall SW-1 was due to global bending of the studs. This is shown through the bending deformation of the wall from the drywall side as well as the widening of the gap between OSB segments near the top of the wall as shown in Fig. 5. Figure 7 depicts the load-axial shortening relationship for stud wall specimen SW-1. It can be observed that the load-shortening curve was not linear at the beginning of the test since the specimen were picking up the load while the top loading beam was in a transition of full contact with the specimen. Then, the specimen showed linear elastic response. However, the load-shortening showed nonlinear deformation in the panel while approaching failure. Table 1 shows that the experimental ultimate jacking load was 253.51, 297.23 and 189.60 kN for walls SW-1, SW-2 and SW-3, respectively. Considering the ultimate factored design axial resisting compressive load is the experimental ultimate load divided by 2.5. Thus, the factored design axial resisting loads were 101.40, 118.89 and 75.84 kN for walls SW-1, SW-2 and SW-3, respectively. The design factored axial compressive load for group C-STUD is 75.84 kN, as the least value of the three. In this case, the design axial compressive load of group C-STUD with 1.2 m width is to be taken as 37.92x0.8 = 30.34 kN as reported in Table 3. By adding the 3.34 kN self-weight of the loading beam, the design axial compressive resistance for C-STUD group is adjusted to be 31.67 kN as reported in summary Table 3. Table 4 shows that the jacking loads at 1/8" axial shortening C-STUD group were 62.41, 66.65, and 69.84 kN for walls SW-1, SW-2 and SW-3, respectively. The design factored axial compressive resistance per the 3 mm (1/8") axial shortening criteria for group C-STUD is taken as 66.30 kN, as the average of the three values. By dividing this value over 2 to obtain the resistance for 1.2 m wide stud wall and by 0.8 due to using 5 studs in the wall rather than 4 over 2.4 m width, the design factored axial compressive resistance is taken as 53.04 kN as reported in summary Tables 3 and 4 for group C-STUD.

3.4 Results of Panel Group G-SIP for Flexural Loading

In this group, five identical panels were tested to-complete-collapse under flexure load. Figure 8 shows views of panel WS19 before testing, while Fig. 9 shows view of the permanent deformed shape of the panel depicting the horizontal shear failure at the interface between the foam and top OSB facing. The failure was between the top surface of the foam and the adhesive over a panel length between the support and the quarter-point line, causing top foam-OSB delamination (de-bonding) over the supports. Similar behavior to WS19 was observed for walls WS20 through WS23 of the same size. Table 2 shows a summary of panel configurations along with the ultimate jacking load for Group G-SIP. It can be observed that the ultimate jacking load was 27.22, 27.77, 24.99, 28.77 and 26.77 kN for panels WS19, WS20, WS21, WS22 and WS23, respectively. Conservatively, the jacking load did not include the weight of the loading system of 2 kN. It is worth mentioning that the ultimate jacking load for each panel is within 15% difference from the average jacking load of the three panels. Thus, the design ultimate jacking load is taken as the average experimental ultimate jacking load divided by a factor of safety of 3 (i.e. 27.10 / 3 = 9.03 kN). This makes the design bending resistance as 2.93 kN.m for 1.2 m panel width as reported in Table 3.

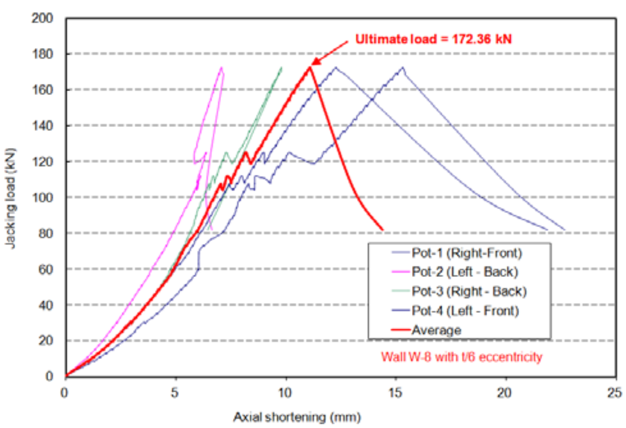


Figure 6. Axial load-axial shortening relationships for model W8

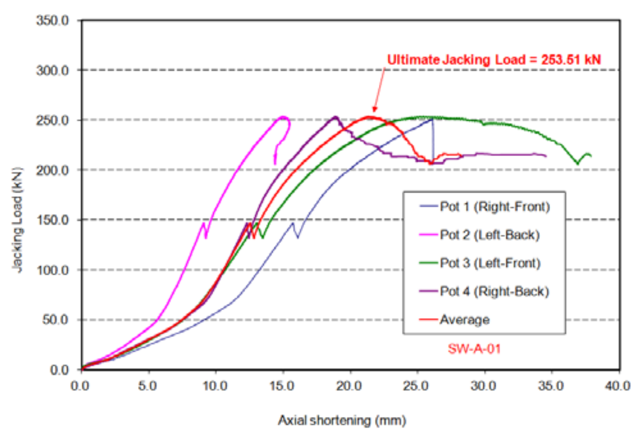


Figure 7.Axial load-axial shortening relationships for stud wall SW-1

Table 3. Capacities of stud walls and SIPs of 1.2 m width excluding weight of the loading system

Wall type	Panel size: length× width× total thick.	Ultimate Resisting jacking load, P_r (kN)	Jacking load at 1/8” axial shortening limit (kN)	Ultimate * Resisting Moment, M_r (kN.m)
SIP	4’×9’×6 ½”	68.94	59.91	2.93
Stud	4’×9’×6 ½”	30.34	53.04	2.81

Table 4. Capacities of stud walls and SIPs of 1.2 m width including weight of the loading system

Wall type	Panel size: length× width× total thick.	Ultimate* Resisting compressive load, P_r (kN)	Jacking load at 1/8” axial shortening limit (kN)	Ultimate Resisting Moment, M_r (kN.m)
SIP	4’×9’×6 ½”	70.19	59.91	3.14
Stud	4’×9’×6 ½”	31.67	53.04	2.97

3.5 Results of Stud Group C-STUD for Flexural Loading

In this group, three identical specimens were tested to-complete-collapse under flexure loading. Figure 10 shows views of panel SF-1 before testing, while Fig. 11 shows view of the permanent deformed shape of the stud specimen after failure. The failure mode was due to pure flexural at the mid-span location. The flexural failure occurred in the first stud in the west side of the specimen as shown in Fig. 11. Tensile fracture in both the stud and the bottom drywall was suddenly occurred at failure. A horizontal splitting in the second stud was observed in the west side of the specimen between the mid-span and quarter point. Table 2 shows a summary of stud specimen configurations along with the ultimate jacking load for Group G-STUD. It can be observed that the ultimate jacking load was 59.68, 67.68 and 67.56 kN for specimens SF-1, SF-2 and SF-3, respectively. The design ultimate jacking load is taken as the average experimental ultimate jacking load divided by a factor of safety of 3 (i.e. 64.95 / 3 = 21.65 kN). This makes the design ultimate bending moment resistance as 7.01 kN.m for 2.4 m panel width. The stud specimens considered in this study have 5 studs per specimen cross-section. So, the design ultimate jacking bending moment resistance of group G-STUD with 1.2 m width is to be taken as 2.93x0.8 = 2.81 kN.m as reported in Table 3.

Figure 8. View of the test setup for model WS19 before testing

Figure 9. View of the horizontal shear failure at the interface between the foam and top OSB in model WS19

Figure 10. View of specimen SF-1 before testing

Figure 11.View of specimen SF-1 after failure

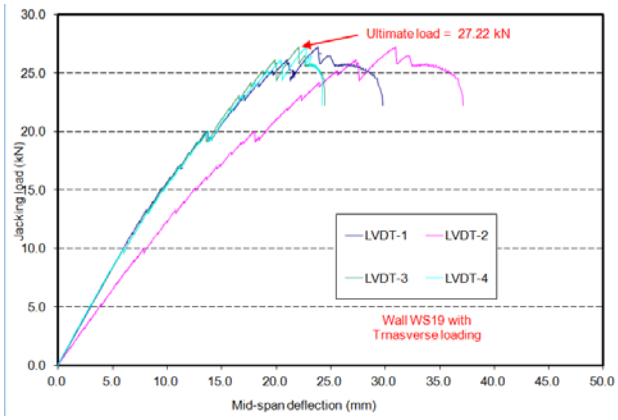


Figure 12. Jacking load-deflection relationship for model WS19

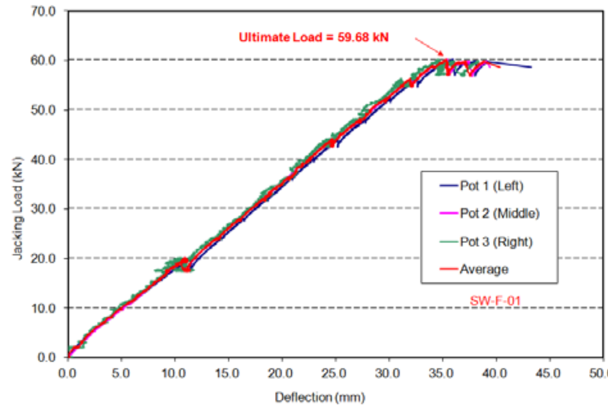


Figure 13. Axial load-axial shortening relationships for specimen SF-1

3.6 Correlation of test results for SIPs and stud wall specimens for gravity loading and bending

Based on the aforementioned sections, Table 3 and 4 was prepared to examine whether the structural qualification of SIPs is being "as good as" the structural capacity of the conventional wood-frame buildings. As it can be observed from Table 4 that the experimental ultimate resisting compressive load for SIPs (70.19 kN) is greater than that for stud wall specimens (31.67 kN) by 121.6%. Also, Table 4 shows that the experimental ultimate resisting compressive load for SIPs at 3-mm deformation (59.91 kN) is greater than that for stud wall specimens (53.04 kN) by 13%. In addition, the resisting moment of the tested SIP panel (3.14 kN.m) is greater than that for stud wall (2.97 kN.m) by 5.7%. For the acceptance criteria of SIPs, Clause 5.2.2 of the CCMC technical guide specifies that SIP panel load at L/360 deflection shall exceed the loads on the conventional panel at L/360 deflection. With respect to deflection acceptance criteria, Table 5 summarizes the flexural load at the deflection limits of L/360 and L/180 for the SIP and stud wall groups.

Table 5. Flexural load at specified deflection limit

Group	Test No.	Panel size: length× width× total thick.	Flexural load at L/360 (kN)	Adjusted fle Load at L/360 (kN)	Average Value for L/360	Flexural load at L/180 (kN)	Adjusted flexural Load at L/180 (kN)	Average Value for L/180
G-SIP	WS19	4'×9'×6 ½"	11.07	11.07		19.14	19.14	
	WS20	4'×9'×6 ½"	10.63	10.63		19.04	19.04	
	WS21	4'×9'×6 ½"	11.00	11.00	10.83	18.77	18.77	18.95
	WS22	4'×9'×6 ½"	10.70	10.70		18.60	18.60	
	WS23	4'×9'×6 ½"	10.75	10.75		19.19	19.19	
G-STUD	SF-1	8'×9'×6 ½"	13.11	5.24*		24.18	9.67*	
	SF-2	8'×9'×6 ½"	12.29	4.92*	5.23	23.75	9.50*	9.92
	SF-3	8'×9'×6 ½"	13.81	5.52*		26.50	10.60*	

* Values divided by 2 for 1.2 m panel width and multiplied by 0.8 for using 5 studs rather than 4 in the tested wall

3.7 Design Table for SIP wall under axial compressive load

Manual calculations were performed to determine the maximum joist span by SIP wall to meet the design requirements for combined dead and live loading. In case of a wall carrying a roof and a floor, two load combinations were considered as follows: case (1): 1.25D +0.5S for the roof and 1.25D + 1.5L for the floor, where L is the floor live load; and case (2): 1.25D +1.5S for the roof and 1.25D + 0.5L for the floor, where L is the floor live load. Considering the floor live load in residential construction as 1.9 kPa, the served span for the first and second load combination cases are 10.3, 9.72, 9.3, 8.8 and 8.43 for case (1) and 12.75, 10.6, 9.1, 7.9 and 7.0 for case (2). Final results are reported in Table 6. CAN/CSA-O86-10 specifies that members subjected to combined bending and compressive axial loads shall be designed to satisfy a specified compression-bending interaction equation.

Table 6. Design table for SIP wall under axial compressive loading

Test type	Panel size: length× width× total thick.	Resisting ultimate jacking load, kN	Resisting ultimate uniform load capacity = design load /1.2, kN/m	Building storeys	Maximum supported joist length [1], [2], based on ultimate strength, m				
					Specified snow load, kPa				
					1.00	1.50	2.00	2.50	3.00
Axial loading (at t/6)	4’×9’×6 ½”	59.91	49.93	Roof only	22.9	16.9	13.4	11.1	9.5
				Roof and floor	10.3	9.7	9.1	7.9	7.0
				Roof and 2 floors	5.7	5.5	5.3	5.2	5.0

(1) Supported joist length = half the sum of joist spans on both sides of internal wall or half joist span of exterior wall.
(2) Maximum supported length of roof is based on 0.5 kPa dead load, 1.9 kPa live load for floors and a specified snow load as shown on flat roofs. Wall (with siding, stucco) weight of 0.4 kPa is considered as dead load

CONCLUSIONS

Based on the experimental findings, the following conclusions can be drawn:

1. The dominant failure mode in SIP panels under eccentric gravity loading is due to crushing of the panel facing at two main locations that let to lateral permanent deformation of the wall panel after failure. These locations are (i) the connection between the OSB facings with the top or bottom stud plates; (ii) the quarter point area of the wall height. In some failure cases, shear de-bonding between the foam and OSB facing was observed. Moreover, some panels exhibited diagonal crack in the foam associated with OSB crushing.
2. The dominant failure mode in SIP under flexural load is due to the horizontal shear failure at foam-OSB sheet interface between the support and the quarter point. However, it was flexural in case of stud wall under transverse loading.
3. Based on the data generated from testing, a design table was developed to provide designers with maximum served joist span when this wall size is used in residential building up to two storey high. Similar design tables can be established for walls subjected to combined compressive and wind.
4. Comparison between the experimental findings for SIPs and stud wall specimens of similar geometry showed that SIPs is being “as good as” the structural capacity of the conventional stud wood-framing with respect to (i) axial capacity and deformation, and (ii) flexural capacity and deformation.

Acknowledgment
The authors would like to ackowledge the finacial support and inkind contribution fro Thermapan Inc. of Fort Erie, Ontario. This support is greatly appreciated.

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