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SEISMIC EVALUATION OF STRUCTURAL INSULATED PANELS

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Abstract

New construction represents a significant portion of the total energy consumed by the building sector. One approach for reducing the energy impact of new construction is to utilize composite materials. Structural insulated panels (SIPs) are stressed-skin panels composed of an energy-efficient core such as expanded polystyrene (EPS) paired with either traditional or novel facing materials including plywood, oriented strandboard (OSB), cement mortar, or steel. Currently SIPs are underutilized in residential construction, which instead predominantly employs timber-framing methods including sheathed timber shear walls. While the seismic performance of the latter has been extensively studied and incorporated into building codes, there is considerably less information available about the general behavior of SIPs, especially regarding their behavior when subjected to seismic loads. This paper focuses on the characterization of the mechanical properties and seismic performance of SIPs using experimental techniques. Specimens studied include both OSB-faced and cementitious SIPs, where panels were tested without panel-to-panel connections.

Keywords: Diagonal tension test, Energy efficiency, Pseudo-dynamic test, Quasi-static test, Seismic resistance, Structural insulated panels.

Introduction

With the rise of industrial nations and developing countries alike comes a demand for housing and infrastructure, which rely on the availability of natural resources. As populations swell, the limitations of current development practices come sharply into focus: approximately 20% of the world's population residing in Western Europe, North

America and Japan, are consuming almost 80% of the total available energy and resources in order to maintain a high standard of living (Mehta, 1999). New construction represents a significant portion of this total energy consumed, with construction in developed countries dominated primarily by the use of steel, concrete, and timber. Noting that the concrete and Portland cement industries are two of the largest consumers of natural resources, such as water, sand, gravel, and crushed rock, it is imperative to find alternatives to energy-intensive materials and construction.

One approach for reducing the energy impact of new construction is to design structures with composite materials, allowing each component of the composite to be used in smaller amounts in a more efficient manner. Composite construction utilizing steel and concrete is widely practiced in the design of large structures, although a variety of structural systems are available for smaller applications such as residential and light-commercial edifices. Structural insulated panels (SIPs) are one example of a small-scale structural system, and are a type of stressed-skin panel, composed of an energy-efficient core such as expanded polystyrene (EPS) with both traditional and novel facing materials such as plywood, oriented strandboard (OSB), cement mortar, or steel. Currently, residential construction predominantly utilizes timber-framing, including sheathed timber shear walls, which have been extensively studied and incorporated into building codes and other design documents. Much of this existing work considers the structural behavior in light of monotonic and cyclic testing (He et al., 1999). Dynamic-related work (Durham et al., 2001) also exists, but is smaller in number.

There is considerably less information available about the behavior of SIPs, especially as pertains to their behavior in seismic regions. Developed over 50 years ago, SIPs are extensively used throughout Europe and North America. Their application in seismically hazardous regions is limited due to unacceptable performance as demonstrated by cyclic testing. While there is a growing database of research pertaining to SIPs, none has attempted to subject the panels to more realistic dynamic loading regimes. As such, this research's long term goals focus on the characterization of the dynamics response of SIPs. Lengthy strides can be made in seismic design approaches and codes in order to permit widespread use of this potentially sustainable material.

Background

In low-rise wood structures such as residences, the primary lateral load-carrying system is a timber shear wall, consisting of framing members sheathed most commonly with plywood or OSB, although non-timber alternatives are also available. According to (van de Lindt, 2004), wood shear walls are capable of dissipating large amounts of energy through their nonlinear behavior at low forces, attributed mostly to the fastener deformations. Another attribute that makes timber shear walls appealing is their low seismic mass, due to a high strength and stiffness to weight ratio. A considerable disadvantage of traditional timber shear walls is the tendency for an energy inefficient structure. Typically insulated with fiber fill, the framing elements provide a break in the thermal boundary. A study in (Choi, 2007) investigated an alternative method of insulating timber shear walls. By incorporating a layer of fly ash, with and without fiber-reinforcement, prior to insulating and dry-walling, it was found that the thermal efficiency could be improved.

SIPs are the basis for an energy efficient means of constructing residential and light commercial buildings. Mechanically, SIPs carry loads in a fashion similar to I-beam shapes: the wood sheathing is analogous to the flanges of the beam, carrying a majority of the moment, while the foam core behaves like the shear-carrying web, refer to Figure 1a. An efficient section is obtained when the weight of the core is roughly equal to the combined weight of the sheathing faces (Allen, 1969). Functionally speaking, the core must be stiff enough perpendicular to the faces to ensure that the faces remain separated by a constant distance. Additionally, the foam must be stiff enough in shear to prevent sliding of the faces, which would result in behavior similar to two independent beams rather than a composite unit. The interface between the foam core and the wood sheathing consists of a layer of adhesive to mainly prevent relative movements of the faces and the core.

Typically, three types of splines are utilized for SIPs panel-to-panel connections: solid pieces of lumber; surface splines consisting of 4-inch strips of OSB (Figure 1b); and foam block splines. Because foam block and surface splines do not result in a thermal break, they are preferred over the solid lumber connections. In locations subjected to point loads, a solid lumber spline is used to create a doubled stud, strong enough to support the concentrated load.

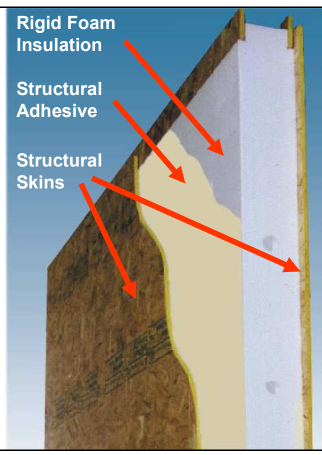
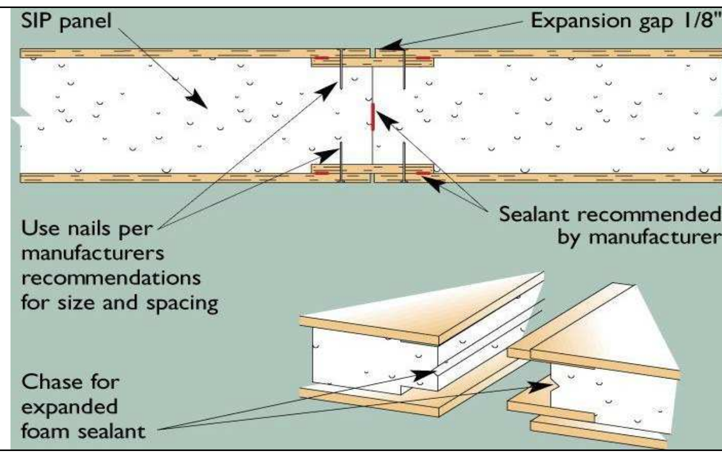
	
a) Components http://blackbros.com	b) Surface spline connection http://www.sips.org

Figure (1): SIPs components and example panel-to-panel connection [1"=25.4 mm]

Jamison (1997) provided one of the earliest studies of the racking performance of SIPs, specifically comparing this performance to existing knowledge on the behavior of timber shear walls. Monotonic and cyclic tests were performed, resulting in a primary mode of failure for the SIPs shear walls at the bottom plate, except in the presence of tie-down anchors, which shifted the mechanism of failure away from the bottom of the wall. The cyclic testing procedure followed the sequential phased displacement (SPD) procedure, which consists of an initial cycle followed by stabilized cycles for each amplitude phase. Use of stabilized cycles attempted to induce behavior representative of that characterized by repetitive cyclic loading, e.g. due to earthquake loads, resulting in fatigue of drywall screws from numerous loading cycles.

Experimental testing of structural systems is vital not only for research and development of new products, but also for gathering information about the dynamic performance of existing technologies. Currently, when knowledge about the seismic capacity of a structure or structural component is desired, there are three approaches to obtain this information experimentally: quasi-static tests, shaking table tests, and pseudo-dynamic tests. Quasi-static tests are conducted at a loading rate sufficiently slow such that strain-rate effects are negligible. Therefore, both monotonic and cyclic tests can be categorized as quasi-static, and many material testing standards, including those of ASTM, are quasi-static. While these tests are the simplest tests to perform and remove some of the variability associated with experimental procedures, they are also the most limited in the usefulness of the information that they can provide about the true dynamic behavior of test specimens.

Pseudo-dynamic tests, first developed in the 1970s, have gained momentum over the last 10 years in the structural engineering community because of their wide applicability and continuing refinements on the part of the academic community. The power of pseudo-dynamic (or hybrid simulation) lies in its ability to physically model structural components that are not well-understood while modeling well-understood components numerically. The analytical substructure usually consists of a numerical model of the governing equations of motion under dynamic excitation, which are solved during the test to determine displacements or forces to be applied in the physical substructure in displacement or force control (Elkhoraibi and Mosalam, 2007). However, for large structures, e.g. multi-story building, where nonlinearity is only expected in few structural elements of the lower story while the rest of the elements and the upper stories remain elastic, this computational model can be extended. For example, the experimental model here focuses on few elements of the lower story, while the numerical model would simulate the rest of the building. This is where the power of hybrid simulation resides as it can isolate substructures and study the behavior without the need for physical models of the entire structure, as in the more costly and size limiting shaking table tests (Elkhoraibi and Mosalam, 2007).

Introduction of pseudo-dynamic methodology to the study of SIPs could allow for improved understanding of the behavior of these elements during seismic loading. Firstly, the testing of SIPs has only been performed in the context of individual panels. While panel interaction has been studied to a limited degree in terms of panel-to-panel and panel-to-diaphragm interaction, there is no existing comprehensive study of a full-scale structure composed entirely of SIPs. Hybrid simulation, through mathematical description of adjacent elements, could allow for the study of SIPs in larger, more complex assemblages. Secondly, cyclic protocols such as those developed in (Krawinkler et al., 2000) were originally defined for wood frame structures, and its extension to panel structures may not be appropriate or immediately justifiable. Lastly, it is important to define seismic performance in terms of response to realistic seismic loading histories.

Material Tests

Cementitious SIPs (CSIPs) composed of fiber-cement mortar facings with EPS core together with traditional SIPs were the subject of diagonal tension tests (ASTM, 1988) used for the evaluation of shear behavior, Figure 2. The CSIPs were provided by Mississippi Structural Insulated Panel Systems. Each panel consisted of two 7/16" (11 mm) fiber-cement mortar facings with EPS core 3.7" (94 mm) thick, for an overall thickness of 4.575" (116 mm). The core material has a nominal density of 1.0 pcf (16 kg/m^3), and the panels are rated with an R-value of 25 for walls. Aside from the brittle behavior of the fiber-cement mortar facings, the polyurethane adhesive between the core and facings also behaves in a brittle fashion. The panels were 2' wide by 2' tall and were loaded monotonically with an average target rate of 2 kips/sec according to ASTM E519 (1988).

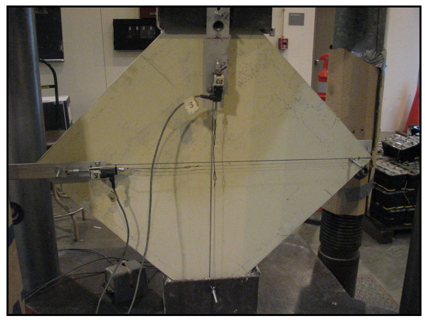
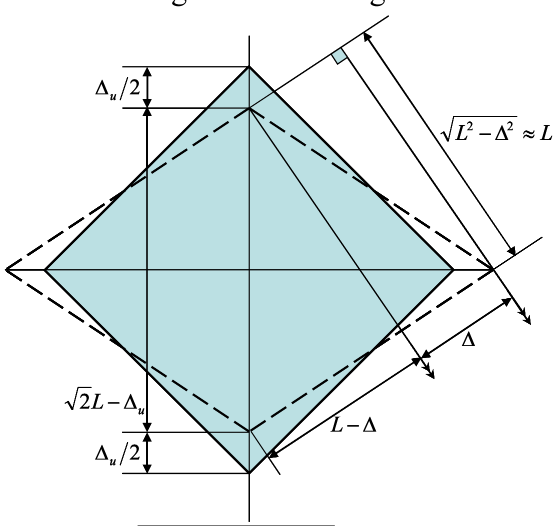


Figure (2): Material test setup and instrumentation of CSIP

The shear force pre unit length of the wall thickness t is defined as:

$$V/t = P/\left(t\sqrt{2}\right) \tag{1}$$

where P is the applied load. The wall's drift ratio, δ, is calculated as illustrated in Figure 3. In the following, the "exact" expression in Figure 3 is used where L is the wall side length and Δ_u is the wall diagonal shortening in the vertical direction.



Exact: $L-\Delta=\sqrt{\left(\sqrt{2}L-\Delta_u\right)^2-\left(L^2-\Delta^2\right)} \Rightarrow (L-\Delta)^2=\left(\sqrt{2}L-\Delta_u\right)^2-(L-\Delta)(L+\Delta) \Rightarrow$

$$2L(L-\Delta)=\left(\sqrt{2}L-\Delta_u\right)^2 \Rightarrow \Delta=L-\frac{\left(\sqrt{2}L-\Delta_u\right)^2}{2L} \Rightarrow \delta=1-\frac{\left(\sqrt{2}L-\Delta_u\right)^2}{2L^2}=1-\left(1-\frac{\Delta_u}{\sqrt{2}L}\right)^2 \tag{2}$$

Approximate: $L-\Delta=\sqrt{\left(\sqrt{2}L-\Delta_u\right)^2-L^2} \Rightarrow \Delta=L-\sqrt{\left(\sqrt{2}L-\Delta_u\right)^2-L^2} \Rightarrow$

$$\delta=\frac{L-\sqrt{\left(\sqrt{2}L-\Delta_u\right)^2-L^2}}{L}=1-\sqrt{\left(\sqrt{2}-\frac{\Delta_u}{L}\right)^2-1} \tag{3}$$

Figure (3): Calculation of the panel drift ratio in diagonal tension tests

Focusing on the peak shear force per unit thickness and the corresponding drift ratio, a complete summary of results is given in Table 1. In this table, the quantities with subscript r refer to the residual (after large peak force drop) values. Consistency in the definition of these residual quantities was maintained in every group of tests by adopting three definitions: δ_r=0.49% for the two CSIPs (CSIP1 and CSIP2) tested dry, δ_r=3.00% for the three SIPs tested with OSB facing (dry, moist, and wet), and δ_r=0.63% for the three other CSIPs (dry, moist, and wet). It is noted that CSIPs experienced sudden drop in the capacity, quantified by the drop ratio in the last column of Table 1, while the capacity of SIPs with OSB facing drops more gradually. Also, the water exposure of SIPs leads to reductions in the strength and the drop ratio (i.e. more ductile behavior). On the other hand, water exposure of CSIPs leads to unclear trend due to inherent variability in the tested panels and further tests with more samples is recommended to understand this phenomenon in CSIPs and to account for the strength reduction due to water absorption as recognized in ASTM.

Table (1): Peak results from material tests of CSIPs and SIPs

Specimen	V/t [kip/in]	δ [%]	$(V/t)_r$ [kip/in]	δ_r [%]	$\left[1-\frac{(V/t)_r}{V/t}\right]\times 100$ [%]
CSIP1	3.87	0.37	1.01	0.49	73.9
CSIP2	3.07	0.37	1.57	0.49	48.9
SIP OSB-Dry	1.29	0.88	0.40	3.00	69.0
SIP OSB-Moist	0.86	0.74	0.37	3.00	57.0
SIP OSB-Wet	0.86	0.74	0.52	3.00	39.5
CSIP-Dry	3.29	0.63	1.45	0.63	55.9
CSIP-Moist	4.06	0.40	1.08	0.63	73.4
CSIP-Wet	2.37	0.37	0.74	0.63	68.8

Racking Test Procedures

Five SIPs consisting of OSB sheathing and EPS insulating foam were subjected to cyclic and pseudo-dynamic testing regimes to assess their behavior under seismic loads. Panels were tested individually, without panel-to-panel connections, Figure 4. The SIPs were fabricated by Premier Building Systems, Dixon, CA. With an overall panel thickness of 4-3/8" (111 mm), the panels consisted of two 7/16" (11 mm) OSB skins with a 3-1/2" (89 mm) thick EPS foam core, Figure 5. While panels may be produced in widths ranging from 4' to 24', the provided SIPs were 4' wide by 8' high. These OSB skins are classified as Structural 1, Exposure 1 wood sheathing complying with DOC-PS-2 (ICC ES, 2004).

The EPS core has a nominal density of 1.0 pcf (16 kg/m^3) and is viewed as a closed-cell material but it absorbs water to 2-5% of its weight – fiberglass is an example of an open-cell material. The EPS is a non-toxic hydrocarbon; its combustion produces water vapor, carbon dioxide, and trace levels of ash. The foam core is joined to the OSB skins with a structural grade Type II, Class 2 laminating adhesive meeting APA AFG-01 specifications (ICC ES, 2004), i.e. this adhesive is suitable for use in panels supporting loads other than panel self weight and has a high resistance to both moisture and creep.

Framing timber, such as the end plates, consisted of nominal 2×4 (1-1/2"×3-1/2" (38 mm×89 mm) actual dimensions) untreated Douglas-fir structural grade lumber. These elements were installed at the top and bottom of each panel in pre-cut channels. Fasteners were common bright nails –nails with a normal surface finish– in two sizes, 8d and 16d, along with 1-5/8" coarse thread drywall screws.

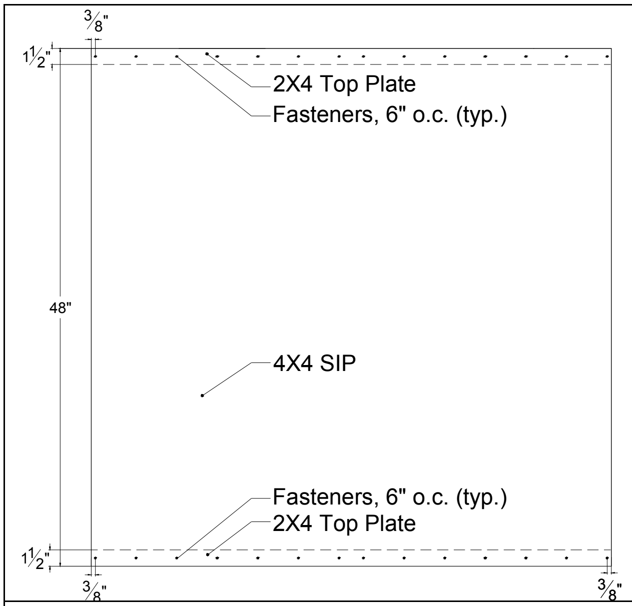


Figure (4): Panel Configuration for racking tests [1"=25.4 mm]

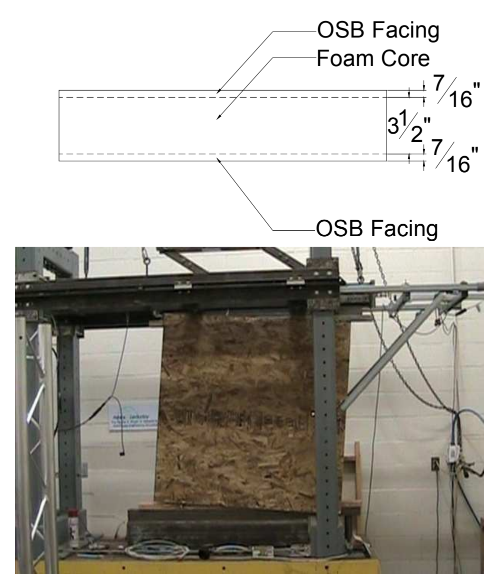


Figure (5): Typical SIP cross-section and test setup [1"=25.4 mm]

Monotonic Testing Procedure

Monotonic tests performed consisted of a static one-directional ramp test, with rates of displacement ranging within 0.02-0.07 in/sec and varying from run to another. Data was continuously recorded at a frequency of 64 samples per second. The monotonic tests were performed in preparation for the quasi-static cyclic tests as recommended by the CUREE-Caltech Woodframe Project (Krawinkler et al., 2000).

Cyclic Testing Procedure

As early as the 1940s, standard racking tests were developed for calculating and predicting the shear strength of wood shear walls (van de Lindt, 2004). Recommendations for cyclic tests evaluating the performance of a complete wall configuration are outlined in ASTM E-564 (ASTM, 1995). However, for the presented tests, the CUREE protocol (Krawinkler et al., 2000) was utilized. This loading sequence is intended for use with tests for which an identifiable deformation parameter relates the component response to the structural system: considering the SIPs, the racking distortion can be directly related to story drift. The protocol serves as an evaluation of capacity level seismic performance of components subjected to ordinary (not near-fault) ground motions with 10% probability of exceedance in 50 years. This protocol is composed of initiation, primary, and trailing cycles based on a reference deformation, Δ, defined as the maximum deformation expected. A monotonic test was performed to determine Δ_m, which is the deformation at which the applied load first drops below 80% of the maximum sustained load. A fraction of Δ_m should be used as the reference deformation, with $\Delta = 0.6\Delta_m$ suggested. This reduction accounts for cumulative damage incurred during a cyclic test which results in discrepancies between the maximum deformations estimated from monotonic and cyclic tests. Data was recorded continuously during the cyclic tests at a rate in the range 20-64 samples per second.

Pseudo-Dynamic Testing Procedure

A single-degree-of-freedom (SDOF) is assumed to represent the computational model of the tested SIP within the framework of pseudo-dynamic testing. The information assumed for this SDOF is based on a previous study conducted on a 1940's design of a two-story wood house over garage (Elkhoraibi and Mosalam, 2007). The equivalent inertia mass of 0.065 kip-sec^2/in of the SDOF was taken as that of the test building. Mass proportional damping was selected with 5% damping ratio. The used time histories for exciting the SDOF model were spectrally matched to the fault normal (FN) and fault parallel (FP) spectra for the design basis earthquake (DBE) and the upper-bound earthquake (UBE), i.e. those with 10% probability of exceedance in 50 and 100 years, respectively. The records from Loma Prieta, CA, 1989 earthquake, Los Gatos station, were adopted assuming a stiff soil site. Three levels of shaking were applied as listed in Table 2.

Table (2): Ground motion levels for the pseudo-dynamic tests on SIPs [1 in=25.4 mm]

Level	Scale	Description	Acc. [g]	Vel. [in/sec]	Displ. [in]
I	0.25×UBE	Elastic (FP only)	0.12	6.36	1.27
II	1.00×DBE	DBE (FP only)	0.42	21.09	3.77
III	1.00×UBE	UBE (FP then FN → long duration)	0.46	25.43	5.06

Results and Discussion

Test results are presented for monotonic and cyclic tests conducted on Specimens 1, 2, and 3 separate from those of the pseudo-dynamic tests conducted on Specimens 4 and 5.

Monotonic and Cyclic Tests

Specimen 1 was tested as a preliminary investigation of the cyclic behavior of SIPs. Before a full cyclic regime was applied, a number of smaller "calibration" motions were applied to ensure that the instrumentation and data acquisition system were operating properly. A peak load of 3.92 kip was carried by the panel at 0.98" of displacement as listed in Table 3.

Using information from Specimen 1, the testing of Specimen 2 represents a more refined approach, incorporating the procedures outlined in the CUREE protocol (Krawinkler et al., 2000). From this monotonic data, the reference displacement of 3.0" was calculated. A single cyclic sequence was applied to the specimen. The overall maximum load of 3.77 kip carried by Specimen 2 occurred during earlier monotonic testing. During the cyclic run, the maximum load was 2.20 kip at 1.88", which highlights the fact that the panel experienced significant damage during the monotonic runs, Table 3, as shown by the asymmetric response with significantly different peak loads and corresponding displacements.

Building on the lessons learned from testing Specimen 2, the same deformation of 3.0" was used as a reference value for designing the cyclic protocol for Specimen 3. In this case, the specimen was not subjected to monotonic runs prior to the cyclic test in order to gain insight into the behavior of an unscathed specimen. In this case, the peak force of 2.62 kip occurred at a deformation of 1.99", Table 3. Small amounts of damage may have been incurred when this specimen was subjected to a small number of initial low-level cycles. Figure 6 presents the load-displacement relationship for Specimen 3.

Table (3): Maximum and minimum loads, cyclic series [1 kip=4.45 kN, 1 in=25.4 mm]

Specimen	Maximum		Minimum	
	Load [kip]	@ Displ. [in]	Load [kip]	@ Displ. [in]
1	3.92	0.98	-3.41	-0.98
2	1.19	2.94	-2.20	-1.88
3	2.62	1.99	-2.51	-1.19

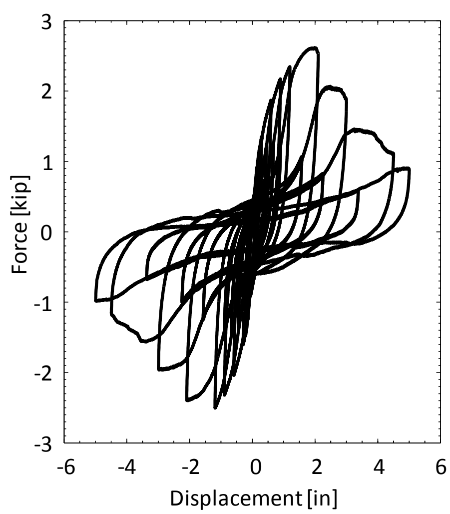


Figure (6): Load-displacement relationship of Specimen 3 [1 kip=4.45 kN, 1 in=25.4 mm]

When evaluating the relative performance of specimens, it is useful to consider the effect of loading rate. Specimen 1 was loaded manually with loading rate varying between approximately in the range 1-3 in/sec within the cycles. The loading rate for Specimen 2 varied in the range 0.03-0.9 in/sec, but the specimen was loaded such that the time required to complete one full cycle was constant at 20 sec/cycle. In contrast, Specimen 3 experienced a constant loading rate of approximately 0.15 in/sec, with the time for completion of a full cycle varying in the range 4-125 sec/cycle. For both Specimens 2 and 3, the loading sequence was automated, in turn providing more regular intervals between cyclic series. The differences in performance of the three specimens can therefore be attributed to a combination of factors, primarily the loading rate, number of cycles, and the presence of initial damage to the specimen due to initial monotonic loading.

In order to change the behavior from a rigid-body rotation to a racking motion, the use of tie-down anchors has proven successful (Jamison, 1997). From the tests performed by Jamison (1997), it was also found that the addition of a second base plate is ineffective in moving the failure mechanism away from the base connection. Schmidt et al. (1994) determined that uplift should be prevented by use of proper anchorage via anchor bolts, as uplift largely contributes to lateral wall displacement. Significant uplift was observed throughout the cyclic tests for all specimens, and should be minimized in future tests.

Failures of all specimens occurred simultaneously along the bottom and top plates of the wall at the connections. Once a number of nails had failed, the wall appeared to rock under rigid body rotation about the center line of the wall. When drywall screws were used in Specimen 1, it was observed that they failed in a highly brittle manner. The nailed connections in Specimens 2 and 3 were slightly more resilient, and could be re-hammered in order to regain some of their function after they started to fail via pull-out. In general, failure was best characterized by OSB splitting at the connection points, nail pull-through or pull-out, and foam crushing as shown in Figure 7 for Specimen 2. With the connections broken, the rigidly rotating wall crushes the foam at either end of the panel, as observed in Figure 8 for Specimen 3. These observations are what would be expected, as experimental tests by Cheung et al. (1988) highlighted the sheathing-to-framing connection as the point of failure, while Stewart et al. (1988) described nail failures, including fracturing of the sheathing nail heads and withdrawal of nail shanks from the framing.

a) Overview of bottom part | b) Detailed top and bottom connections

Figure (7): Failure in Specimen 2

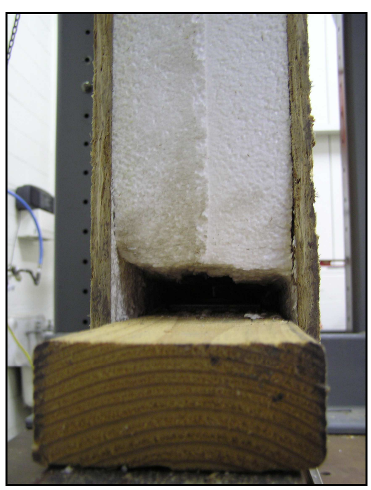


Figure (8): EPS core crushing in Specimen 3

Pseudo-dynamic Tests

Specimens 4 and 5 were tested pseudo-dynamically using the procedures discussed previously and utilizing the ground motions levels listed in Table 2. After levels I and II of earthquake loading, nails were re-hammered to bring capacities back to its original values before application of level III earthquake, Table 2. Both specimens were tested identically for the repeatability of the results, which is confirmed from the comparisons in Figure 9. Therefore, values below are average results from positive excursions of Specimens 4 and 5.

Level I (corresponding to 25% UBE) corresponded to almost elastic response with some energy dissipation through hysteresis and slight pinching response caused by the fasteners (nails) deformation. At this level, the overall peak load of about 1.65 kip was reached at approximately 0.44" displacement. Increasing earthquake level to 100% DBE (level II) results in larger energy dissipation and pronounced pinching response with peak load of about 2.57 kip at approximately 1.41" displacement. Finally, level III (100% UBE) corresponded to loss of shear strength where the load-displacement relationship in positive excursions experienced softening during the strong motion interval. For this level, the peak load was about 2.59 kip corresponding to approximately 0.81" lateral displacement. Adopting a definition of ultimate deformation to correspond to 20% reduction of the lateral load, at 80% of the peak load, the ultimate displacement was 1.82". Assuming the end of the elastic response corresponded to the peak of level I (this is also confirmed by the response to level II), the ultimate displacement ductility can be defined as 1.82/0.44=4.14.

Concluding Remarks

The paper presented test results for material characterization of SIPs and CSIPs subjected to shear stresses. Racking test results of SIPs using quasi-static and pseudo-dynamic techniques represented the main part of the paper. The results confirmed the validity of pseudo-dynamic testing to characterize the SIPs seismic performance with less sensitivity to test parameters than quasi-static testing. Preliminary conclusions include reasonable energy dissipation and ultimate displacement ductility slightly above 4.0 for SIPs without panel-to-panel connections. SIP strength was maintained up to and including 100% of the design basis earthquake (DBE) – 10% probability of exceedance in 50 years. Moreover, significant reduction of strength with large energy dissipation was observed for a longer duration upper-bound earthquake (UBE) – 10% probability of exceedance in 100 years.

For future investigation, one may suggest the following improvements:

- A thorough investigation of the development of common connection types would be beneficial, as this is the most likely point of failure in SIPs and CSIPs. Both panel-to-panel and panel-to-diaphragm connections should be considered. Of special importance is the function of the adhesive within the connections, and whether its use represents any improvements of the performance.
- Developing coupled computational tools for SIPs and CSIPs to account for thermal and structural behavior can advance this research beyond the realm of structural engineering to treat SIPs and CSIPs designs in the context of optimization problems.

- From sustainability point of view and due to increased environmental awareness, life-cycle analysis and assessment of SIPs and CSIPs is an important task.

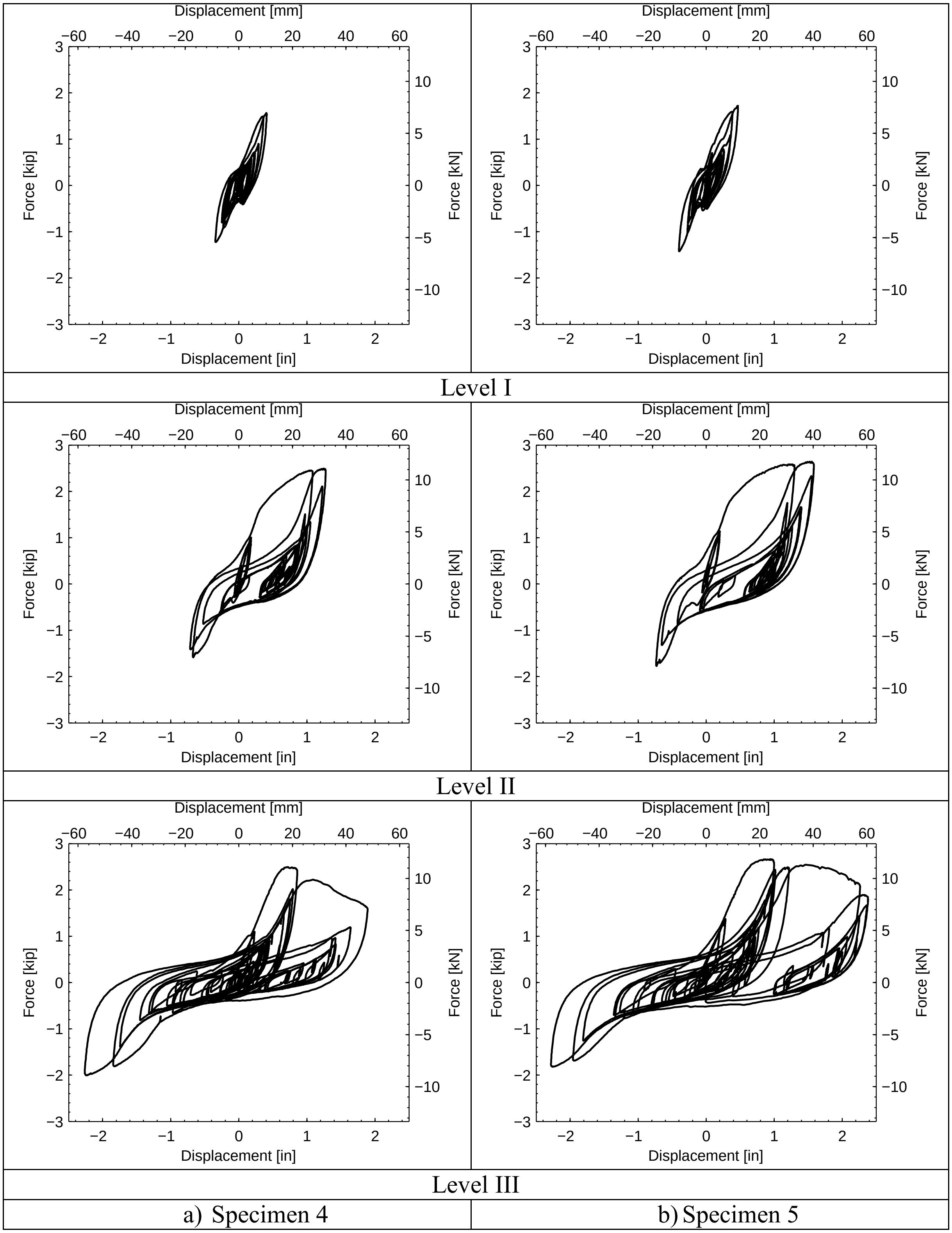


Figure (9): Similar pseudo-dynamic test results from Specimens 4 and 5

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