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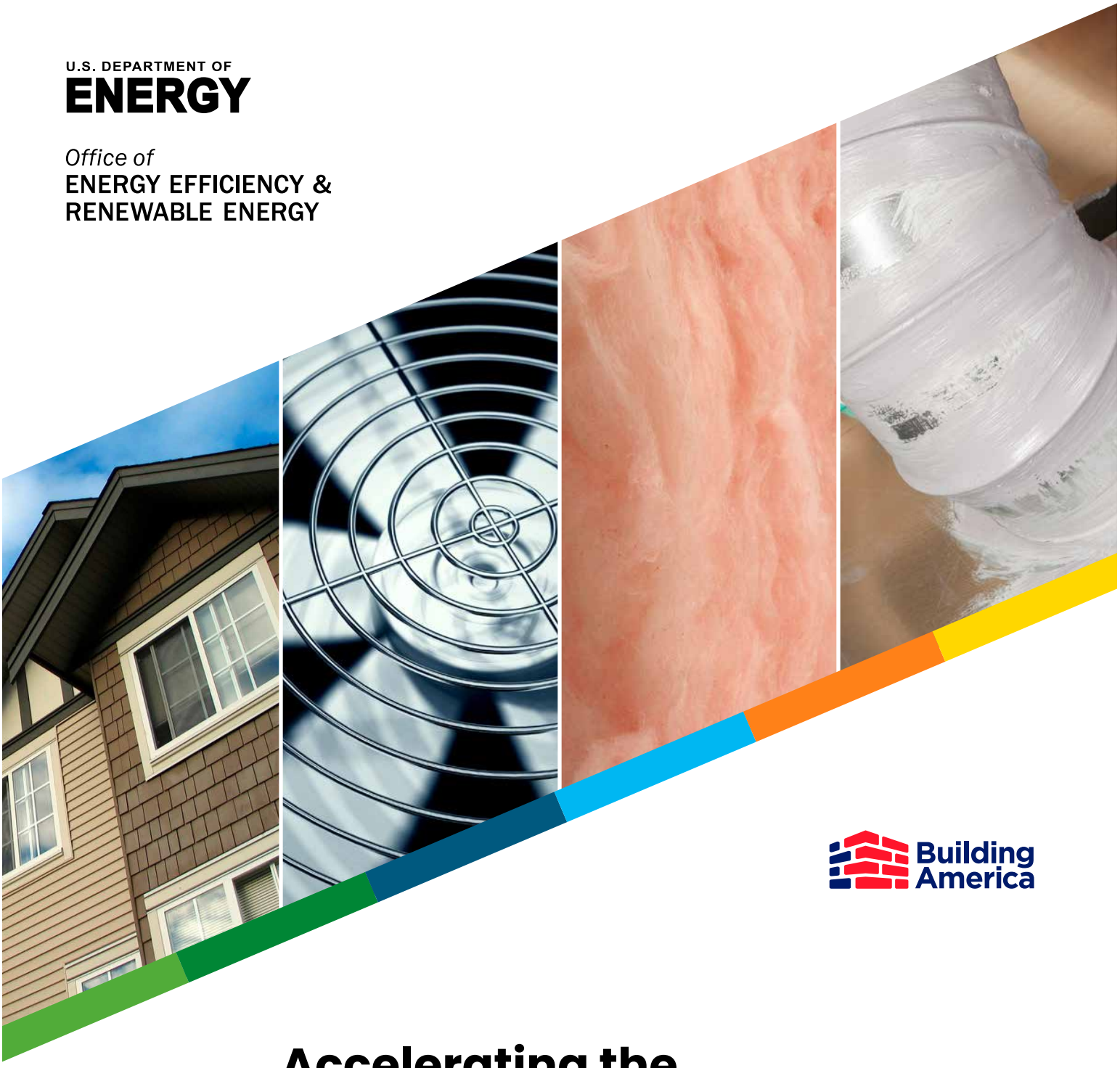

Building America

Accelerating the Adoption of the Solid Panel Structural System

June 2024

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Accelerating the Adoption of the Solid Panel Structural System

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This material is based upon work supported by the Department of Energy's Office of Energy Efficiency and Renewable Energy (EERE) under the Building Technologies Office under Award Number DE-EE0008693.

The work presented in this EERE Building America report does not represent performance of any product relative to regulated minimum efficiency requirements.

The laboratory and/or field sites used for this work are not certified rating test facilities. The conditions and methods under which products were characterized for this work differ from standard rating conditions, as described.

Because the methods and conditions differ, the reported results are not comparable to rated product performance and should only be used to estimate performance under the measured conditions.

Foreword

The U.S. Department of Energy (DOE) Building America Program has spurred innovations in building efficiency, durability, and affordability for more than 25 years. Elevating a clean energy economy and skilled workforce, this world-class research program partners with industry to leverage cutting-edge science and deployment opportunities to reduce home energy use and help mitigate climate change.

In cooperation with the Building America Program, the University of Minnesota's NorthernSTAR Building America Team is one of many **Building America teams** working to drive innovations that address the challenges identified in the program's **Research-to-Market Plan**.

This report, *Accelerating the Adoption of the Solid Panel Structural System*, explores the structural properties, capabilities, and applications for an innovative "studless" building system that uses a plate-like approach and exemplifies "Perfect Wall"[1] principles. A comprehensive testing regime was completed to fully characterize the structural behavior of the Solid Panel Structural system. These results were used to develop an engineering guide that can be used by architects and engineers for design and code approval. In addition, a small survey of builders and subcontractors was conducted to determine key issues for market acceptance and deployment.

As the technical monitor of the Building America research, the National Renewable Energy Laboratory encourages feedback and dialogue on the research findings in this report as well as others. Send any comments and questions to building.america@ee.doe.gov.

[1] "Perfect Wall" is a term that was coined and popularized by Joseph Lstiburek in "BSI-001: Perfect Wall" (Lstiburek 2010). It does not refer to a specific wall type nor a wall without defect. Rather, it is referring to a wall system strategy that places the four control layers for water, air, heat, and vapor outboard of the building's structural system and subsequently covers these layers with a cladding system that can drain and dry. This approach has the potential to be more efficient, durable, and resilient than cavity-insulated, light-framed wall systems.

Acknowledgments

The work presented in this report was funded by the U.S. Department of Energy (DOE), Office of Energy Efficiency and Renewable Energy, Building Technologies Office.

The research was conducted by the University of Minnesota's Northern*STAR* Building America Team with guidance and support from several industry partners. The authors thank these partners and their dedicated staff for their contributions to this project:

- Tom Schirber, Marilou Cheple, Garrett Mosiman, and Rolf Jacobson from the University of Minnesota
- Kevin Kauffman, Deanna Seale, and Tynika Stefun of Home Innovation Research Laboratories
- Kurt Koch, Paigh Bumgarner, Stuart Brock, and Michael Pyle of Huber Engineered Woods.

The authors also wish to acknowledge the support that we have received from the project and technical managers at DOE's Building Technologies Office and the National Renewable Energy Laboratory as well as the many Building America partners that provided feedback on the Solid Panel Structural system. The frequent open and candid conversations about this project with Building America colleagues provided additional insights into potential directions that could enhance this innovative building technology and its market adoption.

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List of Acronyms

ASD	allowable stress design
ASTM	American Society for Testing and Materials
AWC	American Wood Council
CS-WSP	Continuous Sheathing Wood Structural Panel
DOE	U.S. Department of Energy
HUD	U.S. Department of Housing and Urban Development
HVAC	heating, ventilating, and air conditioning
ICF	insulating concrete forms
IRC	International Residential Code
MEP	mechanical/electrical/plumbing
MOE	modulus of elasticity
MOI	moment of inertia
MOR	modulus of rupture
NAHB	National Association of Home Builders
NDS	National Design Specification
OSB	oriented strand board
plf	pounds per linear foot
psf	pounds per square foot
PSW	Perforated Shear Wall
S-TAG	Structural Technical Advisory Group
SDPWS	Special Design Provisions for Wind and Seismic
SIP	structural insulated panels
SPF	spruce pine fir
SPS	Solid Panel Structure
SYP	southern yellow pine
UTM	universal testing machine
WRB	weather resistive barrier

EXECUTIVE SUMMARY

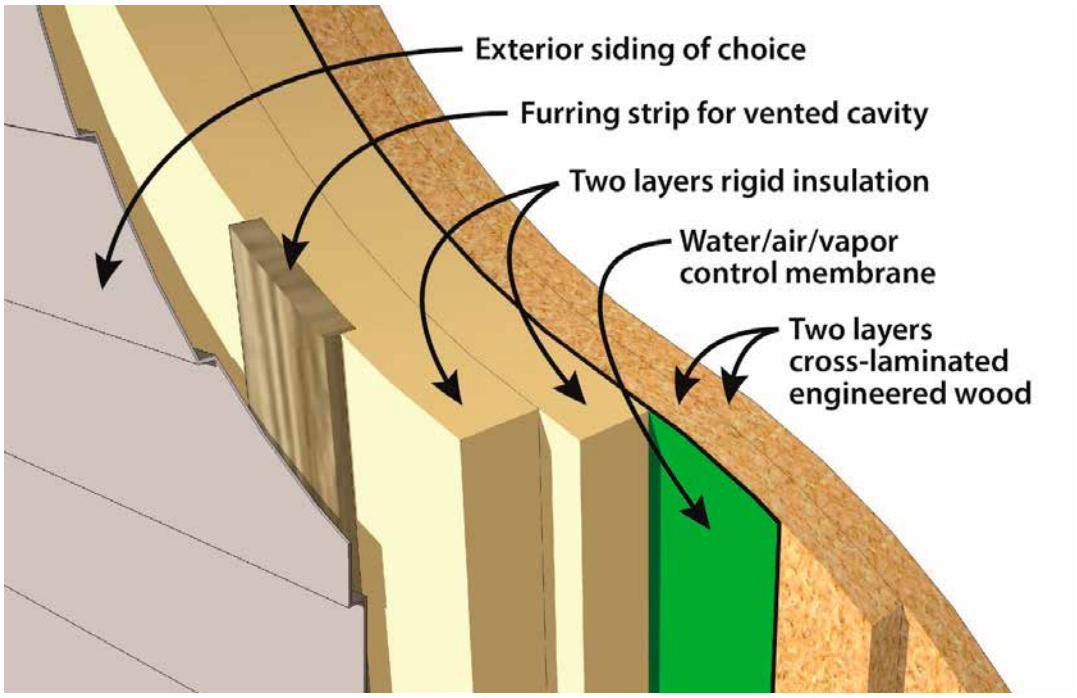
Problem Statement

The primary goal of this project was to position the Solid Panel Structural building system and its "Perfect Wall"[2] approach to thermal and moisture management for improved market adoption. A previous U.S. Department of Energy (DOE) project provided significant market validation for the Solid Panel Structural system within the single-family affordable housing market. The objectives for this project were to accelerate the market adoption process with high-level structural testing that would improve code acceptance and prepare a pathway to move this innovative building system and technology into the residential market.

In 2021, the University of Minnesota Northern*STAR* Building America team completed its DOE-funded project on the Affordable Solid Panel "Perfect Wall" system (Schirber et al. 2022). That project demonstrated and evaluated a novel building assembly called the Solid Panel Structure (SPS), which uses large-format (8' x 24') oriented strand board (OSB) panels to create the wall structure. In addition to being an innovative structural system, the SPS is a unique interpretation of the "perfect wall" concept, in which all of the environmental control layers are located on the exterior side of the structural components.

The primary objective of the previous study was to validate the SPS technology in terms of its constructability, cost, and performance. The overall performance of the SPS system in that research was very encouraging and the constructability

[1] "Perfect Wall" is a term that was coined and popularized by Joseph Lstiburek in "BSI-001: Perfect Wall" (Lstiburek 2010). It does not refer to a specific wall type or a wall without defect. Rather, it refers to a wall system strategy that places four control layers for water, air, heat, and vapor outboard of the building's structural system and subsequently covers these layers with a cladding system that can drain and dry. This approach has the potential to be more efficient, durable, and resilient than cavity-insulated, light-framed wall systems.



SPS wall system. *Image from UMN project team*

and cost data show clear potential for gains in new home construction. However, while the SPS system demonstrated relative ease of construction with limited skilled labor and comparable cost to code-compliant 2" x 6" walls, along with superior energy and moisture performance, there were remaining questions related to the structural aspects and code compliance of this novel plate-based system. Before the SPS system could see widespread adoption on a broader scale, additional data would be needed to better characterize the structural behavior and performance of the SPS.

Research Objectives

The research and work plan for this follow-up study were focused on a sequential set of structural tests to develop and validate an engineering basis for the SPS building system. That structural testing plan was directed by a third-party structural consultant with review and guidance from a Structural Technical Advisory Group (S-TAG) and was conducted by the Home Innovation Research Laboratories.

The project partners developed and executed a high-level structural testing protocol for the SPS system as a structural unit and guided the testing at a level such that an International Code Council – Evaluation Service Report or a model building code change could be developed and submitted in the future. This testing (and subsequent model code acceptance work) would facilitate local code acceptance across the nation. This research provided valuable information on material properties, fastening protocols, and critical design limits.

The project leadership team established and convened an S-TAG to guide the project's research in a way that will ensure broader acceptance of the SPS building system. In addition, a pilot market survey of key builders and subcontractors was conducted by Home Innovation Research Laboratories to identify perceptions of this novel system along with potential construction barriers and opportunities for widespread adoption by the homebuilding industry.

Methodology

All buildings, components, or assemblies must comply with at least the minimum performance intent of the building code. For structural systems like SPS, the key measure is the ability to safely resist

various structural loads (including both live and dead loads) from people, building contents and materials, and natural events such as snow, wind, and earthquakes. This code compliance evaluation objective is permitted by U.S. model building codes to be satisfied in two ways:

- Develop engineering design methodology consistent with code-recognized design theory and standards with confirmation by representative component, assembly, and material property test data.
- Demonstrate direct equivalency through a comparative component or assembly testing approach.

Both of these approaches to code compliance evaluation and verification are valid in that:

- Both are recognized in the code and code-referenced standards as code-compliant ways to determine acceptability or code compliance.
- Both represent successful experience (baselines or benchmarks) from which to assess acceptability or code compliance.
- Both rely on relevant performance data from accepted methods of testing (whether used to compare test results to an existing engineering analysis theory or model, or to compare directly to the tested performance of an accepted construction material and method recognized prescriptively in the code).

Each approach ultimately relies on a representation of and some form of comparison to "what has worked in the past," and, consequently, both are empirical at their roots. In addition, both evaluation approaches are interchangeable. One approach may be used to evaluate code compliance for one structural attribute, and the other may be used for a different structural attribute of same building system, assembly, or component.

Recognizing the potential role of each approach, an overall evaluation strategy was developed for the SPS construction technology. As part of the process of considering a specific evaluation strategy—including a test plan, test methodology, data analysis, and acceptance criteria to fit the implementation strategy for the SPS building technology—it is important to:

- Identify the specific material properties and performance attributes that must be addressed to demonstrate code compliance.
- Identify and control significant sources of uncertainty and bias that may attend any particular approach to code compliance for a particular building technology, such as selection of test methods and performance benchmarks.
- Consider precedents (e.g., technical literature, previous evaluation experiences, code requirements, standards) that are relevant to the building system, assembly, or component being evaluated.
- Develop an evaluation approach that balances the technical demands for a desired scope of application with the available budget and resources to conduct the code evaluation.

For structural safety purposes, the key performance attributes that must be quantified for the SPS wall construction technology are:

- Compressive load resistance of SPS wall.
- Bending load resistance and stiffness (out-of-plane loads) of SPS wall.
- Combined bending and compressive load resistance of SPS wall.
- In-plane shear (racking) load resistance of SPS wall as affected by a range of window and door opening conditions.
- SPS performance as it forms an unconventional deep, slender header as the exterior ply extends upwards from the wall opening to the top of the floor truss or raised roof truss heel.
- Floor and roof truss connection and bearing support performance of an SPS wall.

Important Results

Overall, the SPS testing was successfully implemented and its performance demonstrated the system could meet or exceed code requirements. Data from each of the sequential tests were used to characterize the structural behavior such that a simplified design and engineering guide could be developed. This would aid designers in implementing the SPS system in a structurally smart manner and could be used to demonstrate compliance for local code officials.

A spreadsheet was developed to implement the National Design Specification column buckling equations presented using the design parameter findings and approach recommendations reported and derived from the 1-ply and 2-ply bending stiffness tests. To predict the tested maximum axial load capacity, the material properties used were those representing the average modulus of elasticity (MOE) and average ultimate material stress properties derived from a random sample of OSB panel materials used in the overall test project.

From the testing data and subsequent analysis, a step-by-step design and engineering guide was developed for designers. This will enable a designer or engineer to sign off on the drawings to ease permit application and approval. Furthermore, a manufacturer, fabricator, or builder could use the data from the study to develop an International Code Council Evaluation Service report to gain even broader acceptance on the part of market and code officials.

From the small market survey, the primary benefit of the SPS when compared to conventional light-frame, cavity-insulated construction was the speed of construction. Other key benefits that were mentioned included overall labor cost savings, simplification of the construction process, and smaller crew sizes needed to erect the enclosure. They also recognized the potential for improved thermal performance, moisture control, and reduced air infiltration along with an apparent strength advantage for wind resistance.

The survey respondents did voice some perceived concerns. The most obvious were access, site logistics, and cost for a crane and skilled operator. They also mentioned the possible limitations to building design, shapes, and sizes. Additionally, there was some concern that nontraditional interior details (e.g., electrical raceway) may not be accepted by homebuyers.

Respondents mentioned some other possible drawbacks including the lack of widespread availability of the OSB panels and lead time for ordering and shipping. It was also perceived that there might be challenges with local building inspectors. Several respondents mentioned moisture concerns and possible deterioration of the OSB.[3] In addition, there was some uncertainty expressed about what types of exterior cladding might work with the SPS system.

However, respondents shared several ideas on where the SPS might be a good fit. The primary opportunity mentioned was affordable housing, especially if the system could be offered as a turnkey building enclosure solution (materials and installation). It would be suitable for projects where home designs and key exterior dimensions are repeated or where minimal customization is needed. Due to the speed of erection and dry-in, it might be more desirable in areas with many weather-related delays.

[3] While this concern is understood, it is important to note that the structural OSB resides entirely within the conditioned volume of the home and is completely covered by the water, air, vapor, and thermal control layers. Therefore, moisture damage and deterioration should be a minimal concern.

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1 Introduction

1.1 Overview and Problem Statement

The primary goal of this project was to position the Solid Panel Structural (SPS) building system and the "Perfect Wall"[1] thermal and moisture management approach for market adoption. A previous U.S. Department of Energy (DOE) project provided significant market validation for the SPS system within affordable housing (Schirber et al. 2022). The objectives for this project were to accelerate the market adoption process with high-level structural testing to facilitate code acceptance and prepare a pathway to move this building system and technology into the residential market. This whole-house building system works in all climates zones and with most conventional housing designs.

1.2 Background

In 2021, the University of Minnesota Northern*STAR* Building America team completed its DOE-funded project on the Affordable Solid Panel "Perfect Wall" System (Schirber et al. 2022). That project demonstrated and evaluated a novel building assembly—the SPS—which uses large-format (8' x 24') oriented strand board (OSB) panels to create the wall structure. In addition to being an innovative structural system, the SPS is a unique interpretation of the "Perfect Wall" concept, in which environmental control layers are located on the exterior side of the structural components, as opposed to more traditional cavity-insulated, stud-framed walls.

[1] "Perfect Wall" is a term that was coined and popularized in "BSI-001: Perfect Wall" (Lstiburek 2010). It does not refer to a specific wall type nor a wall without defect. Rather, refers to a wall system strategy that places the four control layers for water, air, heat, and vapor outboard of the building's structural system and subsequently covers these layers with a cladding system that can drain and dry. This approach has the potential to be more efficient, durable, and resilient than cavity-insulated, light-framed wall systems.

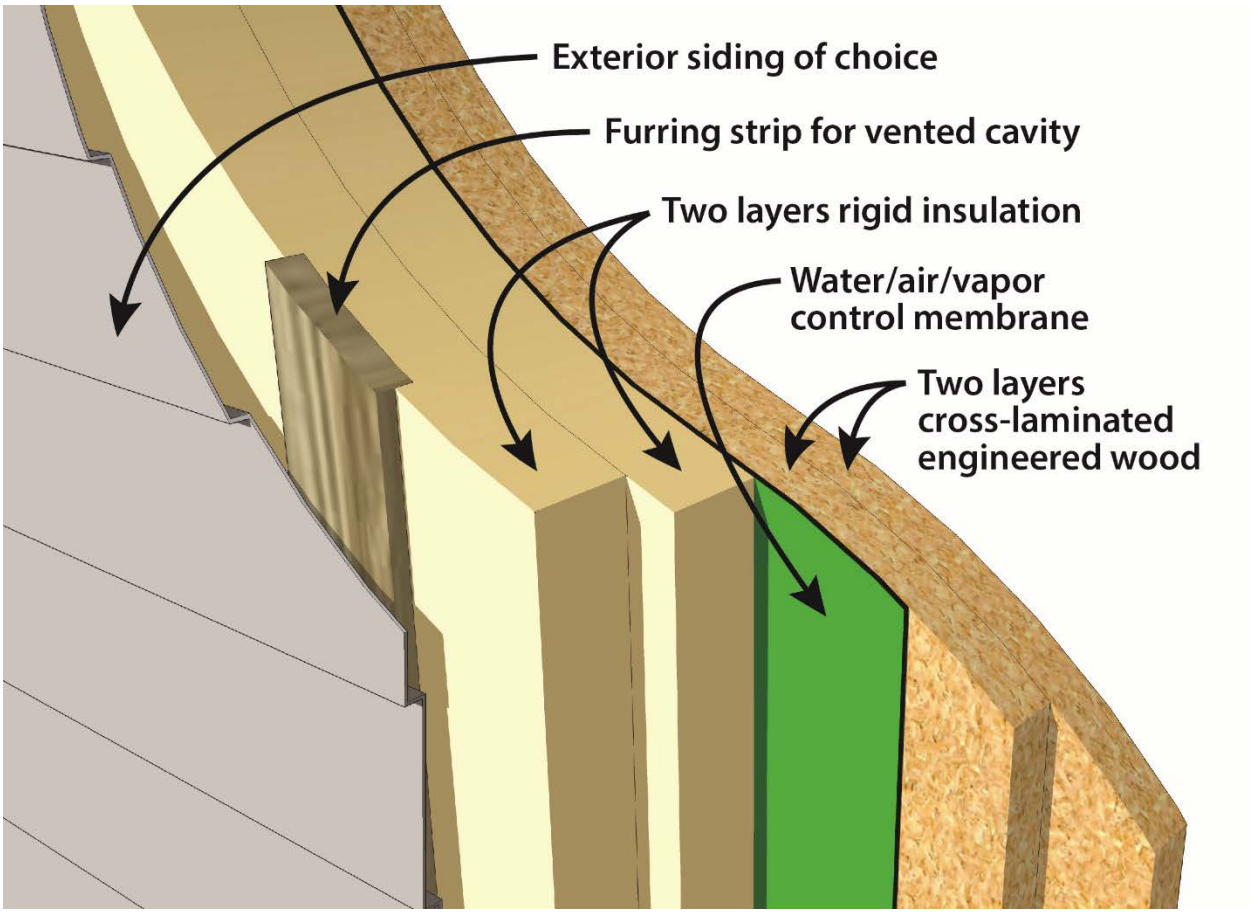

Figure 1. SPS wall system

The primary objective of that study was to validate the SPS technology in terms of its constructability, cost, and performance. The SPS system performance in that research was very encouraging, and the constructability and cost data show clear potential for gains in new home construction. However, while the SPS system demonstrated relative ease of construction with limited skilled labor, comparable cost to code-compliant 2" x 6" walls, and superior energy and moisture performance, there were many remaining questions regarding the structural aspects and code compatibility of this novel plate-based system. Before the SPS system could see widespread adoption on a broader national scale, additional data would be needed, specifically, in relation to the structural behavior of the SPS.

1.3 Scope and Objectives of this Study

The research and work plan for this follow-up study were focused on a sequential set of structural tests to develop and validate an engineering basis for the SPS building system. That structural testing was directed by a third-party structural consultant with guidance from a Structural Technical Advisory Group (S-TAG) and was conducted by Home Innovation Research Laboratories.

The project partners developed and executed a high-level structural testing protocol for the SPS system as a structural unit and guided the testing at a level such that an International Code Council – Evaluation Service Report or a model building code change could be developed and submitted in the future. This testing (and subsequent model code acceptance work) will facilitate local code acceptance across the nation. This research also informs us on material properties, fastening protocols, and critical design limits. The leadership team established and convened a Structural Technical Advisory Group to guide the project's research in a way that will ensure broader acceptance of the SPS building system. In addition, a pilot market survey of key builders and subcontractors was conducted by Home Innovation Research Laboratories to identify perceptions of this novel system along with potential construction barriers and opportunities for widespread adoption by the home building industry.

1.4 Basics of the Solid Panel Structural System

The SPS building system is based on an innovative structural approach. Traditional stud-frame platform construction uses a simple column (studs) and beam (headers and plates) design for the wall systems. Sheathing is added to stiffen the wall members, transfer loads across members, and provide resistance to buckling and shear. The floor and roof systems sit on top of these walls and are connected to act as horizontal diaphragms to provide overall building stiffness and the transfer of shear loads. In contrast, the structural panels for SPS system uses large panels to form these wall elements as shown in Figure 2.

Figure 2. SPS home under construction

The SPS system brings a novel construction approach to home building that is potentially stronger, more cost-effective, and ultimately better than stud-framed buildings. Because the wall panel is solid, the only way to make the structure high-performing is by adding the control layers to the exterior of the structure. This structural

system creates a built-in incentive for builders to upgrade the whole house with “Perfect Wall” components. The SPS wall system can be:

- Stronger: Panels are assembled to provide a monolithic structure for all four walls. The panels are high quality and engineered for strength. The monolithic structure is engineered to withstand loads including wind, shear, and vertical loads at higher rates than stud-framed structures.
- Less Expensive: Depending on the lumber market, the OSB composite panel cost can be more stable and comparable in price to dimensional lumber. The framing assembly can be less expensive because skilled framers are not required, and it takes fewer workers (but requires a crane) to build. The wall and floor system can be installed faster than stud framing.
- Better: The SPS wall system, with fewer seams and continuous exterior control layers, makes it easier to manage heat, air, and moisture with virtually no errors or leaks. In addition, the solid panels create the opportunity for the builder to eliminate drywall on perimeter walls as well as expensive floor coverings. The panels used in this project are industrial OSB panels that are 1-1/8” thick and come in 8’ x 24’ sheets. The vertical exterior panels and horizontal interior panels are cross-laminated on site to simultaneously serve as the columns, beams, and sheathing. The exterior panel runs vertically from the foundation sill plate all the way past the vertical leg of the raised-heel roof truss. A second interior panel runs horizontally between the floor and roof elements. Once fastened together, these two panels act like a singular plate or diaphragm from foundation to roof and from corner to corner. Wall plates are securely fastened to each other and interlocked with the horizontal floor and roof diaphragms as shown in Figure 3. An intermediate floor is not shown in Figure 3, but would be connected to the outer SPS ply of the OSB and supported by a 1-1/8” bearing shelf created by a discontinuity of the inner SPS ply. At this point the system is analogous to a monocoque-like structural shell.

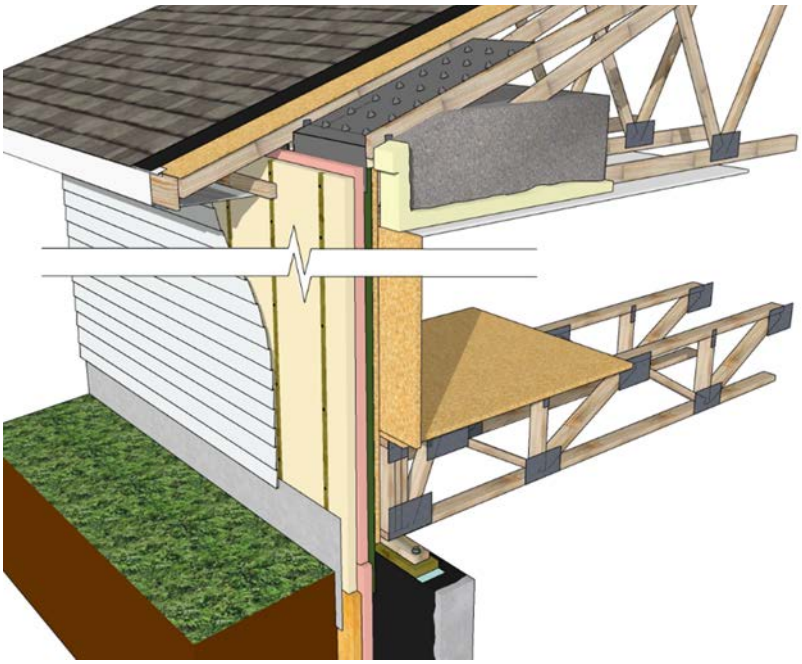

Figure 3. SPS wall system including intersection at roof system and floor-foundation system

Finally, the SPS wall system follows the principles of the “Perfect Wall” approach and employs multiple strategies to keep the critical structural panel moisture-safe. The SPS wall provides a more robust method of air and water leakage control than typical cavity-insulated light frame construction. Furthermore, the continuous exterior insulation places the sheathing in a warmer, more protected position. Both modeling and monitoring of the sheathing moisture content have clearly demonstrated that the SPS wall remained more stable and consistent.

1.4.1 Solid Panel Structure Design Considerations

There are several key design features of the SPS system. Although this panel (or plate) system can be quite flexible, early designs focused on optimizing the dimensions to fully utilize the 8’ x 24’ panels. The most predominant house design thus far has been 24’ x 32’. This cross-laminated panel approach can easily accommodate normal window and door openings. However, it is preferable to avoid vertical seams to maintain panel plate integrity. The system uses three vertical panels for the front and rear elevations and four vertical panels for the side elevations, as shown in Figure 4. Advanced planning for the two-story design (assuming 8’ walls, two 18” floor systems, and 12” roof truss leg) will leave a 4’ x 8’ panel that can be used for interior applications. The horizontal panels go corner to corner on the front and back elevations. Two horizontal panels are needed for the longer sides, with a seam that is generally hidden at an interior partition wall.

1.4.2 Solid Panel Structure Delivery Sequence

A site-fabricated building system is shown below in Figure 4. The exterior frame walls are replaced by large-format OSB panels. The current two-story design requires 24

panels for the cross-laminated wall system and an additional eight panels to use as floor sheathing on the first and second floor. These replace the exterior studs, headers, plates, and sheathing. Once the foundation has been poured, exterior control layers installed, and rough backfill completed, the full sheet OSB structural panels arrive on site and are set near the building site.

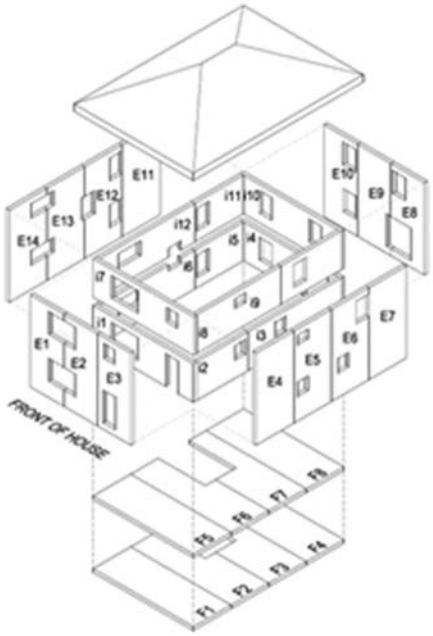

Figure 4. Panel layout for SPS

A cut sheet is provided for each house design. While some panels are used without cutting, many will require a single cut in preparation for erection. These cuts are easily completed while the crane is setting a previous panel. A special set of grabbers (commonly used for sheet steel) are used to lift panels in both the vertical position for walls and the flat condition for the floors.

- Site Preparation and Excavation: All site preparation and excavation activities are the same as in typical construction.
- Footings, Foundation Wall, and Basement Slab: All the project homes used cast-in-place concrete footings and foundation walls similar to typical construction. All basements include one or more egress windows to accommodate code requirements and a future bedroom. These must be integrated into the exterior water and thermal control layers.
- First-Floor Platform: The first-floor platform is installed on the foundation. This process is similar to traditional floor construction with a couple of significant differences. The traditional sill plate is replaced by a sill plate receiver for the vertical panel as shown in Figure 5. It is composed of a regular sill plate over a sill plate sealer with a second receiver plate (usually one dimension smaller than

the sill plate) and spaced 1-1/8” from the outer edge of the sill plate. Both plates are carefully squared and measured to match the panel dimensions and then fastened to the foundation to meet code requirements. The vertical panel sits on the sill plate and is fastened to the receiver plate. Once the sill plate receiver is in place, the floor trusses are set and the OSB panels are delivered.

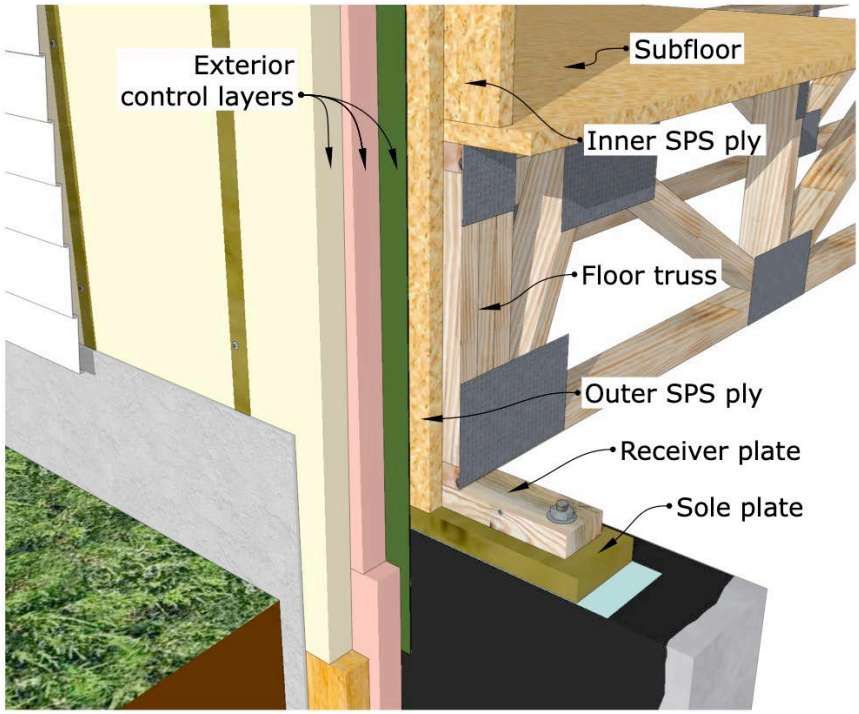

Figure 5. SPS wall configuration a floor-foundation intersection

- Exterior Wall Vertical Panel Erection: At this point, the crane arrives on-site. The first-floor sheathing panels are installed. The first vertical wall panel is cut to size at the pile. The crane lifts this panel to one of the rear corners where it is temporarily braced. The next panel is cut and set to the same corner. These panels are securely fastened in the corner and braced to be square and plumb. This is repeated for the remaining three corners. The remaining vertical panels are then installed. To provide better sightlines for the crane operator, the middle panel nearest the crane is not installed until after the completion of the shell.
- First-Floor Horizontal Panel: The horizontal interior panels are placed on all four walls. The front and rear panels are full sheets, but the longer side elevations require a full sheet plus a partial panel. This partial panel is strategically placed so the vertical seam can be concealed by an interior partition wall. If the interior partition walls are panels or pre-framed, they can be loaded onto the first-floor platform.

- Installation of Second Floor: The second-floor joists are installed inside the exterior panels and on top of the interior panel. The interior panel provides dimensional registration but is not a ledger for bearing. Instead, the floor trusses are fastened in place with designated screws from the outside through the exterior panel and into the vertical blocking at the end of the trusses. The second-floor sheathing is then installed.
- Exterior Wall Second-Floor Horizontal Panel: This panel is placed in the same manner as the exterior wall first-floor horizontal panel, above.
- Roof Trusses and Sheathing: Roof trusses are set in a manner similar to that used in typical construction with one notable exception. The trusses are designed to have a vertical leg that will sit inside the exterior panel and on top of the interior panel. The rafters are fastened in place with designated screws from the outside through the exterior panel and into the vertical truss leg. This type of attachment of each truss as it is erected simplifies truss bracing requirements. The top chord of the truss extends outward to make the roof overhang. The remainder of the roof construction is the same as typical construction. At this point the crane work is complete.
- Window Openings: For the SPS houses in this study, the window selection was simplified to three window sizes. This allows for the use of a simple jig to properly lay out and cut each window opening. The openings can be cut out with a worm drive circular saw, heavy-duty reciprocating saw, or small chainsaw. The larger window size provides a 3’ cut-out that can be used for the stair treads. Window units with a narrow profile fit cleanly in the OSB structural panels and can be conventionally trimmed on the interior.
- Exterior Control Layers and Window Installation: This phase is quite different than typical construction and features several key steps. The first step is priming the OSB and beginning installation of the fully adhered “peel and stick” membrane from foundation up to the head of the first-floor windows and doors. This includes the preparation and proper integration of all penetrations within the first-floor system. Next, the first-floor windows can be installed with a panned sill, compatible jamb tape, and proper integration, sealing, and flashing at the head. The primer and membrane are then installed on the upper level along with the second-floor windows. The exterior rigid board insulation is then installed with two layers staggered at both vertical and horizontal seams. The first insulation layer is set into place starting at the foundation and can be tacked minimally to the wall as needed. The second insulation layer is then placed over the first and the furring strips are installed and secured to the OSB panel.

- Exterior Finishes: The exterior cladding and trim are installed over 3/4” furring strips fastened through the foam and into the exterior OSB panel. For vinyl or metal siding, the furring strips are either embedded in the rigid board foam, or 3/4” foam board is added between the strips to support the cladding. For wood or fiber cement siding, a 1”x 4” furring strip is used.
- Interior Framing: Interior framing can be installed in the same manner as typical stud-frame construction. However, it is possible to use the 1-1/8” OSB panels for interior walls. If these panels are used as partitions, a furring strip is added around the perimeter of the door openings to accommodate normal door jamb thickness and trim.
- Mechanical/Electrical/Plumbing (MEP) Rough-in: The MEP configuration is very similar to that of typical construction. However, a key scheduling benefit and opportunity to further shorten the construction cycle is that the MEP rough-ins can occur immediately after the enclosure has been completed. One significant difference is that all MEP penetrations to the exterior have been preplanned and placed once the SPS walls have been completed. There are special sleeves and devices for each opening to ensure they can be integrated with the fully adhered water and air control membrane. In general, the MEP contractors must use the opening that was provided for them and are not allowed to drill any holes to the outside without guidance from the enclosure contractor. Additional plugged openings are provided to meet future needs, such as patio or backyard lights, grills, or firepit. While penetrations can be made after the exterior control layers have been installed, maintaining the integrity of the water and air barrier requires special care and skill.
    - Heating, ventilating, and air conditioning (HVAC) design, equipment, and installation are very similar to typical high-performance stud-frame construction.
    - Electrical installation is similar with one notable exception. The electrical outlets on the perimeter structural panels are contained in tall and slightly deeper baseboards. Two horizontal furring strips are placed on the wall where the wiring and outlet boxes are installed. A cover board, such as a stair skirt, with a trim molding at the top is installed. Generally, with the exception of entrances, there are no light switches on exterior walls. To accommodate typical door frame depths, the wall is typically furred out. This surround can be extended to accommodate any needed light switches.
    - Plumbing design and installation are very similar to typical high-performance stud-frame construction. Generally, vertical plumbing is avoided on the exterior walls. However, if necessary, it simply requires the wall to be furred

out. If the OSB panels are used for interior partitions, the plumbing wall simply adds a spacer, which is covered by a second OSB panel. With careful planning, this panel can be made removable to access, inspect, or repair the plumbing.

- Interior Finishes: Interior finishes can be the same as typical residential construction, including surface finishes, cabinetry, and trim. However, there are a couple of optional exceptions in the cases of the exterior wall finish and the floor finish. The OSB panels can be covered with drywall and finished in the typical manner, or the walls can be primed and painted. This can provide an acceptable and durable finish at a much lower cost. This can be further enhanced with a fogged or knock-down primer coat prior to painting to provide a very attractive and highly durable finish. If the OSB flooring sheathing panel is protected during construction, it is easy to sand and finish with several coats of polyurethane. When sanded, these panels have a marbled (not flaky) appearance and make a very attractive and durable floor surface.
- Mechanical/Electrical/Plumbing Final: All of the MEP final fixtures, hook-ups, and finishes are the same as in typical construction.

It should be noted that the SPS “perfect wall” building system can easily and affordably meet the requirements of the DOE Zero Energy Ready Home program.

Figure 6. Completed SPS home from previous DOE project

2 Structural Testing Methodology

2.1 Introduction

All buildings, components, or assemblies must comply with at least the minimum performance intent of the building code. For structural systems like SPS (see Figures 1–5 above), the key measure is the ability to safely resist various structural loads from people, building contents, and materials along with natural events such as snow, wind, and earthquakes. This code compliance evaluation objective is permitted by U.S. model building codes to be satisfied in two ways:

- Develop engineering design methodology consistent with code-recognized design theory and standards with confirmation by representative component, assembly, and material property test data
- Demonstrate direct equivalency through a comparative component or assembly testing approach.[2]

Both of these approaches to code compliance evaluation and verification are valid in that:

- They both are recognized in the code and code-referenced standards as code-compliant ways to determine acceptability or code compliance.
- They both represent successful experience (baselines or benchmarks) from which to assess acceptability or code compliance.
- They both rely on relevant performance data from accepted methods of testing (whether used to compare test results to an existing engineering analysis theory or model, or to compare directly to the tested performance of an accepted construction material and method recognized prescriptively in the code).

Both approaches ultimately rely on a representation of and some form of comparison to "what has worked in the past," and, consequently, both are empirical at their roots. In addition, both of the above evaluation approaches are interchangeable. One approach may be used to evaluate code compliance for one structural attribute, and the other approach may be used for a different structural attribute of same building system, assembly, or component. Regardless of which method is used to demonstrate code compliance, an evaluation strategy must be developed for the SPS construction technology, which is addressed in the next section.

[2] The key building code provision supporting the "equivalency approach" is Section 104.11 of the *International Residential Code* and the *International Building Code,* "Alternative materials, design and methods of construction and equipment." An example of a code-referenced design standard's support of the equivalency approach as well as the engineering approach (which is really another form of equivalency) is Section 1.1.1.5 of the National Design Specification (NDS) for Wood Construction.

2.2 Evaluation Strategy

In the process of considering a specific evaluation strategy—including a test plan, test methodology, data analysis, and acceptance criteria—to fit within the implementation strategy for the SPS building technology, it is important to:

- Identify the specific material properties and performance attributes that must be addressed to demonstrate code compliance
- Identify and control significant sources of uncertainty and bias that may attend any particular approach to code compliance for a particular building technology
- Consider precedents (e.g., technical literature, previous evaluation experiences, code requirements, standards) relevant to the building system, assembly, or component being evaluated
- Develop an evaluation approach that balances the technical demands for a desired scope of application with the available budget and resources to conduct the code evaluation.

For structural safety purposes, key performance attributes that must be quantified for the SPS wall construction technology are:

- Compressive load resistance of SPS wall
- Bending load resistance and stiffness (out-of-plane loads) of SPS wall
- Combined bending and compressive load resistance of SPS wall
- In-plane shear (racking) load resistance of SPS wall as affected by a range of window and door opening conditions
- SPS performance as it forms an unconventional deep, slender header as the exterior ply extends upwards from the wall opening to the top of the floor truss or raised roof truss heel
- Floor and roof truss connection and bearing support performance of an SPS wall.

For assessment of these structural attributes of building wall systems, the American Society for Testing and Materials (ASTM) E72 standard (and its series of testing protocols) has been used in a variety of applications and evaluation strategies. For example, it has been used to determine structural properties for engineering design of building walls. However, ASTM E72 does not recommend safety factors. These must be chosen to be consistent with precedents for other similar materials and applications in the code, and in consideration of the variability of performance (both in laboratory index testing and also due to variations in construction quality or end-use conditions). The standard and its testing protocols have also been used to develop adjustment factors for analytically indeterminate wall systems, which are then used to modify or calibrate

accepted design theory or analytical models to match the observed system performance. Finally, the ASTM E72 test method also has been used to directly compare performance with code-recognized wall constructions in an equivalency-based wall evaluation approach. In this approach, a representative sample of code-recognized wall construction must also be tested (or test criteria used based on prior testing of such "benchmark" assemblies).

Certain aspects of the ASTM E72 test methodology may be substituted for other test methodologies that may be more relevant or appropriate. For example, the ASTM E72 in-plane shear load test methodology imposes a fully restrained (against overturning) condition on a wall assembly and utilizes a rigid loading beam. Both of these idealized boundary conditions can vary significantly from what is observed in actual construction applications. Consequently, the ASTM E564 methodology is preferable for evaluation of in-plane shear behavior of an entire wall assembly because it can include overturning restraints and loading mechanisms that are similar to those experienced in the end-use construction. ASTM E564 can also be used to evaluate assemblies with variations in restraint or boundary conditions, or with openings that interact with wall in-plane shear performance including strength, stiffness, and failure modes. A similar test standard, ASTM E2126, differs from ASTM E564 primarily in the use of reverse-cyclic loading protocols rather than a monotonic or stepwise loading. This can be useful in evaluating toughness or ductility for resistance of earthquake loads and can be conservatively applied to characterize in-plane shear resistance to wind loading.

Given the above considerations and the unique nature of the SPS technology, it was decided to use the ASTM E72 testing methodology (with modifications to mimic actual end-use boundary conditions of SPS panels) as a basis for evaluation of compressive and bending load resistance. The ASTM E564 test methodology (with similar modifications) was selected to evaluate in-plane shear load resistance. For initial SPS-panel-only tests, the ASTM E72 compression tests were conducted in a universal testing machine (UTM) with appropriate rigging (see preliminary test report[3]). Subsequent tests addressed in this report used a custom-built test rig with hydraulic actuator and load cell to allow testing of SPS wall assemblies and header assemblies under actual construction conditions, including interconnected floor and roof assemblies, as realistic boundary conditions to an SPS wall.

Next, it must be considered how data from these test methods and tested specimens will be used to assess code compliance, whether with an engineering-based approach or with an equivalency (compared performance) approach. For the SPS technology, an engineering-based approach was preferred, and the test plan was developed accordingly to verify and calibrate the observed performance to relevant engineering

[3] SPS Wall Buckling Test Results and Discussion: Final Report, prepared by Home Innovation Research Labs, subcontractor to University of Minnesota under DOE Research Grant, March 25, 2020.

design theory and practice. This engineering-based approach was chosen for two reasons:

- The SPS system is different from conventional framing systems recognized in U.S. model building codes for typical wood-frame residential construction, rendering selection of comparative equivalency tests very uncertain or unreliable. The SPS technology may experience unique failure modes and behaviors. For example, it was expected from preliminary analysis and testing that the primary failure mode under gravity loading (compressive loads) will be Euler buckling because the SPS system is essentially a slender column (or plate).
- Development of an engineering methodology for the SPS technology would allow for the greatest flexibility in supporting the individual design of future buildings using the SPS technology. It would also allow for the future development of prescriptive construction provisions covering a suitable range of end-use conditions, as commonly practiced in U.S. model building codes for residential construction. The pre-engineered prescriptive construction provisions would then avoid individual engineering costs for most building construction applications.

With the structural performance attributes of interest identified along with the basic test methods to support an engineering-based evaluation methodology, the next step was to identify the specific testing to be conducted. The sequence of testing was also considered important to allow for real-time decision making toward optimizing SPS construction details while at the same time obtaining data needed to support development and confirmation of a code-compliant engineering methodology for the SPS construction technology.

This process of developing an evaluation strategy for the SPS technology was informed by similar past projects to develop, verify, or calibrate engineering procedures for other innovative wall systems like insulating concrete forms (ICFs), and more conventional wall systems like cold-formed steel framing. These procedures were developed into prescriptive construction methods, with sponsorship of the U.S. Department of Housing and Urban Development (HUD) and interested industry organizations, by the National Association of Home Builders (NAHB) Research Center. The process to develop an evaluation strategy was also informed by relatively recent innovations to engineered wood framing, such as the development of the perforated shear wall design methodology and its inclusion in the American Wood Council's (AWC) *Special Design Provisions for Wind and Seismic* (SDPWS),[4] including its further adaptation to prescriptive wood framing wall bracing methods resulting in the now popular "continuous structural sheathing" bracing method in the International Residential Code

[4] American Wood Council. 2021. *Special Design Provisions for Wind and Seismic*. https://awc.org/publications/2021-sdpws/

(IRC) based on HUD and industry research.[5,6,7] All of these structural technologies are now incorporated into IRC, which is used across the United States for home construction.

2.3 Preliminary SPS Engineering Analysis

With the evaluation strategy framework established above, including applicable test methods and performance attributes to measure, it became necessary and prudent to define appropriate engineering theory or methodologies to be used as a "hypothesis" for evaluating its ability to predict testing outcomes for each of the structural attributes of interest. This exercise also informed the development of a specific test plan and test specimen configurations needed to develop, refine, and confirm proposed engineering methodologies.

2.3.1 Engineering Analysis (Prediction) of Preliminary Axial Compressive Load Tests of SPS Specimens

The 2018 National Design Specification (NDS) for Wood Construction[8] was reviewed for applicability of its column design procedures, particularly column buckling and interaction equations, to the SPS system. Based on that review, and in addition to a review of various sources or related technical references in the NDS commentary, it was determined that NDS Section 15.4.1 and use of the general equations for wood columns with side loads and eccentricity provided an appropriate means of analyzing the SPS panel as slender plate columns, even though in some cases the SPS system would have L_e/d ratios greater than the maximum limit of 50 in NDS Section C3.7.1.4. The equations of interest are Equation 15.4-1 and 15.4-2, which are used together as a design check to control column buckling and determine column design axial load capacity under compressive load with or without eccentricity and side loads. It also allows for any initial bowing of the SPS panel (additional axial load eccentricity) to be considered.

Because NDS Equations 15.4-1 and 15.4-2 include terms that are not relevant to the particular analysis of the SPS system (e.g., bending loads on the SPS system will only be in the weak-axis direction of the thin panel/column) those equations are simplified to the following:

[5] NAHB Research Center. 2001. *Wood Shear Walls with Corners*. Washington, DC: U.S. Department of Housing and Urban Development, Office of Policy Development and Research. https://www.huduser.gov/publications/pdf/shearwalls.pdf

[6] Crandell, J.H. 2007. "The Story Behind IRC Wall Bracing Provisions." *Wood Design Focus*, Summer 2007, pp.3-14. https://forestprod.org/page/WDF

[7] Crandell, J.H. and Z. Martin. 2009. "The Story Behind the 2009 IRC Wall Bracing Provisions (Part 2: New Wind Bracing Requirements)", *Wood Design Focus*, Spring 2009, pp.3-10, https://forestprod.org/page/WDF

[8] NDS. 2018. "National Design Specification for Wood Construction with Commentary, 2018 Edition." Leesburg, VA: American Wood Council.

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b2} + f_c(6e_2/d_2)\{1 + 0.234(f_c/F_{cE2})\}}{F_{b2}'\{1 - (f_c/F_{cE2})\}} \leq 1.0$$

and

$$\frac{f_c}{F_{cE2}} < 1.0$$

$$F_{cE2} = \frac{0.822E_{min}'}{(l_{e2}/d_2)^2}$$

where,

f_c = compression stress parallel to column/panel length due to axial load

f_{b2} = flatwise bending stress due to side loads on the wide face of the column/panel only (i.e., inward positive or outward negative wind loading)

F_c' = adjusted compression design value parallel to grain that would be permitted if axial compressive stress only existed, determined in accordance with 2.3 and 3.7 of NDS (use F_c average ultimate unadjusted where equation is used to predict tested ultimate axial load capacity)

F_{b2}' = adjusted flatwise bending design value that would be permitted if flatwise bending stress only existed, determined in accordance with 2.3 and 3.3.3 of NDS (use F_{b2} average ultimate unadjusted where equation is used to predict tested ultimate axial load capacity)

e_2 = eccentricity, measured perpendicular to wide face of panel/column from centerline of column to centerline of axial load

F_{cE2} = Euler buckling allowable axial stress for uniaxial flatwise bending

l_{e2} = effective buckling length in column weak axis bending direction = $K_e l_2$

d_2 = thickness of column/panel (thinner cross section dimension)

K_e = buckling length coefficient (see Appendix G of NDS[9])

L_2 = column length between lateral supports for column weak axis bending direction

[9] NDS. 2018. "National Design Specification for Wood Construction with Commentary, 2018 Edition." Leesburg, VA: American Wood Council.

E_{min}' = adjusted modulus of elasticity (5th percentile divided by 1.66 safety factor) (see Appendix F of NDS[10]): $E_{0.05} = E_{avg}(1\text{-}1.645COV_E)$; $E_{min}' = E_{0.05}/1.66$ – (use E_{avg} unadjusted where predicting average ultimate capacity for comparison to test data)

Both of the above equations must be satisfied. The first addresses various load effects and stress interactions between compression loading, bending load (weak axis of column), and eccentricity of axial load adding to weak axis bending or buckling of column. The second addresses conventional Euler buckling under compressive load. Again, these two equations have been simplified by removing irrelevant terms (e.g., those that reduce to zero) as applied to the SPS panel/column condition.

It is evident from the above equations that various parameters must be defined to determine allowable load capacity. In addition, if the equations are to be used to predict tested ultimate load capacity or calibrate to test results, then material allowable strength parameters must be adjusted to their original short-term loading ultimate capacity basis (i.e., safety factoring and load-duration adjustment factors removed). In general, such ultimate strength data for OSB panels is not completely available from OSB panel manufacturers. However, for the 1-1/8" thick OSB material used in this project, the manufacturer provided typically available material property data from six years of production sampling, which is summarized in Table 1.

Table 1. Material Property Data

Variable	Value (psi)	Coefficient of Variation (COV)
E (Average modulus of elasticity lengthwise bending)	800,080	0.064
E (average MOE cross panel bending, perpendicular to panel length)	315,300	0.079
$F_{b\text{-}ult,para}$ (modulus of rupture [MOR], bending stress in parallel to panel length direction)	4,939	0.089
$F_{b\text{-}ult,perp}$ (MOR, bending stress in perpendicular to panel length direction)	1,985	0.099

To use the NDS column buckling equations, however, the material compressive stress resistance property is also needed, and this property was not monitored by the manufacturer because it is not a requirement and the need is somewhat unique to the SPS wall system application. This signaled the need to quantify OSB material properties

[10] NDS. 2018. "National Design Specification for Wood Construction with Commentary, 2018 Edition." Leesburg, VA: American Wood Council.

for the SPS technology. Therefore, such material property testing was included in the test program for this project.

Despite an incomplete set of material properties, an application of the NDS column buckling equations was compared to prior SPS panel tests to initially assess the efficacy of those equations in predicting the results. To enable this preliminary evaluation, some assumptions were necessary along with a “proxy” value for the compression stress value for the OSB material. These assumptions and material property data needs became a significant part of the focus of the test plan for this project to enable an informed and objective use of the NDS column buckling equations for designing SPS walls (as confirmed in the final stages of this project).

Using the simplified NDS column buckling equations and the ultimate material property data reported above, including the average MOE, the equations predicted the average ultimate axial load capacity of the preliminary 1-ply SPS panel tests very consistently. This included use of a buckling length factor, K_e, of about 0.8 which is reasonably consistent with the end restraint provided by the test rigging and panel bearing on square-cut ends. However, the assumptions necessary to make this initial assessment relied on judgments that proved to be too subjective to justify an objective use of the NDS buckling equations, particularly for code-compliant design purposes (or development of prescriptive construction provisions). This concern became a particularly important focus of this project, among others such as developing a means to predict the in-plane (racking) shear resistance of SPS walls.

The preliminary test data used for the above preliminary engineering analysis and predictions are summarized in Tables 2 through 4. The 2-ply SPS panels were laminated with mechanical fasteners in accordance with Figure 7. This pattern was carried out through all testing in this project. All tests were of 48-inch-wide panels using the ASTM E72 test method as shown in Figure 8. The 48-inch wide specimens of various tested heights shown in Tables 2 through 4 were built from material cut from the original 8’ wide x 24’ long panels supplied by the manufacturer in either the length (strong) or cross-panel width (weak) directions. Table 4 can be used to compare with the 8’ height specimen in Table 2 which was tested with the stronger panel bending direction oriented vertically.

Table 2. 1-Ply Maximum Loads in Compression (Buckling) for 48” Wide Panel Oriented With the Original OSB Panel Length in the Vertical Direction

1-1/8-in OSB, All Specimens 4-ft Width						
1-Ply (Vertical Orientation)	**Specimen ID Panel Height, ft**	**A**	**B**	**C**	**Avg Load (lbs) Per Panel**	**% Variation Among 3 Replicates, Against Average**
Maximum Load in Buckling, lbs	4	38,913	41,406	41,544	40,621	4.2%
	6	18,264#	17,354#	17,256	17,625	3.6%
	8	7,255	7,402	7,964	7,540	5.6%
	9	6,447#	6,412#	6,433#	6,431	0.3%
	10*	4,802√	4,991√	6,809√	5,534	23.0%

During testing, the panel bowed opposite to the intended direction.
√ Specimen contained a noticeable bow; an 8-ft level showed a slight longitudinal bow.
* Constructed using panels from the second shipment of panels, vertical orientation.

Table 3. 2-Ply Maximum Loads in Compression (Buckling) for 48” Wide Panels Oriented in Cross Lamination

-1/8-in OSB, All Specimens 4-ft Width						
2-Ply (Cross-Laminated)	**Specimen ID Panel Height, ft**	**A—D**	**B—E**	**C—F**	**Avg Load (lbs) Per Panel**	**% Variation Among 3 Replicates, Against Average**
Maximum Load in Buckling, lbs	4	71,756	70,255	79,902	73,971	8.0%
	6	40,710	40,108	39,261	40,026	1.9%
	8^	15,589#	16,662#	15,866#	16,039	3.9%
	9*	19,037	21,083	18,220	19,447	8.4%
	10*	15,316#√	15,836#√	14,916#√	15,356	3.1%

^ All three 2-ply 8-ft. specimens were installed in the UTM “backwards,” with fastener heads on the concave side of the bow; also, a rotational hinge developed in the test rigging exterior to the panel which likely increased the effective buckling length of the panel, resulting in a lower tested axial load capacity. This concern was resolved in other tests.
# During testing, the panel bowed opposite to the intended direction.
√ Specimen contained a noticeable bow; an 8-ft level showed a slight longitudinal bow.
* Constructed using panels from the second shipment of panels, vertical orientation.

Table 4. 1-Ply Maximum Loads in Compression (Buckling) for 48” Wide Panel Oriented With Original OSB Panel Length in the Horizontal Direction (Compressive Stress in the Weaker Cross-Panel OSB Strand Orientation)

1-1/8 in OSB, All Specimens 4-ft Width						
1-Ply (Horizontal Orientation)	**Specimen ID Panel Height, ft**	**A**	**B**	**C**	**Avg Load (lbs) Per Panel**	**% Variation Among 3 Replicates, Against Average**
Maximum Load in Buckling, lbs	8	6,118	5,454	5,518	5,697	7.4%

1. GRK R4 #9 x 2" screws at 1'-0" o.c. horizontally and 2'-0" o.c. vertically. The screw head should be firmly engaged with the panel surface.
2. 0.113" x 2" PASLODE ring shank gun-nail at 0'-8" o.c. vertically between the GRK Screws. The nail heads should be flush (or nearly so) with the panel surface.
3. Use this pattern through the field of the panel.
4. At the edge, simply bring the fastener in one inch (reduces the spacing at the perimeter by 1").

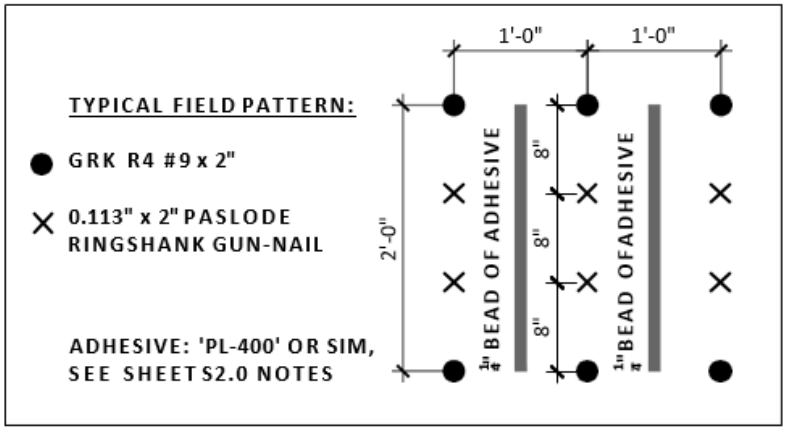

Figure 7. SPS 2-ply panel mechanical fastener schedule for panel lamination (no glue used)

Figure 8. ASTM E72 axial load compression buckling test setup using Home Innovation Research Labs' UTM

The above analysis and comparison to preliminary axial load test data of the panels only mainly confirmed the applicability of the NDS column buckling equations for use with the SPS wall construction. It also confirmed the need to better understand the column buckling length coefficient, K_e, and its applicability to the SPS technology, particularly with regard to the panel end fixity provided by its attachment to floor and roof truss assemblies. This topic was briefly discussed earlier in this section and will be more completely developed in relation to the stiffening effect of SPS connections to roof and floor assembly in the testing and analysis sections of this report (see Sections 2.4.2, 2.4.4, 3.1.3, and 3.1.5). Also, it is important to better characterize the OSB panel material properties actually used in the tests (rather than based on prior manufacturer production data or proxy data from alternative sources). Thus, the planned testing addressed later in this report was developed to include tests which incorporated these assembly effects as well as material property testing.

These preliminary tests and engineering analyses also confirmed that the SPS system had substantial compressive load bearing capability in supporting roof and floor spans. Without a complete engineering approach, however, it would not be possible to determine appropriate use of the SPS technology in relation to widely varying building

design loads (e.g., dead, live, snow, and wind loads), floor and roof spans, effect of window and door openings on gravity load-bearing capacity and also in-plane (racking) shear resistance to design lateral loads from wind or seismic activity. Therefore, this test plan aimed to resolve this gap as the main barrier to SPS adoption by the building industry, whether as a case-by-case engineered system or in the form of a prescriptive construction method with limitations defined by engineering analysis.

2.3.2 SPS In-Plane Shear Resistance (Lateral Force Resisting System) Design

At the start of this project, no tests had been conducted of SPS shear walls (with or without wall openings and various means of addressing over-turning restraint). Such testing was determined as a significant need to support the development of a means of predicting in-plane shear resistance and designing for the in-plane (racking) shear forces caused by wind or earthquake lateral loads on a building structure using the SPS technology.

The code-recognized wall bracing methods most relevant to the SPS wall system are:

- Perforated Shear Wall (PSW) method (2015 SDPWS Section 4.3.3.5)[11]
- Continuous Sheathing Wood Structural Panel (CS-WSP) bracing method (2018 IRC Table R602.10.4, Section R602.10.4.2, and Section R602.10.7).

Both of these bracing methods are based on an empirical curve fitting of tested in-plane shear strength and stiffness using a parameter, r, known as the opening area ratio, as the key independent variable. The opening area ratio is a measure of the area of openings in a given wall assembly and it decreases with increasing opening area as a proportion of the overall wall surface area.

While based on the same empirical approach, the two code-recognized wall bracing applications listed above have different constraints, or limits of use, which result in different performance characteristics. For example, the PSW method requires designed end restraints (e.g., hold-down brackets) to fully react theoretical maximum overturning uplift forces at the terminal ends of the wall line. In its standardization in SDPWS, it also comes with a 2:1 aspect ratio limit for wall segments, which can be extended to a 3.5:1 aspect ratio (narrower segments) with a penalty in shear capacity. This is assessed by multiplying the actual segment length by 2b/h when determining shear capacity (where b is the width and h is the height of a wall segment with full-height sheathing and no opening perforations).[12] Conversely, the CS-WSP bracing method was an adaptation of the PSW method to allow its use on conventional framing of homes and other similar wood frame structures. Instead of using hold-down devices for end restraints to resist

[11] American Wood Council. 2015. *Special Design Provisions for Wind and Seismic* (2015 Edition). https://awc.org/publications/2015-sdpws/

[12] NDS. 2018. “National Design Specification for Wood Construction with Commentary, 2018 Edition.” Leesburg, VA: American Wood Council.

overturning uplift forces, it relies on the corner geometry of the structure to provide "partial" end restraints representative of actual conventional construction practice. Hence it requires a minimum 2' wide corner return panel at each terminal end of a wall line. It also extends the predictive relationship to include walls with narrow segments having aspect ratios of up to 4:1 (e.g., an 8' tall wall with 2' wide wall segments between openings or between a corner and an opening).

The SPS wall system, including its application and design and construction detailing, most closely aligns with the continuous sheathing method in the IRC. Thus, testing of SPS walls for in-plane shear resistance closely follows the testing and criteria used in the development of the continuous sheathing method and its application in the IRC. The testing requirements include a series of wall specimens covering a range of opening area ratios and wall segment aspect ratios, as well as a specimen to evaluate the effect of partial over-turning restraint provided by the SPS corner framing detail. Furthermore, the test rigging and wall specimens must incorporate a portion of floor/roof system along the base and top of the wall test specimens to provide a boundary condition (and restraint) representative of actual end use.

Given the uniqueness of the SPS wall system, the in-plane shear test data generated in this project was fitted to the test results for the SPS system rather than attempting to use the empirical relationship that was derived specifically for sheathed wood frame construction. Furthermore, the criteria applied to the data (including stiffness limits and safety factors) should be aligned exactly with that used in the development of the IRC's CS-WSP bracing method.

With the above empirically derived shear wall capacity predictive relationship defined for SPS, it would then be possible to conduct a multitude of designs for various SPS building configurations, or to derive prescriptive bracing provisions for SPS that would simply "plug into" the IRC's prescriptive construction framework. It can also be used as a basis for lateral-force-resisting system design of individual building projects using the SPS technology.

2.3.3 SPS Header System Design

To support a wood beam design methodology (e.g., Section 3.3 of NDS) for SPS headers, the following material properties would be needed and are not commonly monitored or available for OSB panel products:

- $F_{b\text{-}ult,para\text{-}edge}$ (MOR for bending edgewise with stress parallel to panel length direction)
- $F_{b\text{-}ult,perp\text{-}edge}$ (MOR for bending edgewise with stress perpendicular to panel length direction)
- $F_{v\text{-}ult}$ (edgewise shear through thickness for panel span in length direction)

- $F_{v\text{-}ult}$ (edgewise shear through thickness for panel span in cross panel direction)

In the absence of such material property data, and to avoid requiring such data for application of the SPS construction technology at this stage in its development and implementation, it was decided to base the SPS header designs directly on tested results for conditions tested (e.g., header detailing including tested span limit and limits on location and splicing of any joints that may occur in one or the other ply of a 2-ply SPS assembly). The goal was to enable use of at least a 6' wide window or door opening, provided the testing indicated adequate capacity to support reasonable floor and roof spans. Wider wall openings would require a designed solution, possibly including a strongback within floor trusses at the bearing location on an SPS wall.

2.3.4 SPS to Floor or Roof System Connections

The SPS to floor or roof system connection relies on a combination of a small bearing width created by a bearing shelf the thickness of the inner SPS ply (i.e., 1-1/8" wide) as well as connections from the floor or roof truss end member to the outer SPS ply using a specified schedule of self-drilling wood screws. There are no recognized procedures to design such a connection arrangement with interacting bearing and fastening elements working together to transfer floor or roof truss end reactions. Therefore, a design limit to this connection was planned to be determined directly from testing addressed later (as part of the header testing results where the common failure mode observed was the floor or roof to SPS wall connection, not the SPS header itself).

2.3.5 Other Structural Considerations

Building codes and design standards require that combined load effects be addressed. The primary concern with combined out-of-plane wall loads (e.g., wind pressure) and axial load can be addressed using the NDS column buckling equations mentioned earlier. However, there are two additional combined load interaction conditions that should be considered:

- *Combined in-plane shear and gravity loading.* The SPS wall system is subject to buckling failure modes under gravity loading (Euler buckling) and in-plane shear loads also cause compression loading in reacting overturning forces.
- *Combined in-plane shear and wind uplift loading.* The SPS wall system also has a combined load path for in-plane shear and uplift caused by wind loading on a building. Because the IRC's wood frame wall bracing methods were not tested for combined loading, their use is prescriptively limited to 100 pounds per linear foot (plf) allowable stress design (ASD) wind uplift load at the top of the wall (IRC Section R602.3.5), above which a separate wind uplift load path must be provided. However, the 2015 SDPWS Section 4.4 includes provisions for engineering shear walls where sheathing and sheathing connections are used to resist combined in-plane shear and wind uplift forces. The approach relies on

additional sheathing fasteners (beyond those required for the shear wall assembly) to resist the wind uplift forces. This has relied on calculation per the NDS as verified by full-scale testing.

Realistically addressing the above combined load interaction effects would require special testing methods and equipment. Even so, they are best understood through carefully designed whole building tests. Such testing was considered beyond the scope of this project. However, these combined load interaction effects should be conservatively addressed through the use of test data and engineering methods investigated in this project together with appropriate analysis and judgment.

A final combined load interaction effect of possible concern includes floor bending stiffness interaction with SPS wall axial load resistance. The ends of floor or roof trusses will slightly rotate under floor or roof loading creating an end moment (and rotation) that effectively induces some additional amount of bowing or bending of the SPS panel based on the relative stiffness of the SPS panel and the attached floor or roof system. Stiffer floor and roof truss systems would tend to reduce the influence of this potential interaction effect. This concern may be adequately addressed by imposing an additional eccentricity to the engineering analysis using the NDS column buckling equations mentioned earlier.

2.4 Summary of Test Plan and Objectives

In the previous section, a case was made for various testing needs to fill gaps in the ability to engineer the SPS system as needed for due diligence, code compliance, cost-effectiveness, good practice, and development of efficient and reliable prescriptive construction solutions. This section now provides a summary of the test plan developed for this project with references to other appendices for more detailed test information and test results. The purpose of this test plan summary is to discuss the objectives for each part of the test plan aimed at addressing key structural performance testing needs to support the development of appropriate engineering analysis methods or prescriptive solutions. Opportunities to further expand on this work and further refine the SPS application with additional future testing and development of engineering methods are also mentioned as a means to expand on the scope, objectives, and analyses conducted for this project.

The test plan was ordered in a sequence to allow real-time decisions regarding optimal SPS construction detailing options while at the same time gaining data to support the objective of verifying design methodologies or defining prescriptive design limits for needed structural performance considerations.

2.4.1 SPS Headers for Window and Door Openings

SPS header tests were conducted to quantify structural performance and to determine the impact of SPS 2-ply panel orientation. The test specimens and results are described in Appendix A.

Determining a preferred 2-ply panel orientation (cross-laminated vs. parallel lamination) was considered an important initial decision because it would impact subsequent tests. For example, it was found in the preliminary engineering assessment (see previous section) that orienting both plies with the original 1-1/8" x 8' x 24' OSB panel length (strong direction) in the vertical direction would likely increase the vertical axial compressive load capacity (buckling resistance) of the SPS wall system. However, the parallel lamination of panels would place the original OSB panel weak direction (width) of both OSB plies in the direction of the SPS header's extreme fiber bending stress, resulting in potentially limited SPS header span capability or limited spans for supported floor and roof assemblies. In addition, there was sufficient funding to test only one series of in-plane shear test specimens with one 2-ply panel lamination approach (parallel or cross-laminated). So, a decision was needed on the preferred 2-ply panel lamination approach prior to initiating the in-plane shear and subsequent bending and compressive tests. In this manner, SPS construction detailing decisions were resolved as the testing proceeded to characterize structural performance.

SPS header specimens included 4' and 6' clear spans. Some tests also included splices in the inner or outer OSB plies located at header ends (location of maximum shear stress) and mid-span (location of maximum bending stress). The depth of the SPS headers in these tests intentionally addressed the most conservative (smallest) SPS header depth that would occur at the top of an SPS wall. Thus, the results of the limited testing also could be conservatively applied to SPS headers in other locations (such as below a second-story floor system).

Future testing of deeper SPS header conditions (such as would occur at headers below a second-story floor) should be considered to expand the scope of testing and associated prescriptive design limitations based on this testing project. Furthermore, more extensive SPS header assembly testing together with documentation of OSB material property for edgewise bending and shear could lead to an engineering design methodology for the SPS headers, covering a wider range of end-use conditions.

2.4.2 Floor and Roof Truss Connections to SPS Wall

The connection of floor and roof systems to the SPS wall assembly is a key structural component of the overall SPS construction technology. The SPS header tests described above also yielded data on the structural performance of floor and roof member connections to the SPS wall. Thus, the SPS header test specimen descriptions and test data included in Appendix A are also applicable to evaluation of the floor- or roof-to-wall connections where the connection and not the SPS header was the failure mode. While

the tests were conducted with beams emulating floor or roof truss end connections, the results are applicable to connections with equivalent fastener capacity, member depth, and wood species (density).

2.4.3 In-Plane (Racking) Shear Resistance of SPS Wall Assemblies

In-plane racking shear test specimens and results are described in Appendix B. The series of tests included SPS wall assemblies with full over-turning restraint and varying amounts of wall openings and full-height wall segment aspect ratios adjacent to the openings. It also included tests to determine the performance impact of relying on the partial over-turning restraint provided by a minimum 2' corner return wall.

The test sequence was designed to be consistent with that used to develop the continuous sheathing wall bracing method in the IRC.[13] However, in this case, the test results were used to fit an empirical relationship directly to the SPS wall's observed performance, just as was originally performed for the continuously sheathed wood structural panel wall bracing method recognized in the IRC, and also the perforated shear wall design method recognized in the AWC/SDPWS design standard. All of the tested SPS wall assemblies used a cross-laminated 2-ply approach for constructing the SPS wall panels, as determined to be the preferred approach after completion of the SPS header tests previously described.

Future racking tests should consider parallel lamination of the SPS wall panels as an alternative methodology. Additionally, to the extent that taller than 8' floor-to-ceiling height stories are considered desirable for the SPS construction method, additional tests should be run with SPS wall floor-to-ceiling heights greater than 8'. Consequently, this testing project limits the SPS technology and the derived lateral force resisting system design methodology to maximum floor-to-ceiling heights of 8' using cross-laminated OSB plies for the SPS wall construction.

2.4.4 1-Ply and 2-Ply SPS Bending Stiffness (EI) and Effect of Partial Composite Action and End-Moment Fixity

Bending tests were conducted of individual OSB panels in the panel length (strong) direction and in the original OSB panel width (weak) direction. Bending tests were also conducted for cross-laminated 2-ply SPS panels in both the inward and outward bending directions. The test specimen details and test results are included in Appendix C.

The purpose of these tests was to determine the bending stiffness of individual panels and the 2-ply cross laminated panels to understand the degree of composite action realized by the mechanical fastener pattern used to laminate the two plies. In addition, bending tests were conducted of the SPS wall panels with connection to floor/roof

[13] NAHB. 2001. *Wood Shear Walls with Corners*. Washington, DC: U.S. Department of Housing and Urban Development Office of Policy Development and Research. https://www.huduser.gov/publications/pdf/shearwalls.pdf

beams to determine the degree of end fixity provided by connections of the SPS 2-ply panels to floor and roof systems. This test phase was included to provide a basis for deriving appropriate parameters for an objective and accurate use of the NDS column buckling equations discussed earlier.

2.4.5 SPS Wall Assembly Axial Load (Compression Buckling) Tests

The preliminary axial load tests addressed earlier were conducted without the restraining effects of floor and roof systems attached to the SPS wall panels. In the final axial load tests conducted for this project, the axial load was applied to the SPS wall panels through floor/roof truss beams attached to the panels. In this manner, the axial loads were applied as point loads to the wall panels (rather than a uniform compressive load as done in the preliminary axial load tests). Also, the connections to the floor/roof beams provided a level of restraint to the SPS panels as would be expected to minimally occur in actual construction. The specimen construction details and test results are shown in Appendix D.

2.4.6 OSB Material Properties

Small specimen material property data (sampled from the actual panel materials used in the testing program) were conducted last to support a predictive comparison and verification of the NDS column buckling equations for use with the SPS wall technology in resisting axial compression (gravity) loads and out-of-plane (e.g., wind) bending loads, individually and in combination. The test specimen descriptions and test data are included in Appendix E.

The material property tests also serve to “fingerprint” the OSB material properties in relation to the lateral force resisting system design methodology derived from in-plane shear test data and the header test results, which rely on direct application of the test data to determine prescriptive design load and span limitations. The test data and derived prescriptive results are dependent on material that has equivalent or better material properties. Thus, the fingerprinting of the OSB material properties used in the testing allows appropriate materials to be specified for use with the derived prescriptive solutions.

3 Structural Testing Results and Application

3.1 Engineering Analysis of Test Data

3.1.1 SPS Headers for Window and Door Openings

The SPS header acts as a deep slender beam with lateral-torsional buckling restraint provided by connection to horizontal floor and roof systems located above the header. An example construction drawing of a paired SPS header test specimen is shown in Figure 9. The test data serving as the basis of this engineering analysis of the SPS header performance are described in Appendix B, including specimen construction details, photographs, and test results with failure modes described. The 2-ply lamination of the SPS walls and headers as shown in Figure 5 used the field fastener pattern shown earlier in Figure 7 with added GRK R4 #9x2" screws at 6" o.c. at the perimeter of the wall opening below the header. This fastening approach was also applied to later in-plane shear wall tests where wall openings were included.

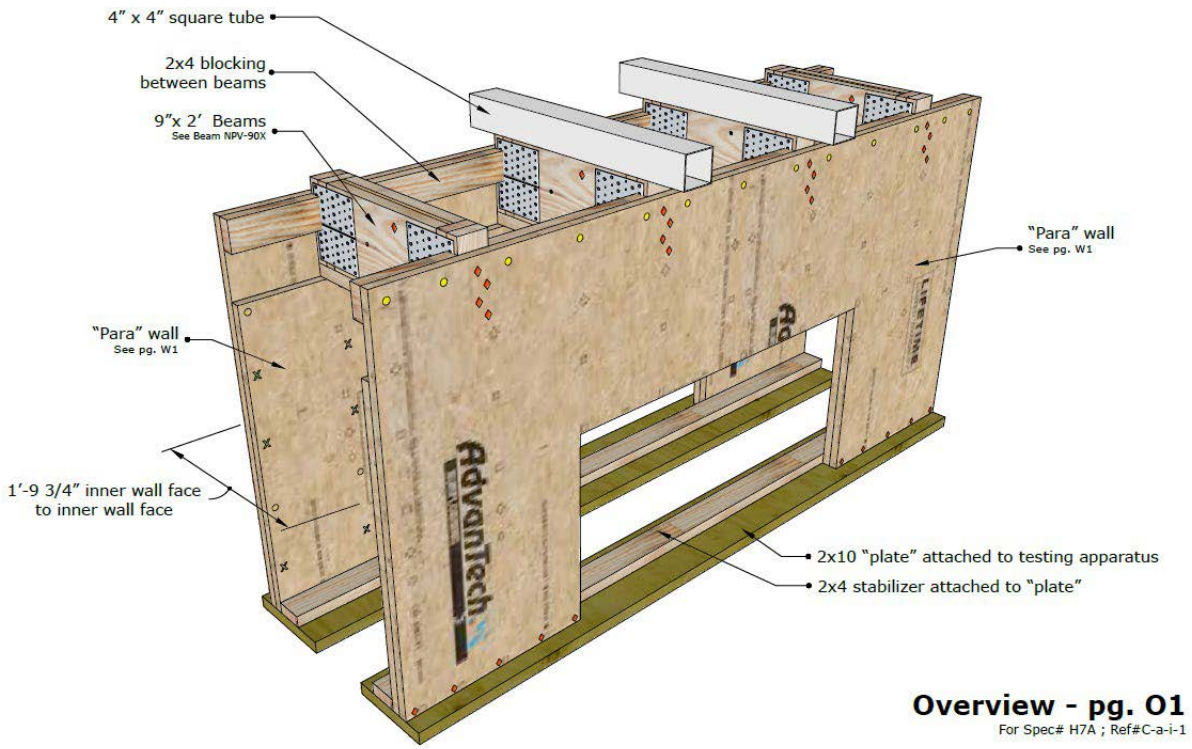

Figure 9. Example of paired SPS header test specimen construction including floor/roof truss connections to outer ply and bearing on inner ply of 1-1/8" thick OSB panels

The test results and engineering properties derived from those results are summarized in Table 5. It is important to note that the only failures associated with the tested SPS headers occurred in specimens with splices: (1) a 4' span SPS header specimen with

an under-reinforced splice joint at the end of the header (Specimen ID 3) which was corrected by adding more splice joint fasteners and retested as Specimen ID 4 without failure of the splice and (2) Specimen ID 7 and 8 for 6’ span SPS headers with center span splices in one of the SPS header plies. For all other specimens, the failure mode was not the SPS header. Thus, the maximum loads for the SPS headers are somewhat less than the ultimate capacity of the SPS 2-ply headers, with the exception of Specimens ID 7 and 8 with center-span splices in one of the plies, and Specimen ID 3 with an under-reinforced splice at the end of the SPS header (which was addressed in Specimen ID 4 with an improved splice connection schedule as shown in Appendix B construction details). The maximum loads where the SPS headers did not fail and the failure mode was associated with the beam bearing/connection failure will be evaluated in the next section.

Table 5. Summary of SPS Header Test Results and Analysis[a]

Test Specimen ID (paired SPS headers)	Max Total Applied Load[b] (lbs)	Max Moment[c] per SPS Header (in-lbs)	Max Shear[d] per SPS Header (lbs)	SPS Header Construction (ply orientation and splice condition)[e]	Failure Mode
4' Span SPS Headers					
1	50,017	150,048	12,504	Parallel plies No splice joints	Connection/bearing, not header
2	41,952	125,856	10,488	Perpendicular plies No splice joints	Connection/bearing, not header
3	33,558	n/a	n/a	Parallel plies Splices joints at ends	End splice joint slip (added splice fasteners in Test ID 4)
4	43,130	129,390	10,783	Perpendicular plies Splice joints at ends	Connection/bearing, not header
6' Span SPS Headers					
5	34,541	207,246	8,635	Parallel plies No splice joints	Unknown, nonobvious to test lab except loud "pop"
6	37,610	225,660	9,403	Perpendicular plies No splice joints	Connection/bearing, not header
7	32,457	194,742	8,114	Parallel plies Splice joints center span	Bending rupture of inner ply at center span on header with splice in outer ply
8	40,029	240,174	10,007	Perpendicular plies Splice joints center span	Bending rupture of outer ply at center span on header with splice in inner ply

a. SPS header deflections were small due to significant apparent stiffness of the SPS header construction (see Appendix B); therefore, deflections are not evaluated as a potential design limit state for the 4' and 6' span SPS header constructions as tested in this project.
b. Max total applied load is the UTM load applied to two beam members (represented floor/roof trusses) spanning between the paired headers and spaced at 24"oc resulting in a two-point load on the SPS header span (located at 1/4 points for the 4' SPS header specimens and at 1/3 points for the 6' SPS header specimens). Thus, each point load was one-fourth of the total applied load.
c. Maximum moment per SPS header is calculated as (1/4 total applied load, lbs) x (distance, inches, of the point loads from ends of the header spans) based on beam diagram for two-point loading where each point load is one-fourth of the total applied load.
d. Maximum shear load per SPS header is equal to the magnitude of the individual two-point loads applied to each paired SPS header (e.g., one-fourth of total applied load) based on the beam diagram for the headers.
e. "Parallel plies" refers to both plies having the original OSB panel length (strong) direction oriented in the vertical direction of the test specimens, meaning that both plies are oriented with the original OSB panel's weak (cross-panel width) direction aligned with header extreme fiber tension stress at bottom edge of headers. "Perpendicular plies" indicates that the inner ply was oriented such that the original OSB panel length (strong) direction was horizontal such that the two plies of the header were cross-laminated.

First, from Test ID 7 and 8 it became clear that orienting both plies in the "parallel" condition (placing the bending extreme fiber tension stresses in the OSB panel's weaker

stress direction for both panels) had a weakening effect on the SPS header capacity. Therefore, the remainder of this analysis focuses on results for the “perpendicular plies” condition, which was selected as the preferred lamination approach for 2-ply SPS wall assembly construction for the remainder of the project.

For engineering design purposes, the 4’ header span Specimen ID 2 provides a basis for establishing the maximum shear load strength that the perpendicular ply (cross-laminated) SPS header can resist with or without end splices detailed in accordance with Specimen ID 4 (which actually exhibited greater strength than Specimen ID 2). The 6’ header span Specimen ID 6 provides the basis for establishing the maximum bending moment strength that the perpendicular ply (cross-laminated) SPS header can resist with or without center splices detailed in accordance with Specimen ID 8 (which actually exhibited greater strength than Specimen ID 6). With the exception of Specimen ID 8, none of the failures were associated with the SPS header and, thus, these tests do not necessarily represent the maximum shear or bending strength of the cross-laminated 2-ply SPS header constructions. However, they provide a conservative basis for SPS header design purposes. These tests did not explicitly address truss end rotation due to bending load on a floor system, but end rotation of the test beams did occur due to some unquantified differential compression or joint slip in the pair-assembly tests (i.e., one end of beam compressed/slipped more than the other such that a rotation occurred). This effect should be considered in future research toward the development of an engineering-based methodology for SPS header design which was beyond the scope of this test program.

Based on the above test data and analysis, the following maximum strength values are proposed for cross-laminated 2-ply SPS headers constructed in accordance with the associated specimen construction details in Appendix B for Specimens ID 2, 4, 6, and 8:

Maximum Shear Strength = 10,488 lbs (based on minimum of Specimens ID 2 and 4)

Maximum Bending Moment = 225,660 in-lbs (based on minimum of Specimens ID 6 and 8)

3.1.2 Recommended SPS Cross-Laminated 2-Ply Header Design

Applying a typical safety factor of 3 to the above maximum values, the following design values are recommended for cross-laminated SPS headers with clear span not exceeding six feet:

ASD Shear Strength = 3,500 lbs ←

ASD Bending Moment = 75,200 in-lbs ←

These design values are applicable to cross-laminated SPS headers with not more than one splice joint in one of the plies located anywhere within the header span, provided

the spice joint fastening schedule complies with that shown in the construction drawings for Specimen ID 4 and Specimen ID 8 in Appendix B. Additionally, these design values are applicable only for OSB material properties consistent with those reported in a later section of Appendix B (with reasonable allowance for material property variance).

For cross-laminated 2-ply SPS headers with clear spans greater than 6', a designed solution is required. Such design solutions may consider use of a strongback or ribbon beam located at the ends of floor or roof trusses above the wall opening and connected to the outer ply of the SPS wall. Additionally, if the interior side of the wall is finished with 2x furring fastened to the SPS wall panels, this furring may be used to support a separate header beam spanning the wall opening beneath the floor or roof system above.

Before addressing the next structural consideration, it is worth mentioning again that there remains substantial opportunity to further economize and expand engineering approaches for SPS headers. These initial tests merely aimed to achieve a workable solution for headers spanning up to 6' wide wall openings within the available project testing budget.

3.1.3 Floor and Roof Truss Connections to SPS Wall

Because the failure mode for all of the SPS header tests (except those with splices in one of the SPS header plies) was associated with the floor/roof truss beam connections, the SPS header tests (see Table 4 and Appendix B) also served the purpose of quantifying the capacity of floor and roof truss connections to the SPS wall using the connection detail shown in Figure 10.

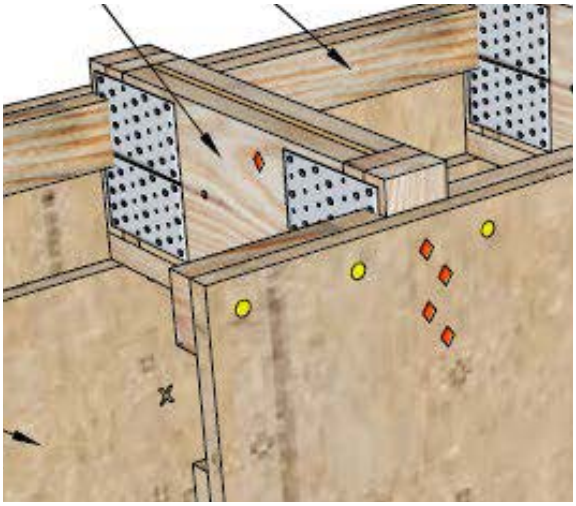

Figure 10. Truss/beam connection and bearing support condition for attachment to 2-ply SPS wall panels (i.e., 1-1/8" wide bearing shelf of inner ply in combination with four GRK ¼" x 3-1/8" washer head RSS wood screws into end of truss/beam)

Based on the header tests reported in Table 4, the selected test specimen IDs shown in Table 6 were limited in maximum capacity due to a floor/roof truss connection failure

mode. Thus, these tests served as a means to quantify an ASD design load limit for such connections as shown in Figure 10 and in test specimen details included in Appendix B.

Table 6. Summary of Floor Truss/Beam Connection Tests from Table 4

Test Specimen ID[a]	Max Total Applied Load (lbs)	Max Beam End Reaction Load (lbs)	Specimen Truss/Beam Construction[b]
1	50,017	12,504	Spruce pine fir (SPF) bottom chord
2	41,952	10,488	Southern yellow pine (SYP) bottom chord
4	43,130	10,783	SPF bottom chord
6	37,610	9,403	SPF bottom chord
	Average	**10,795**	
	COV	**0.12**	

a. Each test specimen included four connections, each resisting 1/4 of the total applied load. Thus, the maximum beam end reaction is 1/4 x (max total applied load). It represents the combined connection shear and bearing capacity of the weakest of four such beam to 2-ply SPS wall assembly attachments for each tested specimen.
b. For the connection and bearing construction details, refer to specimen construction drawings in Appendix B. In this table, only construction condition varies (i.e., test Specimen ID 2 used a 2x4 SYP bottom chord member in lieu of a lower-density and lower-compressive-strength 2x4 SPF member for all other tests). All other connection and bearing conditions were identical for all test specimens (e.g., four GRK 1/4" x 3-1/8" star drive washer head RSS wood screws from outer SPS OSB ply into face grain of a double 2x4 vertical end member of the beam representing a truss end condition).

The first observation of significance from the test data of Table 5 is that the COV of the test results are low (COV = 0.12) in comparison to single fastener wood connection tests which generally have a much greater variation (e.g., typical COV of 30% or more). This favorably low variation can be attributed to the combined small bearing area (1-1/8" shelf created by the inner ply) and the use of four GRK 1/4" x 3-1/8" washer head RSS wood screws. Consequently, it appears appropriate to use a safety factor of 3 with the average tested maximum load to derive the following design recommendations:

- Recommended Floor Truss to SPS Wall Connection Design:
  - ASD Allowable Design Floor Truss/Beam End Reaction = 3,600 lbs ←
  - The above allowable design value is based on the construction detail as shown in Figure 9 and the specimen construction drawings of Appendix B. It applies to the case where a minimum SPF 2x4 flatwise bottom chord is bearing on a minimum 1-1/8" thick OSB inner ply and the outer ply fastened to the face of a minimum two-2x4 vertical member at the end of the floor truss/beam. It applies to the specific type of screw fasteners used, or those of at least equivalent properties.

For the case of roof trusses with edgewise 2x members, the bearing area is reduced as well as the size of the end member for receiving the four wood screws from the SPS outer ply. This will tend to reduce the allowable design value provided above for the floor truss condition. The following design recommendation is made based on the smaller bearing area (without changing the mechanical fastening) using an adjustment factor of 0.7 applied to the above recommended design value. This ratio is based on capacity as determined using NDS dowel fastener shear and wood member bearing strength design requirements for the two bearing area conditions assuming bearing and connection shear forces are additive.

- Recommended Roof Truss to SPS Wall Connection Design:
  - ASD Allowable Design Floor Truss/Beam End Reaction = 2,500 lbs ←
  - The above allowable design is based on the construction detail as shown in Figure 9 and the specimen construction drawings of Appendix B with the exception that a single edge-wise raised heel roof truss vertical end member is used.

It is recommended that additional connection tests and engineering evaluations be conducted to further refine the above design recommendations. For example, the tested conditions included only mock floor trusses which provide bottom chord end bearing across the width of a 2x4 member and connection into the face grain of a double 2x4 end member. Raised heel roof trusses use 2x4 members in an edge-wise condition creating a lesser bearing area with fastening into the 1.5” thickness of the raised heel member and should be similarly tested to confirm and refine the above design recommendation. Such tests should be aimed at developing a design methodology that accounts for the interaction of the fastener shear connections and the small bearing area.

3.1.4 In-Plane (Racking) Shear Resistance of SPS Wall Assemblies

A total of eight cross-laminated SPS wall assembly in-plane shear tests were conducted using test specimens with a wall-to-ceiling height of 8’, lengths from 8’ to 14’, varying wall opening amounts and sizes, and solid full-height wall segment aspect ratios ranging from 1.0 to 4.0. The construction details and configuration of the test specimens are shown in Appendix C. The test apparatus and an example test specimen are shown in Figure 11.

Figure 11. Shear wall test apparatus and example test specimen of cross-laminated 2-ply SPS wall assembly with a window opening

The in-plane shear test data reported in Appendix C were evaluated as shown in Table 7 to determine a shear capacity prediction relationship for the SPS wall construction. As mentioned, the approach followed here is consistent with that used to develop similar relationships for the continuous-sheathed wall bracing method in the IRC and also the perforated shear wall design method in the AWC/SDPWS standard for lateral design of wood frame construction. Figure 12 indicates a consistent and predictable empirical trend between the SPS shear wall capacity and the amount of openings in a wall assembly.

Table 7. Analysis of SPS Shear Wall Test Data Following the Perforated Shear Wall Design Methodology

Wall ID	Description	Wall Height (ft)	Length (ft)	Full-height Segment Aspect Ratio	Wall Gross Area (ft^2)	Sum Length of Segments (ft)	Total Opening Area (ft^2)	Opening Area Ratio [r]	Shear Capacity Ratio [F]	Max Shear (lbs)	Unit Shear (PLF)	Unit Shear Segments Only (PLF)
Full Overturning Restraint Tests												
Avg 1B-2&3	8'x8' baseline - no openings	8	8	1:1	64	8	0	1.00	1.00	16,296	2,037	2,037
3	8'x12' one 4'x62.25" windows	8	12	2:1	96	8	20.75	0.76	0.94	22,990	1,916	2,874
4	8'x12' one 8'x82" door	8	12	4:1	96	4	54.67	0.37	0.30	7,449	621	1,862
5	8'x14' two 4'x62.25" windows	8	14	4:1	112	6	41.5	0.54	0.64	18,389	1,314	3,065
6	8'x12' one 7'x8' garage door	8	12	4:1	96	4	56	0.36	0.30	7,314	610	1,829
Partial Overturning Restraint Tests (corner return)												
2A	8'x12' with 2' corner restraint - no openings	8	12	0.67:1	96	12	0	1.00		11,081	923	923
2B	8'x12' with 2' corner restraint - no openings	8	12	0.67:1	96	12	0	1.00		10,415	868	868
											Avg	**896**
							Partial Restraint Shear Capacity Reduction Factor:					**0.44**

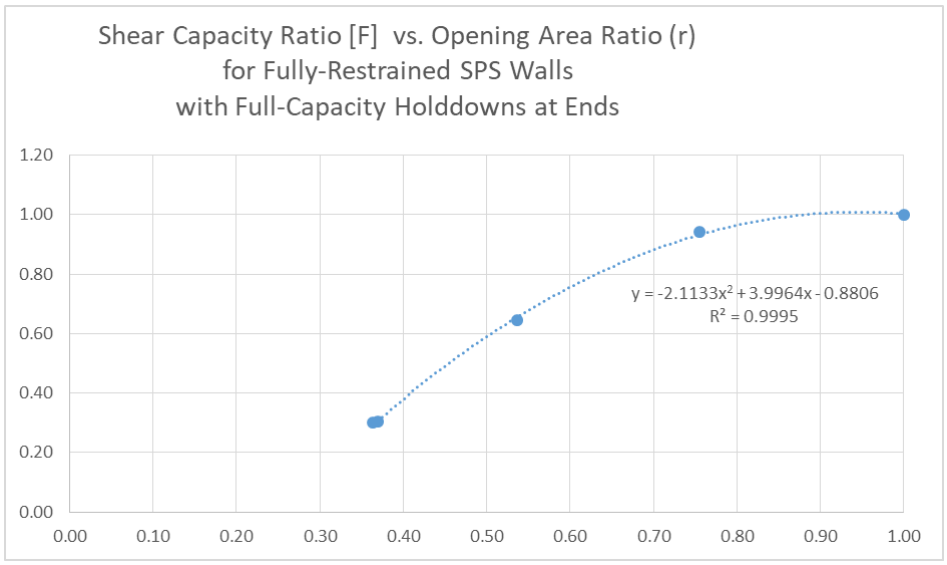

Figure 12. Trend of test data from Table 7 and Figure 10 relating shear capacity ratio (F) to the opening area ratio (r) of each SPS shear wall that was fully restrained at its ends to resist uplift overturning

Based on the analysis shown in Table 7 and Figure 12, the following design method and variables are defined for lateral force resisting system design with the SPS wall system constructed in accordance with construction detailing shown in Appendix C.

- Recommended SPS Perforated Shear Wall Design Method:

  VASD = V/(S.F.)

  V = vC x L x F x Jhd

  Where,

  VASD = allowable stress design shear resistance of SPS wall assembly

  (S.F.) = safety factor (min. 2.0 recommended for consistency with IRC and SDPWS); alternatively, V may be multiplied by a resistance factor of 0.8 for use with strength-based design provisions.

  V = SPS shear wall capacity prediction (lbs)

  vC = 2,037 plf [unit shear capacity (plf) based on average of Tests #1B-2 and 1-B3]

  L = total SPS wall length (ft)

  F = shear capacity ratio relative to wall without openings (dimensionless) ≤ 1.0

  = -2r2 + 4r - 1 (simplification of coefficients shown in Figure 12 to result in F ≤ 1.0 in all cases)

r = Opening Area Ratio = 1 / [1+ Ao / (H x ∑ Li) and must be at least 0.35

Ao = total area of openings

H = wall height

∑ Li = sum of full-height wall segment lengths (with max. 4:1 aspect ratio as determined by the floor-to-ceiling height to the along-wall length of each segment; full-height wall segments of a greater aspect ratio shall not be included in ∑ Li).

Jhd = hold-down overturning restraint adjustment factor as a ratio of the shear strength of a partially restrained to a fully restrained wall assembly (0<Jhd<1.0 for partially restrained)

= 0.44 for min. 2' corner return hold-down restraint (based on ratio of average max shear of Specimens 2A and B vs. Specimens 1B 2 and 3 as shown in Figure 11).

= 1.0 for fully restrained with rated hold-device and designed connection to SPS wall panels

Based on shear load vs. drift plots in Appendix C, the above design approach will limit shear wall drift at the ASD shear resistance (50% of shear capacity) to about 0.2 to 0.5 inches (i.e., not more than about H/240 story drift). Comparatively, the SPS perforated shear wall system is much stiffer and stronger than wood frame continuously sheathed braced walls or perforated shear walls. However, the load-deflection plots and observed failure modes reported in Appendix C indicate that it may have a substantially lower level of ductility for seismic design.

A summary of key limitations for use of the above SPS perforated shear wall design method are as follows:

- Limited to use for resistance of wind lateral loads and in seismic design categories A and B, as well as C for single-family homes (alternatively, a conservative seismic response modifier of 2.0 may be used where seismic lateral force resisting system design is required)
- Limited to 2-ply SPS walls with plies cross-laminated (inner panel oriented horizontally and outer panels oriented vertically)
- Limited to maximum 8’ floor-to-ceiling heights with no horizontal joints in inner OSB ply of the SPS wall assembly
- Limited to construction detailing and fastening as shown in test specimen drawings of Appendix C, including 2-ply panel lamination mechanical fastener schedule, receiver plate fastening schedule, floor/roof assembly fastening to SPS

wall, and also corner fastening detail where a minimum 2’ corner return is used for partial overturning restraint at the ends of the SPS wall assembly

- A minimum 2’ wide full-height segment of SPS wall construction is required at each end of the exterior wall lines used to resist lateral load.

Additional testing should be considered to expand the scope of the derived in-plane shear resistance design methodology to include use of longer corner return panels, potentially increasing the value for the hold-down overturning restraint adjustment factor, J_{hd}, and thereby increasing the allowable in-plane shear resistance of the SPS system. In addition, use of an engineered wood receiver plate may improve the uplift resistance of the corner return as testing showed the limiting failure mode to be splitting of the receiver plate under uplift load at the corner return panel. Alternative means of providing partial or full overturning restraint should also be considered. Where parallel lamination of the OSB plies is considered useful (as an alternative to perpendicular or cross-lamination), in-plane shear testing and potential alterations of the fitted design equation for the shear capacity ratio, F, should be evaluated. Other additional testing and engineering evaluations to consider include testing under cyclic loading to determine seismic design parameters. Also, combined in-plane shear and wind uplift or gravity axial compressive loads would provide improved insight into combined loading performance. Such testing could be further enhanced by conducting whole building system tests.

3.1.5 1-Ply and 2-Ply SPS Bending Stiffness Tests to Determine Composite Action and End Fixity Effects

As detailed and evaluated in Appendix D, bending tests were conducted on 1-ply and 2-ply OSB panels under simple span conditions to determine the degree of bending composite action that is achieved by the panel mechanical fastener lamination method. The panels were nominal 1-1/8” thick and 48” wide and tested with a 90” span using a two-point load set up and simple (pinned) end bearing supports.

The individual 1-ply tests yielded values for EI and E reasonably consistent with that reported in Appendix F for the ASTM D3042 Method C tests and were conducted to serve as a baseline for evaluating the 2-ply tests. The following bending stiffness result was produced for the cross-laminated 2-ply composite panels for bending in either the outward or inward direction:

$EI\_tested = 9{,}480{,}288$ lb-in.^{2} [COV = 0.032]

Using the modulus of elasticity (MOE or E) values from the 1-ply tests for each of the ply orientations relative to the original OSB panel length and using a transformed section analysis for an idealized fully composite, monolithic cross-section (e.g., perfectly rigid lamination between the two plies which have different E values), the calculated

moment of inertia (MOI or I) and stiffness (EI) values are as follows for a 2-ply panel of 48” width and average thickness of 1.144” for each of the two plies:

$I_{transformed\ section,\ fully\ composite} = 24.392\ in.^4$

$EI_{transformed\ section,\ fully\ composite} = 22{,}280{,}088\ lb\text{-}in.^2$

It is worth noting that if both laminated panels are oriented in the same original panel length or cross-width direction such that each panel has the same MOE for the direction of bending stresses, then there would be no need to perform the above transformed section analysis and the MOI for a fully composite (monolithic) construction would be $I = 1/12(b)(d)^3 = 1/12(48\ in.)(2.25\ in.)^3 = 45.6\ in.^4$ as used for solid rectangular member. If both plies are oriented such that bending stress is in the original panel length (strong) direction, then the EI would be approximately 37,500,000 lb-in.^{2} or greater depending on the E value used to characterize the material bending in a direction aligned with the original OSB panel length direction. However, if both plies are oriented with bending stresses aligned with the cross-panel width direction of the original OSB panel, the EI would be approximately 16,000,000 lb-in.^{2} or less depending on the E-value used to characterize material bending in the weak direction of the original OSB panel.

To typify the opposite extreme of no composite action whereby both plies are assumed to bend independently with effectively no lamination (panels are free to slip at their interface), the following net MOI for two independent panels sharing bending load equally was calculated by summing the average MOI result of the 1-ply tests for each panel bending orientation relative to the original OSB panel length direction:

$EI_{non\text{-}composite,\ independent} = 7{,}097{,}420\ lb\text{-}in.^2$

Comparing the 2-ply EI_tested value to the above two theoretical extremes for characterizing stiffness, the percentage of fully composite action realized relative to the difference of EI between the two extremes is about 15.6% (see Table 8 for analysis). Thus, some benefit in bending stiffness and axial load buckling resistance may be achieved with additional fastening or addition of a rigid adhesive for panel lamination.

Table 8. Analysis of Amount of Composite Action Achieved by the Lamination Fastening Schedule for 2-Ply SPS Walls (see Figure 6)

Fully Composite Section Properties (EI and I) Using Transformed Section Method for 2-ply Panel and Eperp and Epara Values from 1-Ply Test Results		
Eperp =	274,013	psi (average from manufacturer test data is 315,300 psi and ASTM D3042 Method C tests per Appendix F is 350,389 psi)
Epara =	913,427	psi (average from manufacturer test data is 800,080 psi and ASTM D3042 Method C tests per Appendix F is 822,077 psi)
b =	48.045	in. (width of panels)
t_avg =	1.144	in. (average thickness of panels, each ply)
Calculated Values		
n =	0.300	(ratio Eperp/Epara for section transformation to material with homogenous MOE = Epara)
b'-perp =	14.413	in. (adjusted width of transformed panel with MOE = Eperp)
y_bar =	0.836	in. (location of neutral axis of composite section from outer face of Epara panel)
MOI_trans =	24.392	$in.^4$ (moment of inertia, I, of transformed section with MOE = Epara)
EI_fc =	22,280,088	$lb\text{-}in.^2$ (stiffness of fully composite 2-ply panel)
Non-Composite 2-Ply Panel EI Summing MOI x E for Each Panel Bending Independently [i.e., $I_{nc} = 1/12(b)(t_{avg})^3 \times (Epara) + 1/12(t_{avg})(t_{avg})^3 \times (Eperp)$]		
I_{nc} = 7,117,979 $in.^4$		
Non-Composite 2-Ply Panel EI (Each Panel Bending Independently)		
EI_perp	1,638,974	$lb\text{-}in.^2$ (avg from 1-ply test data above)
EI_para	5,458,445	$lb\text{-}in.^2$ (avg from 1-ply test data above)
EI_nc	7,097,420	$lb\text{-}in.^2$ (stiffness of non-composite 2-ply panel with individual panel tested EI)
% Composite =	**15.6%**	**percentage of full composite action realized where % Composite Action = [((tested avg EI 2-ply) - EI_nc) / (EI_fc - EI_nc)] x 100%**

The final stage of bending tests used 2-ply panels as above, but with a pair of 2-ply panels each with the outer ply attached at the top and bottom to floor/roof members spanning a short distance between the two panels as would occur in actual SPS wall construction. Two-point bending loads were applied to the wall panels with the paired wall panel assembly laid horizontally. A test was conducted in each bending direction

(inward and outward). Such bending tests were repeated for another assembly of the same construction, but with the addition of minimum 4” thick foam sheathing and two 1x4 wood furring strips attached as shown in the specimen construction details of Appendix D.

From these tests, two effects were investigated and quantified (see Table 9) as further described in Appendix D to support use of the NDS column buckling equations to accurately predict the SPS wall system’s axial compressive strength with and without combined out-of-plane bending loads:

- The degree of end fixity defining the difference between simply supported (pinned ends) and idealized fixed end beam bending was found to be about 56.9%. In other words, the 2-ply cross-laminated SPS panel behavior with ends connected to a floor/roof assembly was slightly more like a fixed end beam than it was a simply supported beam. Thus, the SPS 2-ply wall panel could use a beam model and associated beam equations that roughly split the difference between the stresses and deflections that these two bounding beam end reaction assumptions would yield. Also, the effective column buckling length factor, K_e, could be interpolated between a value of 1.0 (pinned end column) and 0.5 (column with both ends fixed) as follows: $K_e = 1.0 – 0.569(1.0 – 0.5) = 0.72$. While these values may be appropriate for use with the NDS column buckling equations to predict column axial buckling load capacity, it is recommended for design purposes that a conservative degree of end fixity of 50% be used for bending analysis and, consequently a K_e value of 0.75.
- The degree of added stiffness provided by the addition of two layers of 2” thick foam sheathing and two 1x4 furring strips attached to the SPS 2-ply panels resulted in an average increase in the SPS 2-ply bare panel bending stiffness, EI, of about 15% (or a factor of 1.15 times the EI of the bare panels). However, for the two tests conducted, the bending stiffness factor was 1.02 for the outward bending direction and 1.27 for the inward bending direction. The cause of this difference is unknown but may be related to how the assembly responded to the applied load whereby the outward bending load application resulted in less friction (due to no normal force) between the SPS panels and the furring and foam material layers, thereby reducing the degree of composite action. With load applied to the furring side of the assembly for inward bending, the opposite condition occurred, resulting in a potentially inflated degree of composite action. Given this concern, it is likely that the actual degree of composite action effect is closer to the 1.02 factor and therefore essentially negligible (or modestly conservative to ignore). The number of tests was very limited, so these tests are considered exploratory. Design analyses should not factor in the additional bending or compressive strength provided by these additional nonstructural wall

components. In the next section, it will be shown that the contribution to axial load capacity is not more than about 2.6%, which is reasonably consistent with the findings in these bending tests.

Table 9. Analysis of Degree of Fixity of Top and Bottom of 2-Ply SPS Panels Fastened to Floor and Roof Members and Analysis of Added Bending Stiffness (EI) Caused by Inclusion of Exterior Wall Materials (e.g., two layers of 2-inch thick foam sheathing and two 1x4 wood furring strips at 24 inches on center).

								Pinned End Assumed	Fixed End Assumed	
Test Name	**Load [a] (lbs)**	**Avg Defl. @ Load 1 (in.)**	**Load [b] (lbs)**	**Avg Defl. @ Load 2 (in.)**	**Nom. Panel Width (in.)**	**Avg. Panel Thickness (in.)**	**Span (in.)**	**EI-simple Stiffness (lb-in.^{2})**	**EI-fixed Stiffness (lb-in.^{2})**	
SPS 2-ply panels tested "bare" as in tests 3-1 through 4-3										
13-1 Inward	260	0.2	1,296	1.15	48	1.144	96	17,122,717	3,722,330	
14-1 Outward	292	0.21	1,345	1.18	48	1.144	96	17,044,849	3,705,402	**Degree of End Fixity (%)**
							Average	**17,083,783**	**3,713,866**	**56.9%**
SPS 2-ply panels tested with added 4" thick foam sheathing and two 1x4 pine wood furring at 24"oc centered on face of panels										
16-1 Inward	411	0.2	1,076	0.68	48	1.144	96	21,752,889	4,728,889	
17-1 Outward	322	0.2	1,075	0.88	48	1.144	96	17,386,918	3,779,765	**Added Stiffness Factor**
							Average	**19,569,903**	**4,254,327**	**1.15 [c]**

a. Bending load was applied as two point loads at third points of span. Reported loads in table are total applied loads by UTM.

b. Degree of end fixity is unknown so EI is determined using a pinned end beam and a fixed end beam assumption to compare with the actual EI of 9,470,288 lb-in.^{2} from previously tested 2-ply panels using a simply supported beam with pinned ends (see bending test data and analysis for specimens 3-1 through 4-3). EI-simple= $23PL^3/648xDefl$ and EI-fixed= $5PL^3/648xDefl$ where deflection is measured at center span, L=span, and P=1/2 applied load. The calculated degree of end fixity indicates the extent to which the SPS wall panel assembly behaves as a fixed beam relative to that of a simply supported beam.

c. The “added stiffness” factor is the average ratio of EI determined for the bare panel tests (13-1 and 14-1) and the tests with added two layers of 2” thick foam sheathing and two 1x4 pine wood furring at 24”oc installed centered on face of panels. See construction drawings for attachment of furring. The foam sheathing was specified and labeled with a minimum compressive resistance of 15 psi. For the two tests (16-1 and 17-1) the added stiffness factor ranged from 1.02 to 1.27 with outward bending resulting in the lower added stiffness factor.

3.1.5.1 Recommended Parameters and Approach for Design of SPS Walls Using NDS Column Buckling Equations

- **K_e = 0.75 ←** (use 0.72 if predicting axial load capacity)
  - Use to determine column or SPS wall panel effective buckling length.
- **Composite Action Factor = 15% ←** (use 15.6% if predicting SPS wall segment combined axial or combined axial and bending load ultimate capacity)
  - Use to determine effective MOI for a partially composite 2-ply SPS wall panel for determining the extreme fiber tension stresses in both SPS panel bending directions caused by applied inward or outward bending loads.
  - For 2-ply SPS panels in a cross-laminated condition where the MOE differs for the two plies due to different OSB material properties for stress oriented in the OSB panel length direction vs. across its width direction, the fully composite MOI must be determined using the transformed section approach or similar method. The non-composite MOI is then determined by evaluating the two plies of the transformed section as though they are independent sections and adding their individual MOIs together. The effective MOI is then proportioned by the composite action factor between these two bounding conditions of fully composite and completely non-composite action.
  - In addition, the applied compressive stress in the transformed 2-ply SPS wall panel should be based on the transformed section's cross-section area and the compressive resistance based on that property of the outer panel whose greater MOE served as the basis for transforming (reducing) the width of the inner panel.
- **Degree of End Fixity = 0.5 ←** (use 0.569 if predicting SPS wall segment combined axial and bending load capacity)
  - Use to evaluate bending moment from applied bending load using beam equations for pinned ends vs. fixed ends (and applying the degree of end fixity factor to weight the result accordingly between these two bounding conditions) such that together with the effective MOI due to partial composite action (see above) the extreme fiber bending tension stress can be determined as an input to the NDS column buckling equations. Additionally, the calculated bending stress for the inner ply (with transformed reduced width) for inward bending direction must be factored by the ratio of the MOE of the two plies to determine the actual bending stress on the inner ply (based on actual untransformed width).
  - It is important to note that the transformed section has asymmetric bending characteristics, so the maximum bending stress from evaluation of both

bending directions should be used as the input to the NDS column buckling equations.

- **Effective 2-Ply SPS Panel Thickness = d_eff ←**
  - An effective 2-ply panel thickness, d_eff, must be used in the NDS column buckling equations because the 2-ply SPS panel is not a solid homogenous "column," and consequently does not exhibit 100% composite action in bending or buckling.
  - It is recommended that d_eff be determined using the composite action factor (above) to scale between two bounding assumptions for determining thickness of the slender SPS wall panel column: (1) d_fully composite = twice the distance from the outer face of the outer SPS panel (with the higher MOE in the vertical stress direction) to the transformed section's neutral axis which corresponds to a fully composite transformed section as determined for the effective MOI above and (2) d_noncomposite = the individual ply thickness which corresponds to a case where each panel is acting in a completely non-composite manner and free to bend or buckle independently of the other.

The above recommended parameters for design of SPS walls using the NDS column buckling tables to predict the design axial load resistance (under axial load only or combined axial and out-of-plane bending loads) are used later to predict the results of SPS wall assembly axial load tests addressed in the next section as well as the preliminary axial load tests of individual SPS wall panels addressed earlier in this report (see Section 2.3) from a previous test project.

3.1.6 SPS Wall Assembly Axial Load (Compression Buckling) Tests

Details for this final SPS testing stage can be found in Appendix E. The test results are rather simple, yet significant for the purpose of characterizing the axial load behavior of the SPS wall construction. In these tests, the paired 2-ply wall assemblies used in the previous section for non-destructive bending stiffness tests were oriented in the vertical direction and axial loads applied through the top floor/roof members to create axial point loads (floor/roof reactions) at the top of the panels. The 2-ply SPS wall panels had a clear span (floor-to-ceiling unsupported height) of 96" with the floor and roof members attached only to the outer panel ply. The panels were 48" wide with the two nominal 1-1/8" thick cross-laminated OSB plies in each SPS wall panel.

The following maximum axial load capacities were realized with the load-deflection plot shown in Figure 13 for specimen 18-1.[14] Specimen 18-1 included a layer of 4" foam

[14] The max total load reported is the total load applied to two SPS 2-ply wall panels (load applied to center span of two floor/roof beams attached at each end to top of each SPS wall assembly; the two floor/roof beams located 1' in

sheathing and 1x4 wood furring attached to the surface of the outer ply of both 2-ply wall panels. Deflection measurements are the average of two measurements at each of the edges of each 48” wide SPS 2-ply wall panel.

- Test 15-1: Bare 2-ply SPS wall panels (2, paired): Max total axial load = 37,052 lbs
- Test 18-1: 2-ply SPS wall panels (2, paired) with 4” foam sheathing and two 1x4 wood furring strips: Max total axial load = 38,014 lbs

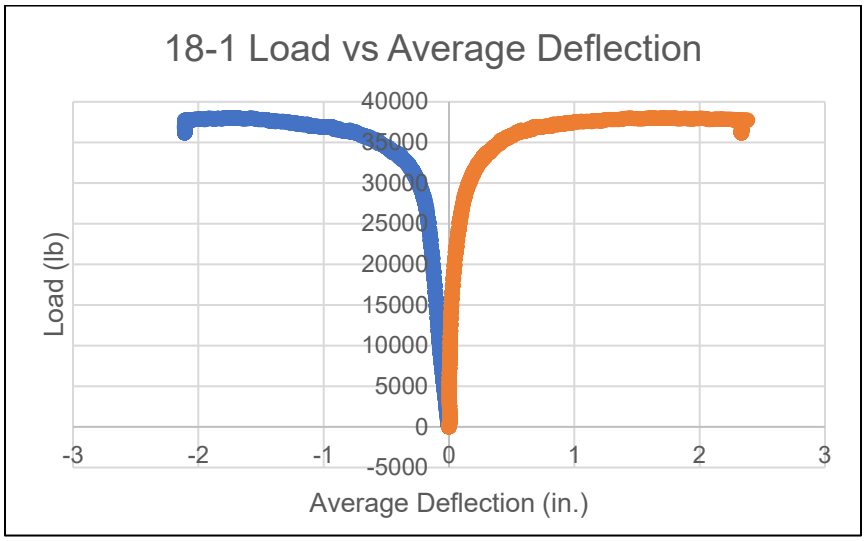

Figure 13. Total axial load vs. mid-height out-of-plane panel deflection plot for paired wall assembly specimen 18-1 where the two deflection plots shown are for each of the 2-ply SPS wall panels in the tested assembly[15]

Taken at face value, Test 15-1 indicates substantial axial load carrying capability of the SPS wall construction with loads applied realistically as floor or roof truss reaction point loads. This test indicates potential for a unit axial load capacity for an 8’ ceiling height 2-ply cross-laminated SPS wall of about 37,052 lbs/2 = 18,526 lbs per 4’ segment of wall, or about 4,632 lbs/ft of solid wall length. Using a conventional safety factor of 3 for this test would result in an ASD design axial load resistance of about 1,540 lbs/ft of solid wall length which would accommodate roughly a 30-ft clear span floor system with 10 psf dead and 40 psf design loads.

The above simple “axial load design value” approach does not, however, account for the presence of any combined out-of-plane bending load (e.g., wind load) which would

from each edge of SPS panels with 2’ spacing between). Failure mode was panel buckling in both tests. Test 15-1 was terminated before reaching peak load to remove the string potentiometers (deflection measurement) and restarted.

[15] Specimen 18-1 included a layer of 4” foam sheathing and 1x4 wood furring attached to the surface of the outer ply of both 2-ply wall panels. Deflection measurements are the average of two measurements at each of the edges of each 48” wide SPS 2-ply wall panel.

tend to reduce the axial load design resistance. It also does not efficiently use a design method and customary safety factoring applied to the design of columns (like a slender SPS panel wall column) resisting axial and bending loads. For these design concerns to be effectively and efficiently addressed, a design methodology (like the NDS column buckling equations) must be properly modified and applied for use with the SPS 2-ply cross-laminated OSB wall construction in a manner that is consistent with the performance observed in the above test result as well as the preliminary panel-only axial compression tests discussed earlier in this section. This final step is addressed in the next section.

Before continuing, however, it should be recognized that the addition of foam sheathing and furring to the SPS wall construction (specimen 18-1) resulted in only a small benefit to the measured axial load capacity (about a 2.6% increase over that for the "bare" specimen 15-1). This is consistent with similar findings for the bending tests addressed in the previous section. Ignoring this nonstructural composite action effect appears to introduce only a small conservative bias to characterizing the SPS wall system's axial compressive load resistance.

3.2 OSB Material Property Tests and Recommended Design Properties

This section reports findings of material property tests of a representative sample of the lot of OSB material used for the various SPS structural behavior tests reported in previous sections. The detailed test data and associated analyses to derive material properties are included in Appendix F. These properties serve two purposes: (1) "fingerprinting" the OSB material properties and (2) providing OSB material properties for use in analyzing the observed performance of the SPS wall panels and assemblies using accepted engineering analysis methods as presented in prior sections of Appendix F.

For "fingerprinting" the OSB material properties associated with header testing and in-plane shear testing and the associated recommended design values or method, allowance should be given for reasonable variation from tested material properties reported in this section during the actual OSB material production and required production sampling for OSB material property quality control. For example, it would seem reasonable to allow for as much as a maximum -10% variation for a given lot of OSB material produced that is intended for use with SPS wall construction. This fingerprinting requirement would not necessarily apply to use of the NDS buckling equations for the SPS wall system provided the appropriate OSB material properties are used as inputs and are factored and adjusted to a reference ASD design basis as done later in this section based on currently available material property data.

Finally, recommended design values are provided based on manufacturer-provided property data (presented earlier in Appendix F) and those obtained from this project that

are not currently available from the manufacturer (e.g., lengthwise and cross-width panel compressive stress properties).

3.2.1 ASTM D4761 Section 8 – Bending Flatwise, Two-Point Loading

Using a benchtop UTM at Home Innovation Research Labs, flatwise bending tests were performed on 10 specimens for each of two specimen length orientations relative to the original OSB panel's length (strength axis). The specimen dimensions were nominally 1.125" thick, 12" wide, and 18" long with a span between support reactions of 16". Loading was applied at the third points of the span in accordance with ASTM D4761, Section 10. The applied total UTM load vs. UTM crosshead deflection plots are shown in Appendix F. In all cases, a bending rupture failure mode was observed.

The following results were obtained:

- $F_{b,ult,perp}$ = 2,366 psi [n=10, COV=0.058]
- $F_{b,ult,para}$ = 4,652 psi [n=10, COV=0.104]
- $E_{perp,avg}$ (apparent) = 204,759 psi [n=10, COV=0.042]
- $E_{para,avg}$ (apparent) = 416,197 psi [n=10, COV=0.063]

The average $F_{b,ult}$ results are within one standard deviation of MOR data reported by the OSB manufacturer for several years of production data for the 1-1/8" thick OSB material (see Section 2.3). These small-sized flatwise two-point bending load tests do appear appropriate as a means to "fingerprint" the MOR or Fb of the OSB material. However, corrections were not made to derive Fb values on the bases of a "true" moment incurred due to the nature of the simple test setup used without deflection measurements separate from the UTM crosshead movement.

The E_{avg} values are about half those reported earlier by the manufacturer as well as those reported next using the ASTM D3043 test method which applies a pure moment to the test specimens. The flatwise bending tests used a very low L/d ratio which likely resulted in significant shear deflection being included in the overall specimen deflection behavior. Therefore, these E values should not be used for the purpose of evaluating the SPS structural behavior tests reported earlier or for design purposes, although they could serve as a means of "fingerprinting" the material behavior. They are provided here for informational purposes only.

3.2.2 ASTM D3043, Method C, Pure Moment Test (Bending Flexure)

These "pure moment" bending tests are preferred over the two-point bending load test method used in the previous section. A TECO QL-3 panel bending test machine was used which follows ASTM D3043 Method C as specified in Section 7.5 of the DOC PS 2-18 standard for OSB materials. The detailed test data and other details are found in Appendix F. Tests were run only in the elastic range to characterize the bending

stiffness and modulus of elasticity (E) of the sampled OSB material from the lot of material used in this testing project. The following results were obtained:

- $E_{avg,para}$ = 822,077 psi [n=10, COV=0.054]
- $E_{avg,perp}$ = 350,389 psi [n=10, COV=0.038]

Additionally, these tests required multiple panel thickness measurements and constitute the only instance in the entire test project where panel thickness was documented. As shown in Appendix F, the average panel thickness was 1.144”. Therefore, this thickness value was used in engineering analysis of the SPS panel and assembly test data addressed earlier in the appendix.

3.2.3 ASTM D4761 Section 10 – Axial Strength in Compression

Compression tests were performed on 10 specimens for each of two specimen length orientations relative to the original OSB panel’s length (strong direction). The specimen dimensions were nominally 1.125” thick, 6” wide, and 12” tall. Tests were conducted in accordance with ASTM D4761, Section 10. In all cases, compression crushing failure modes were observed without specimen buckling. Compression load-deflection plots are shown in Appendix F.

The following results were obtained:

- $F_{c,ult,perp}$ = 2,724 psi [n=10, COV=0.10]
- $F_{c,ult,para}$ = 3,050 psi [n=10, COV=0.09]

The above values are used in evaluating the SPS wall assembly axial load (compression buckling) tests conducted in this project and reported earlier in Appendix F.

Additionally, the following typical values for compression modulus were determined from the central tendency of the elastic (linear) range of the compression load-displacement plots in Appendix F:

- $E_{c,perp}$ = 330,000 psi
- $E_{c,para}$ = 437,000 psi

3.2.4 Material Property Summary and Recommended Design Values

Table 10 summarizes the above material property test data for OSB materials used in this project for SPS test specimen constructions, including appropriate ASD design values.

Table 10. Tested Properties

	Thickness (in.)	Density (pcf)	E-para (psi)	Fr-para (psi)	E-perp (psi)	Fr-perp (psi)	Fc-para (psi)	Fc-perp (psi)
AVG/Ult.	1.144	x	822,077	4,642	350,389	2,366	3,050	2,724
COV	x	x	0.054	0.104	0.038	0.058	0.09	0.1
DESIGN (unadjusted)	1.125	x	451,236	1,851	197,883	1,029	1,250	1,095
n=10 for all, except E where n=5; 1 lot								

The compression modulus properties shown in Table 11 were also derived from material property compression tests conducted to derive the F_c values in the above table, although there is not a specific need for design of the SPS wall system.

Table 11. Compression Modulus

Ec-para (psi)	Ec-perp (psi)
437,000	330,000

For comparison, the OSB material manufacturer material property data presented earlier in this Section are shown in Table 12, including appropriate ASD design values.

Table 12. Huber 1-1/8" OSB Reported Properties

	Thickness (in.)	Density (pcf)	E-para (psi)	Fr-para (psi)	E-perp (psi)	Fr-perp (psi)
AVG/Ult.	1.133	40.7	800,080	4,939	315,300	1,985
COV	0.005	0.049	0.064	0.089	0.079	0.099
DESIGN (unadjusted)	1.125	40.7	431,233	2,028	165,256	799
n = 61 lots, 2001 through 2006						

Based on the above data sources with adjustments and factoring consistent with wood material design properties in accordance with the NDS, the design properties shown in Table 13 for 1-1/8 thick OSB material used for SPS wall construction are recommended for use with the NDS column buckling equations addressed earlier in this appendix:

Table 13. Recommended Design Values for SPS OSB Panels 1-1/8” Thick

Thickness (in.)	E-para (psi)	E-perp (psi)	Fb-para (psi)	Fb-perp (psi)	Fc-para (psi)	Fc-perp (psi)
1.125	431,233	165,256	2,028	799	1,250	1,095

4 Summary of Technical Advisory Groups

4.1 Structural Technical Advisory Group

Early in the project, a Structural Technical Advisory Group (S-TAG) was formed to guide the test plan and specific test protocols. This group was a cross section of representatives from composite industry trades and testing organizations, product manufacturers, and structural engineers. The first meeting introduced them to the SPS system and some of the key structural questions that need to be answered to aid market adoption and code acceptance. The project team also laid out the proposed test plan for their input and feedback. The S-TAG was convened following each test sequence to review the results, share any insights or potential concerns, and to go over the next test sequence.

In addition to ensuring efficiency in our test plan, sequencing, and protocols, the S-TAG identified several building applications and new delivery approaches for the SPS system. They suggested pursuing different markets, such as modular or manufactured housing, where structural integrity and rigidity might be particularly advantageous. They also pointed out the SPS panel could be used for adding shear capacity to more traditional stud-frame construction.

4.2 Constructability Survey

4.2.1 Overview

Home Innovation conducted twelve interviews with industry professionals regarding the SPS Wall System. The purpose of the interviews was to gain insight from a small group of industry professionals on their current system and introduce them to the SPS wall system. We wanted to obtain their feedback regarding the perceived benefits and drawbacks of the system, how it would integrate with or affect their current practices, and concerns and questions that they may have.

The interviews were conducted by a Home Innovation qualitative research specialist between July 19 and July 26, 2023 in the form of one-hour videoconference interviews. Interview participants were contacted and qualified for the interviews using a recruitment screener to ensure sufficient knowledge and experience to provide insight into the discussion topics. The screening document is shown in Appendix F. Professions recruited and interviewed were Home Builder (6), Custom Builder/Remodeler (2), Architect (2), Plumber (1), and Electrician (1). The states that the professionals performed work in include Michigan, Ohio, District of Columbia, Kansas, Texas, Minnesota, Iowa, California, Tennessee, and Washington.

Home Innovation market research staff also drafted a discussion guide that was followed during the interviews (see Appendix F). The guide began with a discussion on experiences with current wall systems and the pros and cons associated with those systems. The SPS system was then introduced with a series of slides that included

photographs, written descriptions, and a time-lapsed video of a full installation. Initial reactions were recorded and discussed. Afterwards, participants were asked for their input on how they believed the SPS system would impact their construction process, labor, scheduling, and home performance. Participants were encouraged to offer potential solutions to eliminate drawbacks and resolve concerns. The interviews concluded with exploring the opportunities and barriers for the SPS system.

4.2.2 Current Practices

It was most common among the professionals interviewed that their current practice for exterior wall systems was traditional lumber framing with 2’ x 4’ or 2’ x 6’ walls. Some of the common practices included using engineered wood headers, wood structural panel sheathing, continuous exterior insulation, and Zip System sheathing. The primary reasons for selection of the current system were cost-effectiveness, labor force familiarity with the technique, wide availability of materials, the possibility for last minute changes at job sites, and the ability to meet code-required insulation. Respondents were not completely satisfied with the structural systems they were using; there were some issues noted regarding existing systems such as cost, lumber quality, and a lack of skilled labor. These issues drove people to, in the past, try at least one new system such as structural insulated panels (SIPs), cold-formed steel framing, ICFs, or some form of offsite wall panelization. Most were open to new systems that would reduce the skill level that is needed for installation.

4.2.3 Initial Reaction to SPS System - Benefits

The discussion leader introduced the SPS system through slides and a short video demonstrating the system at a project site. The professionals interviewed saw the primary benefit of the system as the speed of construction. An additional benefit that was appealing was the smaller crew sizes needed to erect walls and the associated overall labor cost savings. Another appeal was the high-performance home that resulted from the construction with increased thermal performance, moisture control, and reduced air infiltration. Those who already built or designed high-performance homes expressed that they could get high-performing homes with their current methods, but the SPS system was appealing in that this method was faster. There were several professionals who noted that the method appeared to have high strength, particularly regarding wind resistance. Simplification of the process and materials on site was noted as another appeal. Other benefits mentioned were the ability to train their own crews on the system and eliminating the need for an insulation contractor for walls. Additionally, the flexibility created from on-site cutting of windows, doors, and so forth was seen as a benefit, especially considering that other panelized systems did not allow for adjustment of window and door location without extensive modification.

4.2.4 Initial Reaction to SPS System - Drawbacks

The primary drawback of the SPS system was the difficulty of logistics, particularly when using a crane. The professionals had used cranes previously, but some believed that this system requires a more skilled crane operator, taller cranes, and a longer rental period that would result in higher costs. There would also be added difficulty as some lots are not well suited for the crane—with steep grades or egress issues for small lots. The builders assumed that there would be issues due to the added complexities with electrical and plumbing details. However, the electrician and plumber did not share that viewpoint as these respondents were experienced with other solid wall systems such as concrete walls. Another drawback discussed was the limit to building shapes that included the use of curves and bays, which led them to believe they would need a hybrid approach to achieve those styles. The interior finishes such as raceways, electrical boxes, and wall bump outs may not be accepted by homeowners. Lastly, some mentioned potential limits on home designs and dimensions, particularly wall heights, due to the maximum 8’ x 24’ panel size. One mentioned the potential difficulty of installation on windy days.

4.2.5 Initial Reaction to SPS System - Concerns

There were some concerns that were discovered during the interviews; these concerns are not problems but rather areas that would need to be addressed when the SPS system is introduced and communicated to potential buyers. Lumber and sheathing are ubiquitously available; what is the availability of this product and the lead time for ordering and shipping? If there is an unplanned need for additional materials due to panel damage during transit/loading, the builders would need to be assured material would be readily available. Another area of concern was building inspectors, as this new method would need to be introduced to all inspectors in each of the jurisdictions in which the system is being used. There was doubt about how easily the necessary precision could be achieved for the sill plate and receiver plate alignment. Moisture concerns about the intersection between panels and the underside of gypsum board were noted. Structural integrity due to moisture damage was a concern if the OSB has prolonged exposure to moisture after construction is completed. The ability to hide the panel seams on the interior if left uncovered by drywall or flooring was unclear. The length of fastener needed to accommodate accessories like light fixtures would need to be extended due to the thickness of the insulation installed, and guidance was needed. Lastly, some of the exterior finishes such as stucco may not be suitable with this system.

4.2.6 Initial Reaction to SPS System - Questions

Participants had questions that would need to be answered before they utilized the system. The ability to handle a point load, such as where floors rest on the ledge of the inner panel, was brought up during the interviews. Questions regarding windows and

openings and what percentage of the wall could be cut out required further answers. In addition, the SPS system integral header span capacity was unknown, and the maximum window or door opening size needs to be known. Participants were also uncertain whether an engineer was required for each building project. There were several questions regarding finishing of insulation edges at windows and the flashing details around window openings to keep water from behind insulation. There were also questions regarding the fire performance compared to a traditional wall, length of time the panels can be exposed to the weather during the construction phase, resistance to termites, and the ability to relocate the weather resistive barrier (WRB) to the outside of the exterior insulation. Participants also questioned whether the SPS system was suitable for all types of exterior wall finishes, particularly those that could be affected by thick exterior insulation. There were also questions about changing the location of the exterior insulation—moving it to the inside or using a combination of interior and exterior insulation to meet R-value requirements. This would change the core concept of the Perfect Wall but if it made the system more constructable and more likely to be adopted, it may be worth consideration.

4.2.7 Where the SPS System Is Best Suited

It was agreed that one of the best-suited applications for this concept is affordable housing. It is also well suited in projects where customization is minimal, and the designs/dimensions are repeated. Another area well suited for the system could be locations where weather delays are common. In addition, a turnkey solution where the supplier of the material would perform the installations was appealing to many participants.

4.2.8 Next Steps

This was an initial study of twelve people to gain insight into current practices and impressions when introduced to the SPS wall system. These results allow us to learn what areas to focus on for further research using a larger number of industry professions. A larger study audience would provide greater insight into the impact of cost and cost savings on adopting the new system, builder needs regarding marketing and technical support, and how to overcome the hesitancy of subcontracted labor to support its adoption. Answering these questions would allow us to identify the most promising market segments for market entry and to quantify the market potential of these markets.

5 Discussion and Conclusions

5.1 Interpretation and Significance of Results

Both the structural testing and the market survey results suggest the SPS system has strong potential for production homebuilding. While it might not be immediately transferable to some house designs, engineering guidance is provided that could inform small design changes that would make the SPS amenable to a broader cross section of small-to-medium-sized entry-level houses. Based on the previous study, the SPS system was shown to be particularly fitting for affordable housing developers, especially when using repeated designs for infill or new developments. The superior energy performance—which is frequently required for subsidized housing—at a comparable and potentially lower cost with volume is an attractive value proposition for affordable housing providers. Also, non-profit affordable housing developers are attracted to the faster close-in time for improved site security and moisture control. This offers subcontractors and volunteers much earlier entry into the home to complete their tasks, reducing overall cycle time. Finally, the local affordable housing developers provided a better opportunity for a controlled side-by-side performance and cost analysis in the previous study on the Affordable Solid Panel "Perfect Wall" System (Schirber et al. 2022).

The results of this study provide a strong backbone for an International Code Council Evaluation Service report to simplify code compliance for the SPS system. In fact, there is likely enough potential and depth of data to develop a code change proposal that would clear the way for even easier market adoption. However, this would take additional funding and/or an industrial sponsor.

Additionally, the test plan was intentionally designed with one-story specimens so the structural analysis could inform and support future development of the SPS into an offsite panelized system. The SPS strength and shear performance would seem to make it amenable to modular and/or manufactured housing construction. In addition, the testing would support use of the SPS as a structural enhancement, especially for adding shear resistance to more traditional light-frame construction.

5.2 Potential Limitations of the Experimental Design

This study was explicitly set up to determine the general structural behavior for a 2-ply cross-laminated system using a fixed fastening pattern. A series of specific tests were conducted to characterize the structural response and develop an engineering basis for designing with the SPS system. Due to budget and testing limitations, the test plan was focused on the 2-ply cross-laminated system with an 8' floor-to-ceiling height. However, certain test results presented in this report could be used to provide insight into other configurations.

5.3 Applicability of Findings and Actionable Guidance

5.3.1 Application of NDS Column Buckling Equations to Predict SPS Tested Performance

A spreadsheet was developed to implement the NDS column buckling equations presented in Section 3.1.5 using the design parameter findings and approach recommendations reported above and derived from the 1-ply and 2-ply bending stiffness tests. To predict the tested maximum axial load capacity of Specimen 15-1 as reported in the previous section, the material properties used were those representing the average MOE and average ultimate material stress properties reported in the next section as derived from sample of OSB panel materials used in the overall test project.

Using the NDS column buckling equations in the above-described manner, the following comparison is made:

- Predicted axial load capacity of the cross-laminated 2-ply SPS wall panel: 17,330 lbs
- Tested maximum axial load (per panel of test specimen 15-1): 18,526 lbs

The prediction bias is conservative by -6.5%, indicating that the recommended parameters and approach to using the NDS buckling equations may have a small conservative tendency in predicting ultimate axial load capacity (and thus design axial load resistance when using design OSB material properties as presented in the next section).

Similar comparisons were also made to the prior (preliminary) SPS wall panel tests reported at the beginning of this report (see Section 2.3). Assumptions regarding design parameters were required because of unquantified end-restraint conditions affecting simple panel tests in an ASTM E72 test apparatus (e.g., the k_e factor for panels restrained by bearing on their square-cut ends is not quantified). Additionally, those older test specimens came from a different lot of OSB material, so material properties were based on the OSB manufacturer data for MOE and Fr, while Fc was based on material property tests in this project. Even so, the average axial compression load capacity prediction bias was similarly conservative at about -4%.

5.3.2 Example Application of SPS Wall System Design Method

To demonstrate a realistic application of the SPS design methodologies and design data derived from this test project and presented in this report, the following SPS wall design example is considered:

- First story 2-ply cross-laminated wall of home, 24’ in length, 8’ floor-to-ceiling height, supporting second floor and roof with clear span of 30’.
- Design loads:

- Dead loads (D): Roof 15 psf, Floor 10 psf, Wall 10 psf (2-ply SPS + finishes)
- Second floor ASD live load (L): 30 psf (bedrooms)
- ASD roof snow load (S): 25 psf
- ASD components and cladding wind load (W_c): -20 psf and +15 psf (115 mph basic wind speed in wind exposure B)
- MWFRS ASD wind load (W_m): -8 psf and +12 psf (for use with load combinations involving more than just D+W loads)
- ASD wind shear force on SPS braced wall line: 4,122 lbs
- Seismic design category: A/B/C (lateral design controlled by wind; not evaluated)

Determine ASD gravity loads per linear foot of wall length at top of first story wall by load type:

D: (15 psf roof) x (16 ft) + (10 psf floor) x (16 ft) + (10 psf wall) x (10 ft) = 500 plf

L: (30 psf floor) x (16 ft) = 480 plf

S: (25 psf snow) x (16 ft) = 400 plf

Determine design axial load and out-of-plane bending load required for ASD load combinations (wind load factor of 0.6 already included in ASD wind loads above):

- (1) D+L: Axial load = 500 plf + 480 plf = 980 plf (no bending load from wind)
  - Load duration factor, C_d = 1.0
- (2) D+S: Axial load = 500 plf + 400 plf = 900 plf (no bending load from wind)
  - Load duration factor, C_d = 1.15
- (3) D + 0.75L + 0.75S: Axial Load = 500 plf + 0.75(480 plf) + 0.75(400 plf) = 1,160 plf
  - Load duration factor, C_d = 1.15
- (4) D + 0.75L + 0.75S + 0.75W: Axial load = 1,160 plf with bending load from wind, W_m, of:
  - 0.75(-8 psf) = -6 psf (outward bending) and
  - 0.75(+12 psf) = 9 psf (inward bending).
- (5) D+W: Axial Load = 500 plf with W_c = -20 psf (outward bending) and +15 psf (inward bending)

- (6) 0.6D+W: Combined wind uplift and bending will not control panel buckling strength or in-plane shear strength; where W_m from roof uplift exceeds 0.6D, lateral shear resistance of SPS shear wall will be affected unless additional uplift connections are provided by design (not considered in this analysis).

DESIGN CHECK #1: Using NDS column buckling equations with the design parameters and approach recommendations and design material properties presented in Section 3.2, determine the minimum required length of solid wall for the worst-case load combination:

(1): Minimum length of solid SPS wall required = 12.5 ft < 24 ft wall length (OK)

(2): Minimum length of solid SPS wall required = 11.5 ft < 24 ft wall length (OK)

(3): Minimum length of solid SPS wall required = 14.7 ft < 24 ft wall length (OK)

(4): Minimum length of solid SPS wall required = 15.6 ft < 24 ft wall length (OK)

(5): Minimum length of solid SPS wall required = 7.2 ft < 24 ft wall length (OK)

CONCLUSION: Minimum length of SPS wall required to resist column buckling is 15.5' as controlled by Load Combination (4). The wall is 24' long, so this allows room for up to 8.5' of wall length to be occupied by window and door openings. NOTE: If the 2-ply SPS individual panel thickness was 1-1/4" (instead of 1-1/8") resulting in a 2.5" thick SPS wall panel, the minimum required wall length of 15.5' would decrease to 10.7', allowing for almost 5' more of the wall to be occupied by window and door openings.

DESIGN CHECK #2: Check SPS wall line for in-plane shear resistance assuming two windows 2.5' x 5.5' and one door at 3' wide occupying 8' of the allowable 8.5' for openings determined by the above column buckling design check. Use the recommended 2-ply SPS shear wall design method presented earlier in Section 3.1.4 as follows:

$\sum$ Li = sum of full-height wall segment lengths = 24 ft – 8 ft = 16 ft of full-height solid wall segments (each with aspect ratio of greater than 4:1 meaning only solid wall segment lengths of at least 2 ft only are counted toward $\sum$ Li)

Ao = total area of openings = 2 (2.5' x 5.5') + (3.2' x 6.7') = 48.9 ft^2

H = wall height = 8 ft (span between floor surface and ceiling of SPS wall panel)

r = Opening Area Ratio = 1 / [1+ Ao / (H x $\sum$ Li) and must be at least 0.35

= 1/[1 + 48.9/(8ft x 16ft)]

= 0.72

F = shear capacity ratio relative to wall without openings (dimensionless)

$= -2r^2 + 4r - 1$

$= -2(0.72)^2 + 4(0.72) - 1$

$= 0.84$

v_c = unit shear capacity of SPS wall system = 2,037 plf

J_{hd} = hold-down overturning restraint adjustment factor

= 0.44 (based on use of minimum 2-ft wide corner return panels at each end of wall line)

V = shear capacity of wall line = vC x L x F x Jhd

= (2,037 plf)(24 ft)(0.84)(0.44)

= 18,069 lbs

VASD = allowable shear resistance = V/(S.F.)

= 18,069 lbs / 2

= 9,035 lbs < 4,122 lbs (OK)

CONCLUSION: In-plane (racking) shear from wind lateral shear design load does not control the SPS wall line design (i.e., wall openings could be increased if they were not limited by the previous SPS panel column buckling analysis). If axial design loads or floor or roof spans were smaller, wall opening amounts could be increased.

DESIGN CHECK #3: Check end reaction of 24"oc floor and roof trusses to ensure they are less than the recommended ASD design load limit for the attachment to and bearing on the 2-ply wall construction as addressed earlier in Section 3.1.3.

Floor truss end reaction ASD load = (truss spacing)(D+L) = (2 ft)(980 plf) = 1,960 lbs < 3,600 lbs allowable (OK)

Roof truss end reaction ASD load = (truss spacing)(D+S) = (2 ft)(900 plf) = 1,800 lbs < 2,500 lbs allowable (OK)

CONCLUSION: Floor and roof truss end reactions are not a limiting factor provided the connections to and bearing on the SPS wall system are as presented earlier in this report.

DESIGN CHECK #4: Check SPS header bending moment and shear load is less than the recommended ASD design values derived from test data and presented earlier in Section 3.1.1.

Assume the larger 3’ door opening contains a floor truss end reaction either at the center (for maximum moment) or at the end of a header span creating maximum shear.

Maximum bending moment on 3’ SPS header (assumes truss reaction at center span)

= (1/2 truss reaction ASD load) x (1/2 header span)

= 1/4(1,960 lbs)(18”) = 8,820 in.-lbs < 75,200 in.-lbs allowable bending moment (OK)

Maximum shear on 3’ SPS header (assumes truss reaction near end of header)

= truss reaction ASD load = 1,960 lbs < 3,500 lbs allowable shear load (OK)

CONCLUSION: The SPS header construction does not control the width of any of the window openings in the wall line or require alternative design. NOTE: The above header design values apply only for cross-laminated 2-ply SPS header spans not exceeding 6’. Also, the above analysis assumes roof loads and wall dead load above the headers are distributed through the SPS wall system above to the segments adjacent to the wall openings. Certain window or door configurations in the SPS wall of the story above the header could cause loads on the header not counted in this analysis (e.g., a narrow wall segment between door openings in the story above that is located within the span of a header below). These conditions would be unusual, however, for typical construction conditions and SPS building designs.

The primary conclusion from this design example is that it appears the 2-ply SPS wall panel axial and bending load buckling strength may be the limiting factor in many applications, unless for example floor loads are supported with a center beam rather than clear span. Some improvements might also include use of slightly thicker panels (e.g., 1-1/4” instead of 1-1/8” thickness) as that is the most sensitive parameter affecting the SPS buckling strength and ability to accommodate wall openings as shown in Design Check #1 above.

5.4 Future Work

The future work can be divided into two separate but supportive pathways. The first and foremost task would be to streamline market awareness, acceptance, and adoption. Given what has been learned in this project and the previous SPS study, it seems the best route for quicker market adoption is to find or develop a company that can move the system from the product manufacturer to the builder. Similar to SIPs, ICF, and other wall system technologies, it seems necessary to have another party that supports the

product between the manufacturer and the general contractor. It appears a commodity OSB manufacturer is not likely to develop the additional steps in the supply chain to design and distribute the SPS system directly to the builder. The earlier study looked at a model where an enclosure contractor sourced the materials and delivered them directly to the site. This approach would require a significant accumulated demand within a comfortable service area for the enclosure contractor. With minor changes in the supply and delivery system, the SPS could be a very attractive alternative for a semi-custom home builder looking for an innovative and robust building system to provide high-performance, passive house, and net-zero homes.

A second task would be to expand the application and configurations through additional testing. Undoubtedly that transition could move faster with additional testing on other SPS configurations that might improve the flexibility in both design and erection. The structural testing and analysis in this project suggested that it might be possible to develop an offsite panelized version of the SPS system. Another angle would be to further pursue modular and manufactured home builders to incorporate the SPS system into their current or future model lines. While the weight factor might be a concern, the inherent strength and stiffness would be particularly attractive for transportation and erection. Lastly, the increasing awareness and market share of cross-laminated timber may point to some new market opportunities for the SPS system. It might be an alternative for tip-up construction for multifamily residential and light commercial construction.

References

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Schirber, Tom, Rolf Jacobson, Dan Handeen, Garrett Mosiman, and Patrick Huelman. 2022. *Affordable Solid Panel “Perfect Wall” System*. Minneapolis, MN. DOE/GO-102022-5654. www.nrel.gov/docs/fy22osti/81397.pdf.

Appendix A. SPS Header and Floor/Roof Truss Connection Tests

Tests and data reported in this appendix were provided by Home Innovation Research Labs with additional explanatory information provided by Jay H. Crandell, PE of ARES Consulting. Test specimen construction drawings were provided by the University of Minnesota.

A.1 SPS Header Test Apparatus and Specimen Designs

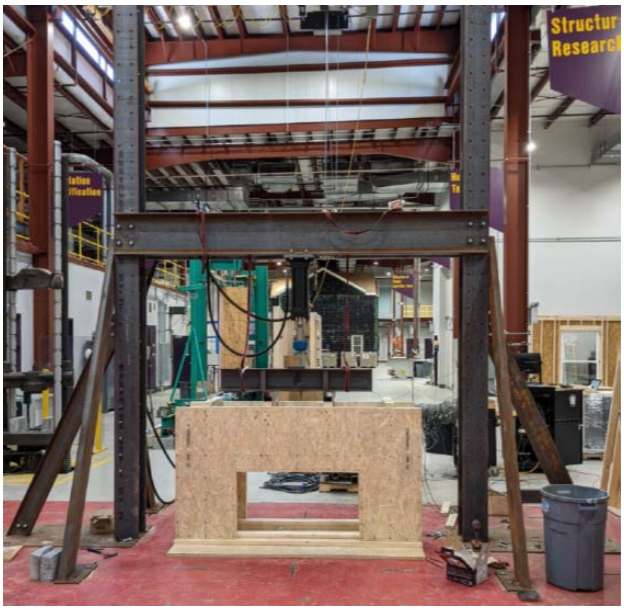

Figure A-1. SPS header test apparatus

The tests were conducted as paired assemblies with wood floor/roof beam members spaced at 24” o.c. and spanning between the pair specimens. The tests included stub wall segments to support the header as would occur in actual SPS wall construction. Except when splices were intentionally included in the inner or outer ply of the 2-ply SPS header, the assembly was monolithic (i.e., both plies of the paired SPS header test assembly were cut from an original 1-1/8” x 8’ x 24’ OSB panel). The 2-plies for a given SPS header construction with either (1) both plies oriented in “parallel” where the original OSB panel length direction was oriented vertically for both plies or (2) the plies oriented in “perpendicular” (e.g., cross-laminated) such that the inner ply of the SPS header was oriented so the original OSB panel length was horizontal and, for the outer ply, the original OSB panel length was oriented vertically. The test specimen construction drawings and details are included at the end of this appendix. The sample size was explicitly selected to provide sufficient size and boundary conditions to account for the interaction of headers in bending with local buckling of a minimally sized adjacent SPS panel. Thus, it is highly unlikely that some type of buckling interaction was not captured in these tests.

For 4’ SPS header tests (see Figure A-2), the two-point loading was applied through floor/roof beams located at the 1/4 points of the header span (each 1’ from the end of the header span over the opening). For 6’ SPS header tests (see Figure A-3), the two-point loading was similarly applied, but with the floor/roof beams located at the 1/3 points of the header span.

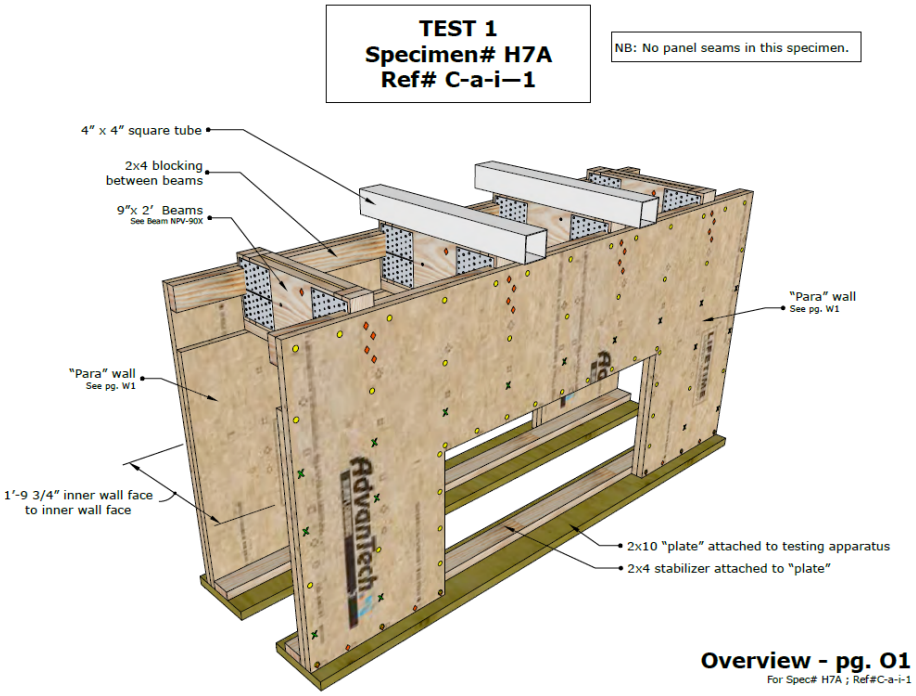

Figure A-2. Test specimen drawing for 4’ header (parallel without seams)

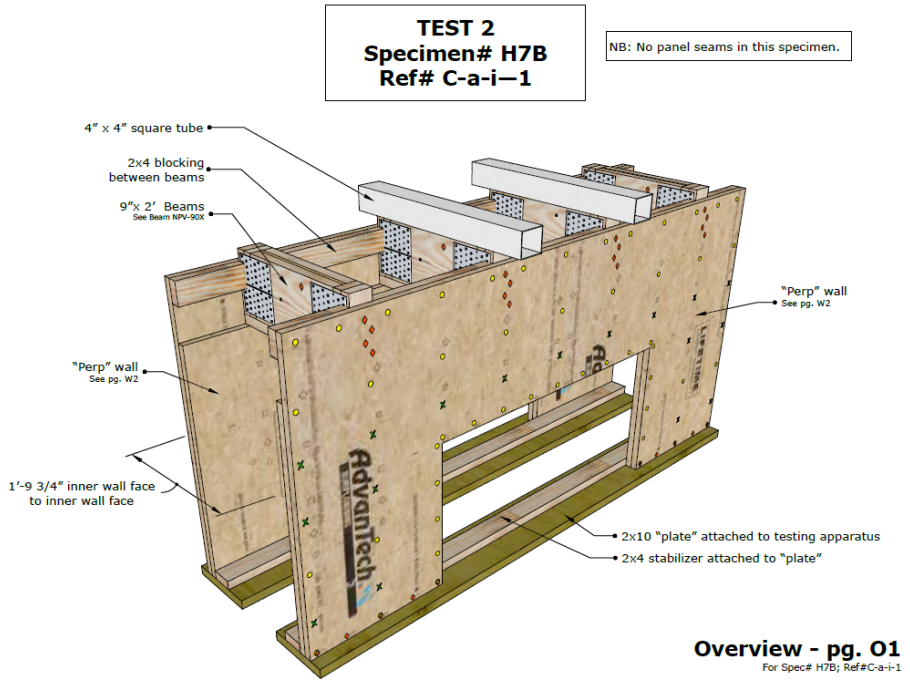

Figure A-3. Test specimen drawing for 4' header (perpendicular without seams)

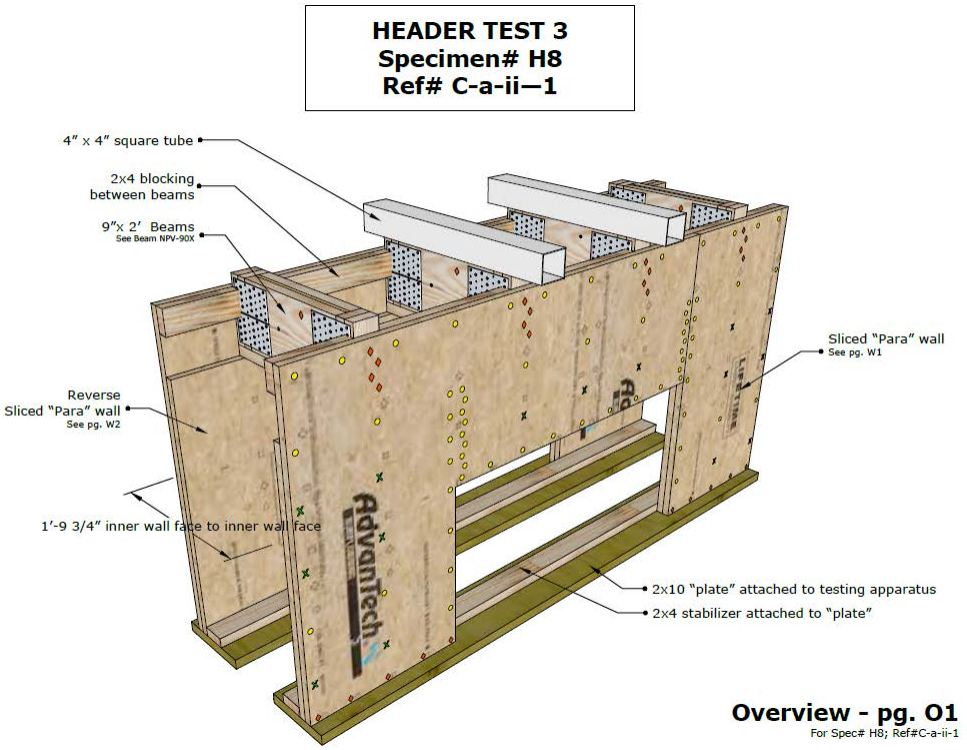

Figure A-4. Test specimen drawing for 4' header (parallel with edge seams)

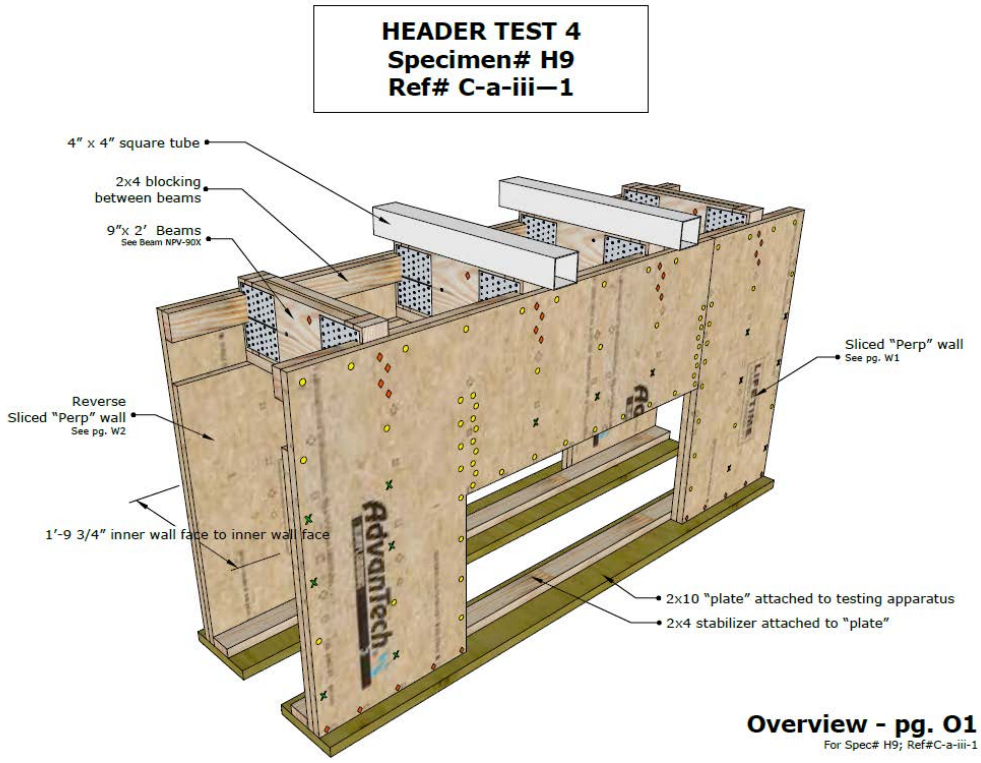

Figure A-5. Test specimen drawing for 4' header (perpendicular with edge seams)

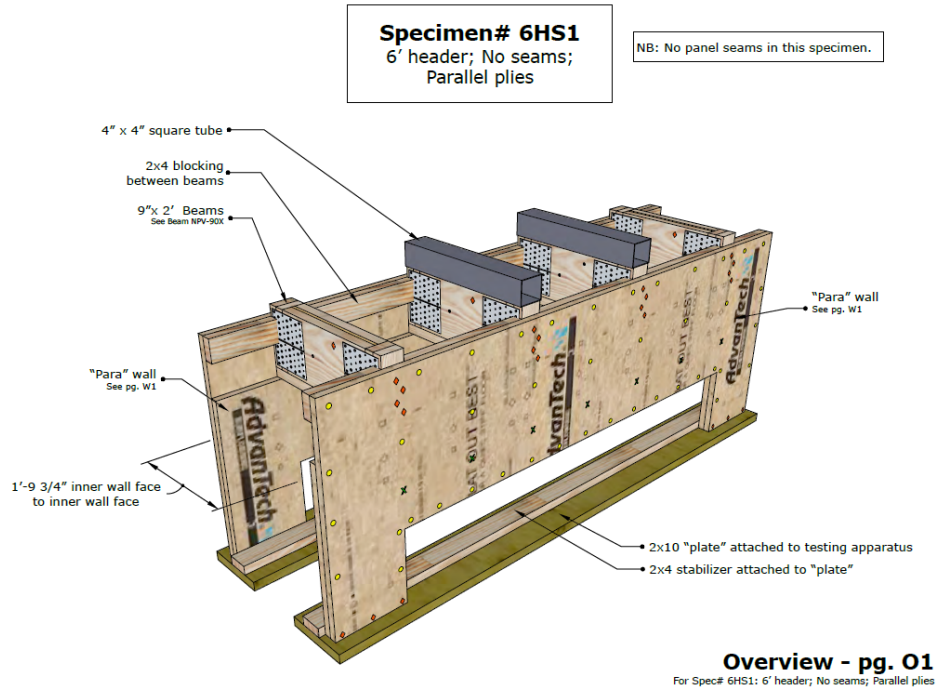

Figure A-6. Test specimen drawing for 6' header (parallel without seams)

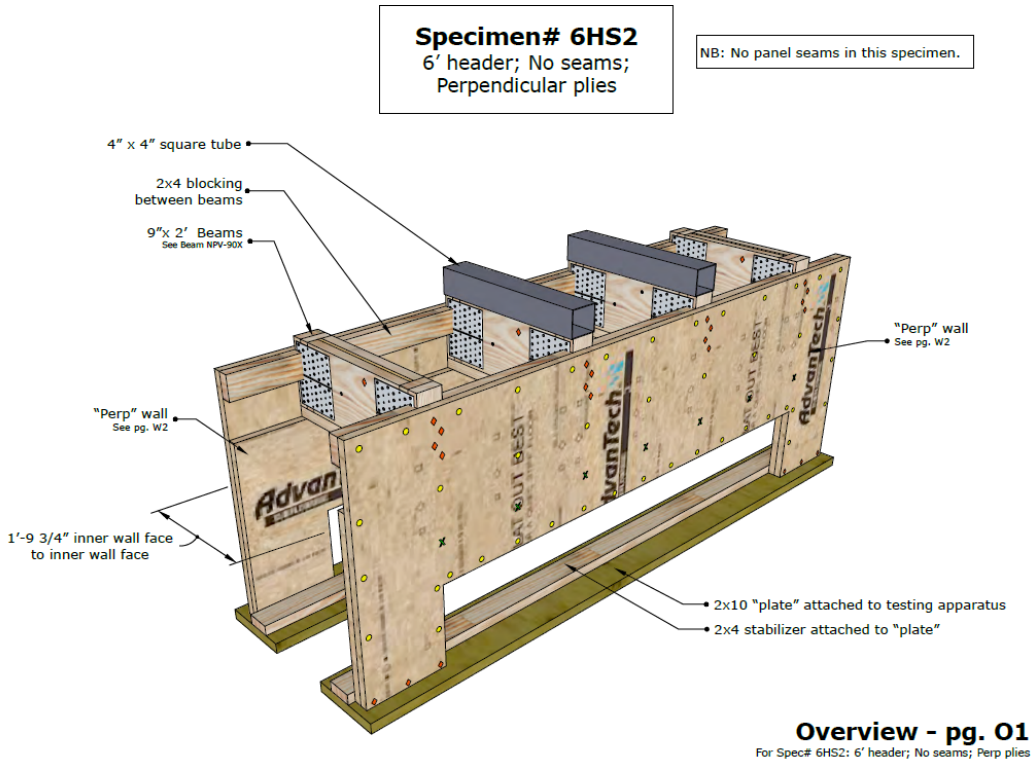

Figure A-7. Test specimen drawing for 6' header (perpendicular without seams)

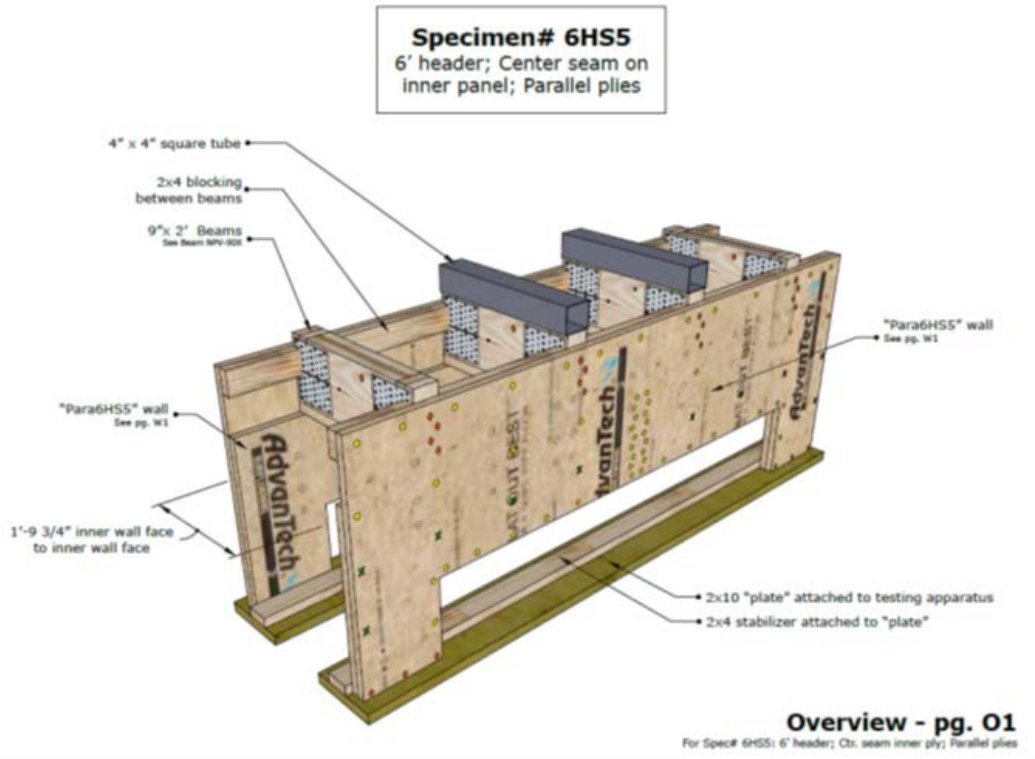

Figure A-8. Test specimen drawing for 6' header (parallel with inner center seams)

Figure A-9. Test specimen drawing for 6’ header (perpendicular with center seams)

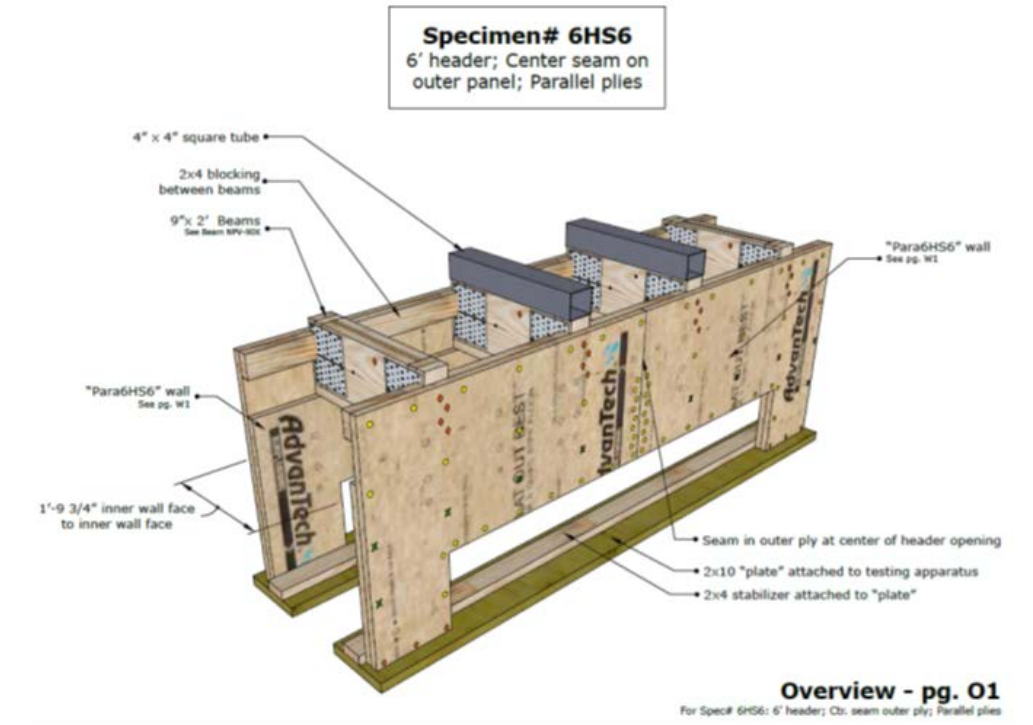

Figure A-10. Test specimen drawing for 6’ header (parallel with outer center seams)

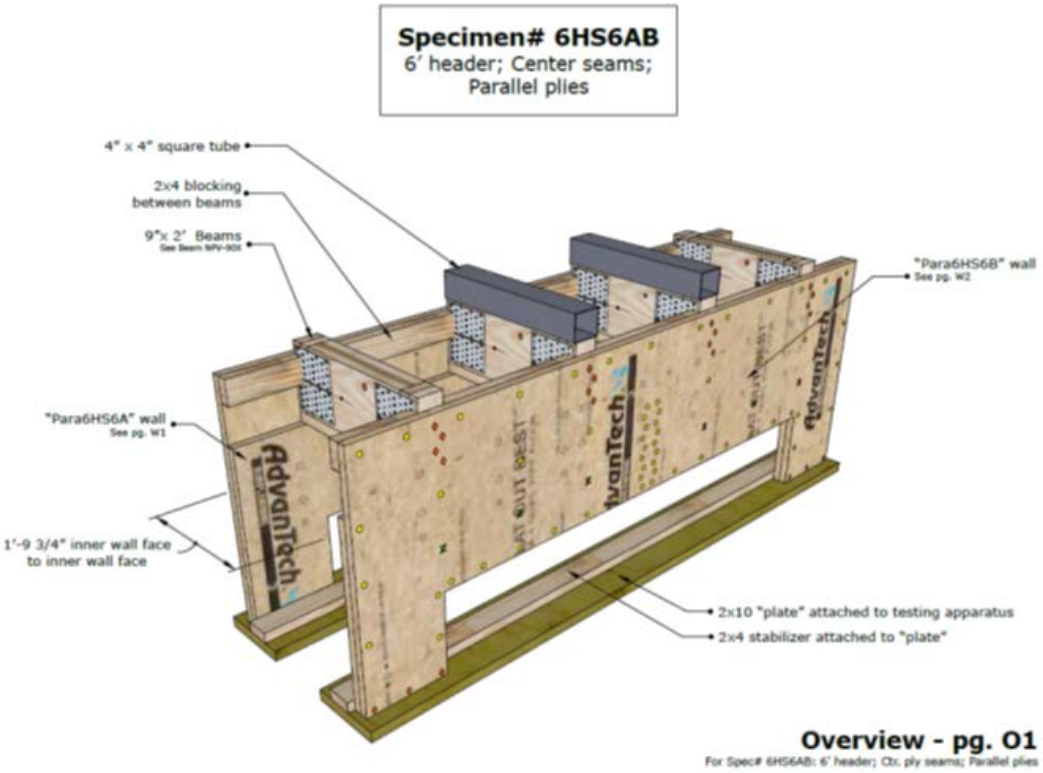

Figure A-11. Test specimen drawing for 6’ header (parallel with center seams)

A.2 SPS Header Test Specimens and Results

Table A-1. SPS Header Tests and Results

Test ID	Maximum Load (lbs) and Deflection (in) Data					Build Notes	Failure Mode
1	Max Load	Defl 1 @ Max Load	Defl 2 @ Max Load	Defl 3 @ Max Load	Defl 4 @ Max Load	4 x 2 ft opening, **Parallel**, SPF bottom chord on floor truss NO JOINTS IN HEADER PLIES	Did not fail; reached max load of 50k. Bottom chord of floor beams compressed and sheared.
	50017	-0.076	-0.253	-0.039	-0.700		
		Max Defl 1	Max Defl 2	Max Defl 3	Max Defl 4		
		-0.085	-0.257	-0.060	-0.726		
2	Max Load	Defl 1 @ Max Load	Defl 2 @ Max Load	Defl 3 @ Max Load	Defl 4 @ Max Load	4 x 2 ft opening, **Perpendicular**, SYP bottom chord on floor truss NO JOINTS IN HEADER PLIES	SYP Bottom chord of floor truss compressed into a wedge. Floor beam slipped off of inner panel. Bottom chords of floor beams also cracked. No/minimal apparent damage to panels.
	41952	-0.141	-0.278	-0.038	-0.713		
		Max Defl 1	Max Defl 2	Max Defl 3	Max Defl 4		
		-0.146	-0.374	-0.058	-0.846		
3	Max Load	Defl 1 @ Max Load	Defl 2 @ Max Load	Defl 3 @ Max Load	Defl 4 @ Max Load	4 x 2 ft opening, Parallel, SPF bottom chord of floor truss. **Alternating Panel seams at corner of opening**	Noticeable panel-to-panel movement at corner of opening on side with inner panel seam.
	33558	-0.172	-0.032	-0.142	-0.635		
		Max Defl 1	Max Defl 2	Max Defl 3	Max Defl 4		
		-0.182	-0.108	-0.143	-0.812		
4	Max Load	Defl 1 @ Max Load	Defl 2 @ Max Load	Defl 3 @ Max Load	Defl 4 @ Max Load	4 x 2 ft opening, **Perpendicular**, SPF bottom chord on floor truss. **Alternating panel seams at corner of opening.** Additional row of fasteners on opening side of panel seams. Two additional rows of additional fasteners.	Damage to inner panel ledge as floor beams gouged inner panel. Crushing of bottom chord on floor beam.
	43130	-0.196	-0.195	-0.160	-0.385		
		Max Defl 1	Max Defl 2	Max Defl 3	Max Defl 4		
		-0.196	-0.240	-0.163	-0.868		
5	Max Load	Defl 1 @ Max Load	Defl 2 @ Max Load	Defl 3 @ Max Load	Defl 4 @ Max Load	6 x 1 ft opening, specimen still 8 ft wide. Parallel, **no seams in panels.**	Not sure how this test failed. The load very quickly dropped around 16%, and a loud pop was heard. Currently, no obvious damage to the specimen. Closer examination will be required to determine failure mode.
	34541	-0.293	-0.301	-0.305	-0.222		
		Max Defl 1	Max Defl 2	Max Defl 3	Max Defl 4		
		-0.294	-0.323	-0.307	-0.226		
6	Max Load	Defl 1 @ Max Load	Defl 2 @ Max Load	Defl 3 @ Max Load	Defl 4 @ Max Load	6 x 1 ft opening, specimen still 8 ft wide. Perpendicular, **no seams in panels.**	Floor beam slipped off of wall again. Damage to inner panel ledge similar to the last perpendicular test. Crushing of bottom chord of floor beam.
	37610	-0.317	-0.306	-0.299	-0.392		
		Max Defl 1	Max Defl 2	Max Defl 3	Max Defl 4		
		-0.321	-0.363	-0.300	-0.952		

Test ID	Maximum Load (lbs) and Deflection (in) Data					Build Notes	Failure Mode
7	Max Load	Defl 1 @ Max Load	Defl 2 @ Max Load	Defl 3 @ Max Load	Defl 4 @ Max Load	6 x 1 ft opening, specimen still 8 ft wide. Parallel, Alternating center seams.	Panel failure center of header on side with exterior seam. Inner panel failure. Around 10K, the gap in the exterior panel closed.
	32457	-0.301	-0.235	-0.484	-0.304		
		Max Defl 1	Max Defl 2	Max Defl 3	Max Defl 4		
		-0.304	-0.243	-0.495	-0.311		
8	Max Load	Defl 1 @ Max Load	Defl 2 @ Max Load	Defl 3 @ Max Load	Defl 4 @ Max Load	6 x 1 ft opening, specimen still 8 ft wide. Perpendicular. Alternating center seams.	Panel failure center of header on side with interior seam. Outer panel failure. Around 14K, the gap in the exterior panel closed.
	40029	-0.571	0.117	-0.328	-0.891		
		Max Defl 1	Max Defl 2	Max Defl 3	Max Defl 4		
		-0.572	-0.703	-2.136	-0.896		

A.3 Selected Photos of SPS Test Specimens

Test ID #1 – Joist/Truss Connection/Bearing Failure (not SPS header)

Figure A-12. Test 1 setup

Figure A-13. Test 1 failure

Test ID #2 – Joist/Truss Connection/Bearing Failure (not SPS header)

Figure A-14. Test 2 truss connection failure

Test ID #3 – Panel Joint Slip of Inner Panel Splice at End of Header and Connection Failure of Joist/Truss (resulting in change to add fasteners at the splice for subsequent tests – see Test ID #4)

Figure A-15. Test 3 panel joint slip of inner panel splice at end of header

Figure A-16. Test 3 connection failure of joint truss

Test ID #4 – Same as Test ID #3 Except with Cross-Laminated SPS Plies and Added Fasteners at Inner Panel Splice (failure of joist/truss connection without splice joint movement or failure)

Figure A-17. Test 4 setup

Figure A-18. Test 4 failure of joist/truss connection without splice joint movement or failure

Test ID #5 – Joist/Truss Connection Failure with Bottom Chord Crushing

Figure A-19. Test 5 setup

Test ID #6 – Joist/Truss Connection Failure with Bottom Chord Crushing (opposite side)

Figure A-20. Test 6 setup

Test ID #7 – Failure of Splice in Inner Ply at Mid-Span

Figure A-21. Test 7 setup

Figure A-22. Test 7 Failure

Test ID #8 – Outer Ply Bending Rupture Failure at Mid Span with Splice in Inner Ply at Mid Span

Figure A-23. Test 8 setup

Figure A-24. Test 8 failure

Appendix B. In-Plane Shear (Racking) Tests of SPS Wall Assemblies

Tests and data reported in this appendix were provided by Home Innovation Research Labs with additional explanatory information and analysis provided by Jay H. Crandell, PE, of ARES Consulting. Test specimen construction drawings were provided by University of Minnesota and are included at the end of this appendix.

B.1 SPS Shear Wall Test Apparatus & Specimen Designs

Figure B-1. Shear wall test apparatus

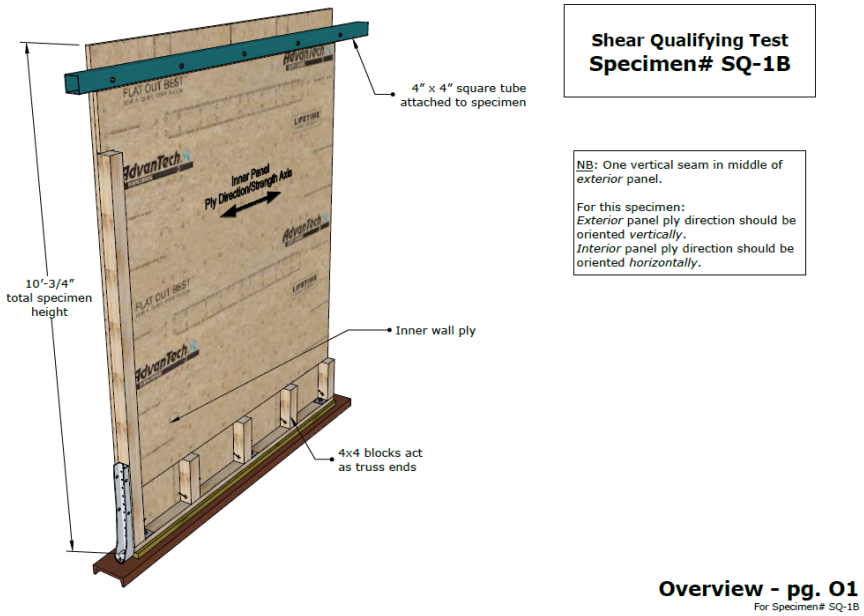

Figure B-2. Test specimen drawing for shear wall (solid)

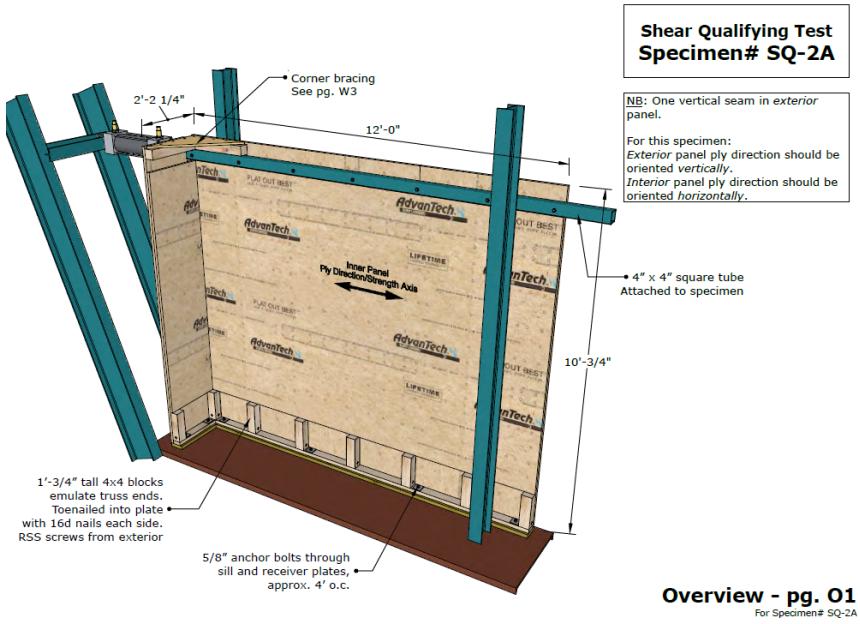

Figure B-3. Test specimen drawing for shear wall (solid with return)

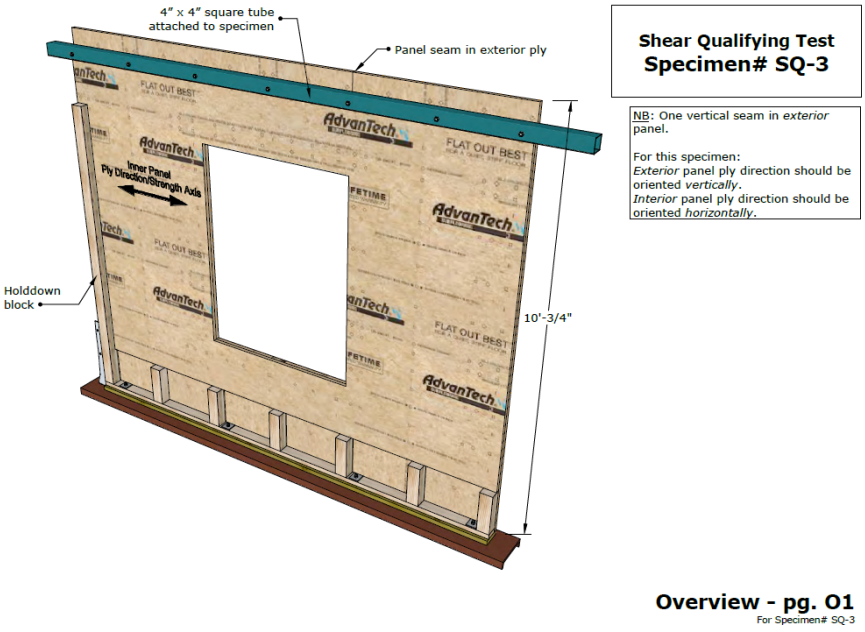

Figure B-4. Test specimen drawing for shear wall (single small opening)

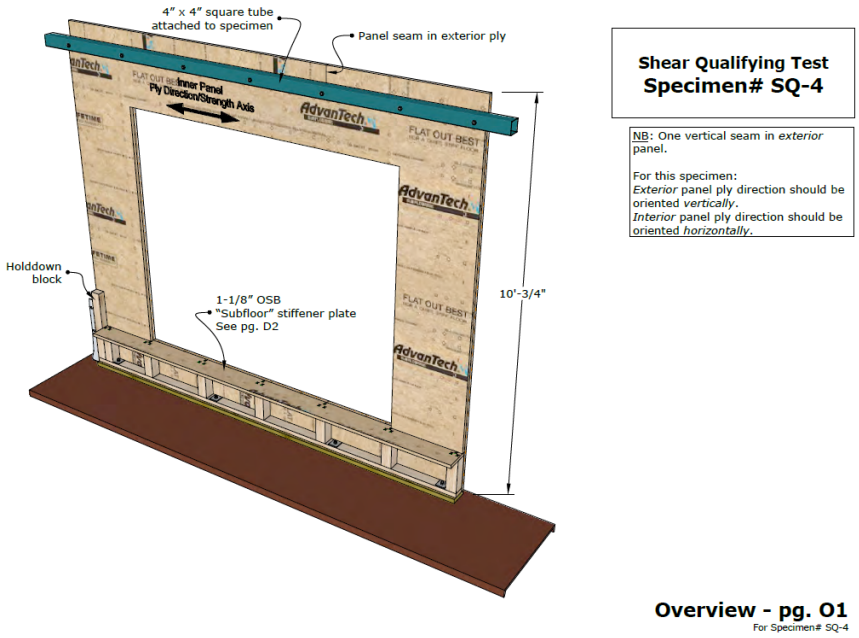

Figure B-5. Test specimen drawing for shear wall (single large opening)

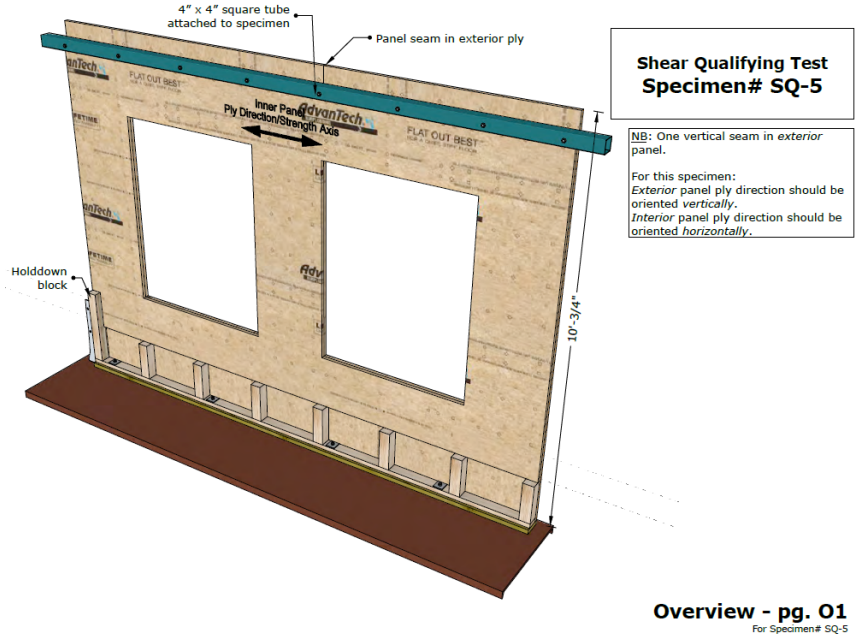

Figure B-6. Test specimen drawing for shear wall (two small openings)

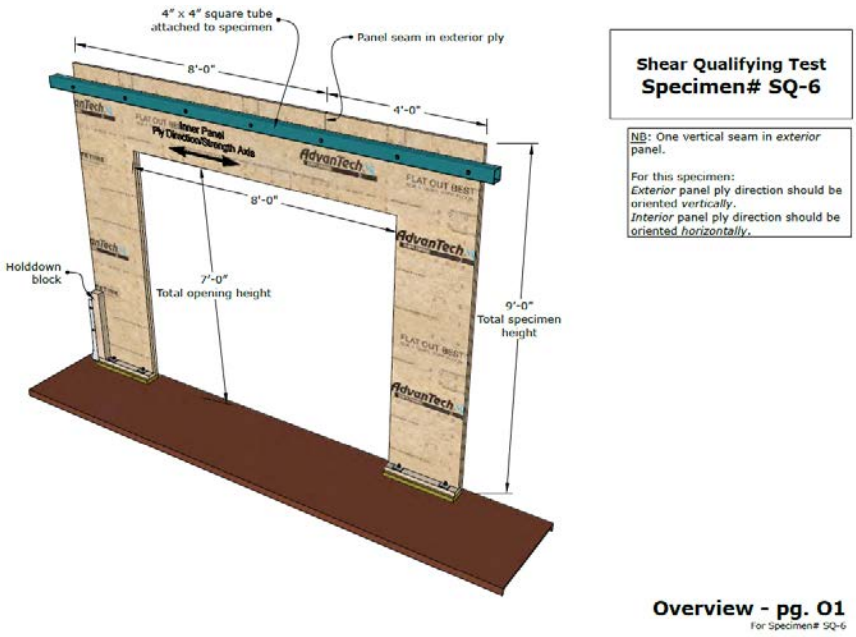

Figure B-7. Test specimen drawing for shear wall (very large opening)

B.2 SPS Shear Wall Test Specimens and Results

Table B-1. Test Specimens and Test Results

Test Name	Length (ft)	Max Load (lbs)	Load Per Ft	Drift at Max Load (in)	Block Uplift at Max Load (in)	Panel Uplift at Max Load (in)	Panel Compression at Max Load (in)
SQ-1A 8x8 Baseline	8	11391	1424	0.976295	0.080153	0.439088	0.197194
SQ-1B-1 8x8 Base full 8ft 4x4	8	22775	2847	1.594911	0.135987	0.451769	0.232135
SQ-1B-2 8x8 Base full 8ft 4x4	8	16137	2017	1.844990	0.123192	0.398237	0.489146
SQ-1B-3 8x8 Base full 8ft 4x4	8	16454	2057	1.936151	0.161122	0.377980	0.443916
SQ-2A 8x12 with 2ft Corner HD	12	11081	923	0.854989	0.510684	0.754761	0.118727
SQ-2A-2 8x12 with 2ft Corner HD	12	10415	868	0.791779	0.505523	0.607703	0.115186
SQ-3 12ft (1) 65% full 4x4 HD	12	22990	1916	0.978766	0.081060	0.176985	0.192478
SQ-4 8x12 ft (1) 85% Opening	12	7449	621	0.982955	0.025507	0.144623	0.086083
SQ-5 8x14 ft (2) 65% Openings	14	18389	1313	0.910773	0.078406	0.292567	0.069157
SQ-6 8x12 ft (1) 85% No Floor	12	7314	610	1.001368	0.033891	0.180780	0.099383

Note: ASTM E564 test method was used with a monotonic displacement (drift) rate of 0.1 in/min.

B.3 Analysis of SPS Shear Wall Test Results

Table B-2. Analysis of Test Results

Wall ID	Description	Wall Height (ft)	Length (ft)	Full-height Segment Aspect Ratio	Wall Gross Area (ft^2)	Sum Length of Segments (ft)	Total Opening Area (ft^2)	Opening Area Ratio [r]	Shear Capacity Ratio [F]	Max Shear (lbs)	Unit Shear (PLF)	Unit Shear Segments Only (PLF)
Full Overturning Restraint Tests												
Avg 1B-2&3	8'x8' baseline - no openings	8	8	1:1	64	8	0	1.00	1.00	16296	2037	2037
3	8'x12' one 4'x62.25" windows	8	12	2:1	96	8	20.75	0.76	0.94	22990	1916	2874
4	8'x12' one 8'x82" door	8	12	4:1	96	4	54.67	0.37	0.30	7449	621	1862
5	8'x14' two 4'x62.25" windows	8	14	4:1	112	6	41.5	0.54	0.64	18389	1314	3065
6	8'x12' one 7'x8' garage door	8	12	4:1	96	4	56	0.36	0.30	7314	610	1829
Partial Overturning Restraint Tests (corner return)												
2A	8'x12' with 2' corner restraint - no openings	8	12	0.67:1	96	12	0	1.00		11081	923	923
2B	8'x12' with 2' corner restraint - no openings	8	12	0.67:1	96	12	0	1.00		10415	868	868
											Avg	**896**
							Partial Restraint Shear Capacity Reduction Factor:					**0.44**

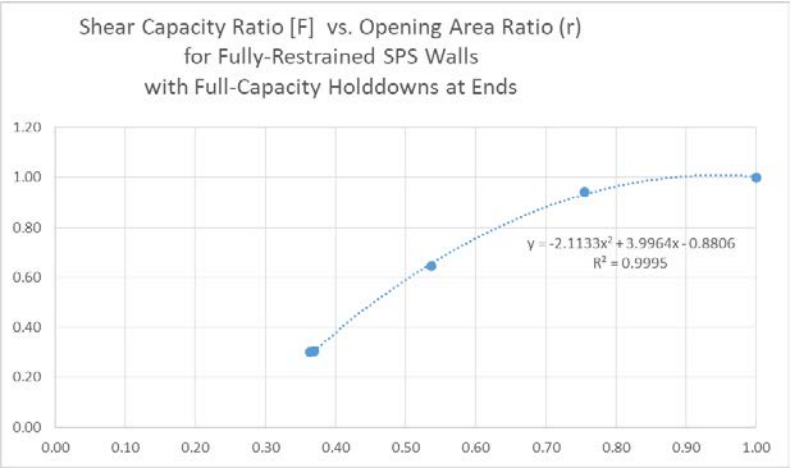

Figure B-8. Shear capacity ratio vs. opening area ratio

F = shear capacity ratio relative to wall without openings (dimensionless) ≤ 1.0

= -2r2 + 4r - 1 (simplification of coefficients shown in F vs. r graph to result in F ≤ 1.0 in all cases)

r = Opening Area Ratio = 1 / [1+ Ao / (H x ∑ Li)

Ao = total area of openings

H = wall height

∑ Li = sum of full-height wall segment lengths (with max. 4:1 aspect ratio as determined by the floor-to-ceiling height to the along-wall length of each segment; full-height wall segments of a greater aspect ratio shall not be included in ∑ Li).

B.4 Selected Photos of Shear Wall Test Specimens and Load-Drift Plots

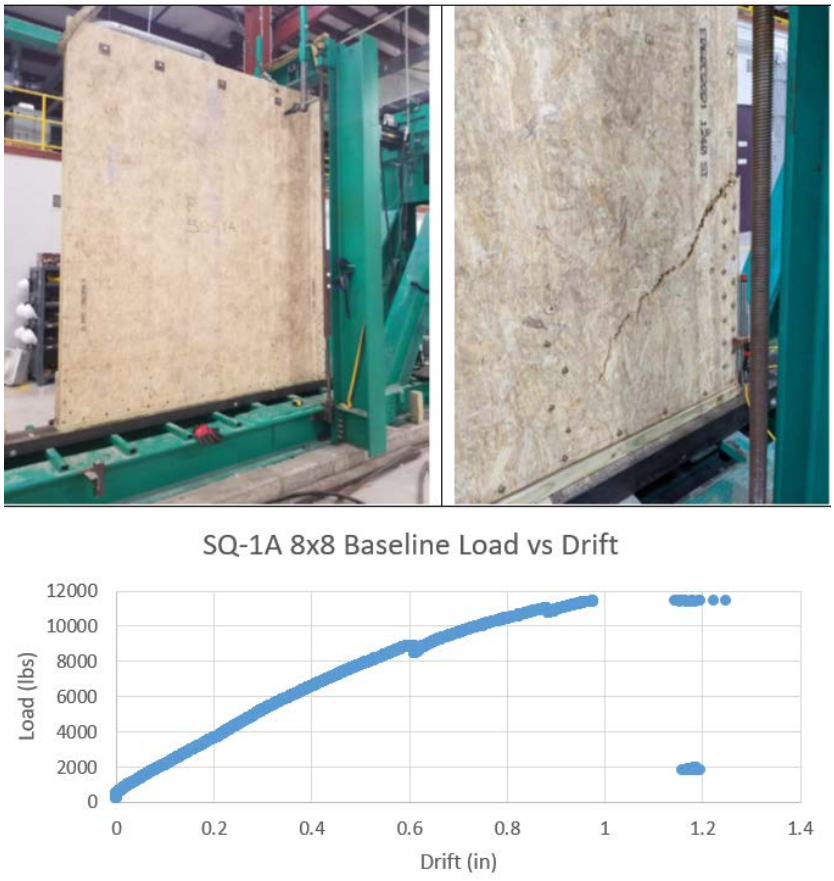

Figure B-9. Shear test of SPS Wall A

Initial baseline 8’ x 8’ SPS wall shear test showing SPS outer panel tension fracture about hold down with partial-height 4x4 blocking at hold down device (Simpson HDU11-

SDS2.5). The 4x4 blocking was extended to full 8’ height in all subsequent testing to provide full overturning restraint as intended, except for tests of SPS corner returns (see Figure B-3). Additionally, the panels were cross-laminated (outer panel oriented with its length (strength axis) in vertical direction and inner panel oriented with its length in the horizontal direction (panel weak axis or width in the vertical direction). NOTE: For all tests, the 8’ wall height dimension is based on floor-to-ceiling height excluding the additional outer panel height to accommodate floor and roof assembly depth).

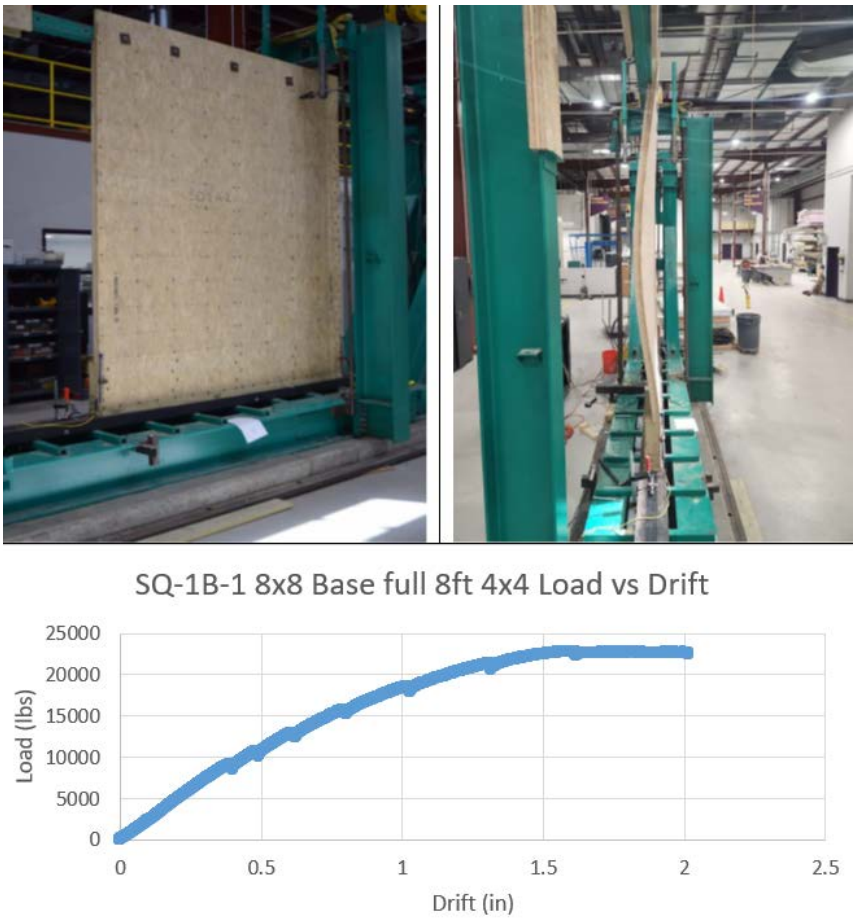

Figure B-10. Shear test of SPS Wall B

Baseline 8’ x 8’ SPS shear wall test showing failure mode of inward panel buckling at compression end of SPS wall specimen (see Figure B-4).

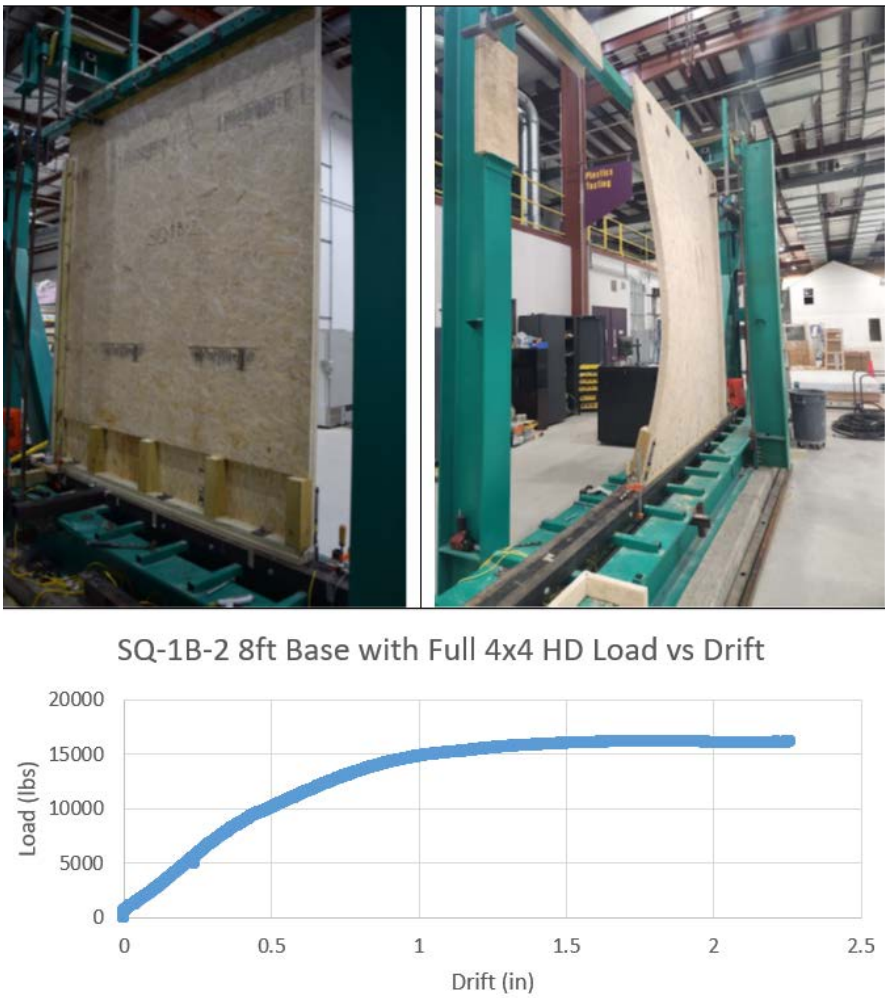

Figure B-11. Shear test of SPS Wall C

Baseline 8’x8’ SPS shear wall test showing failure mode of outward panel buckling at compression end of SPS wall specimen (NOTE: the 4x4 blocks represented ends of floor truss members but without a floor diaphragm and framing to restrain the outward buckling observed).

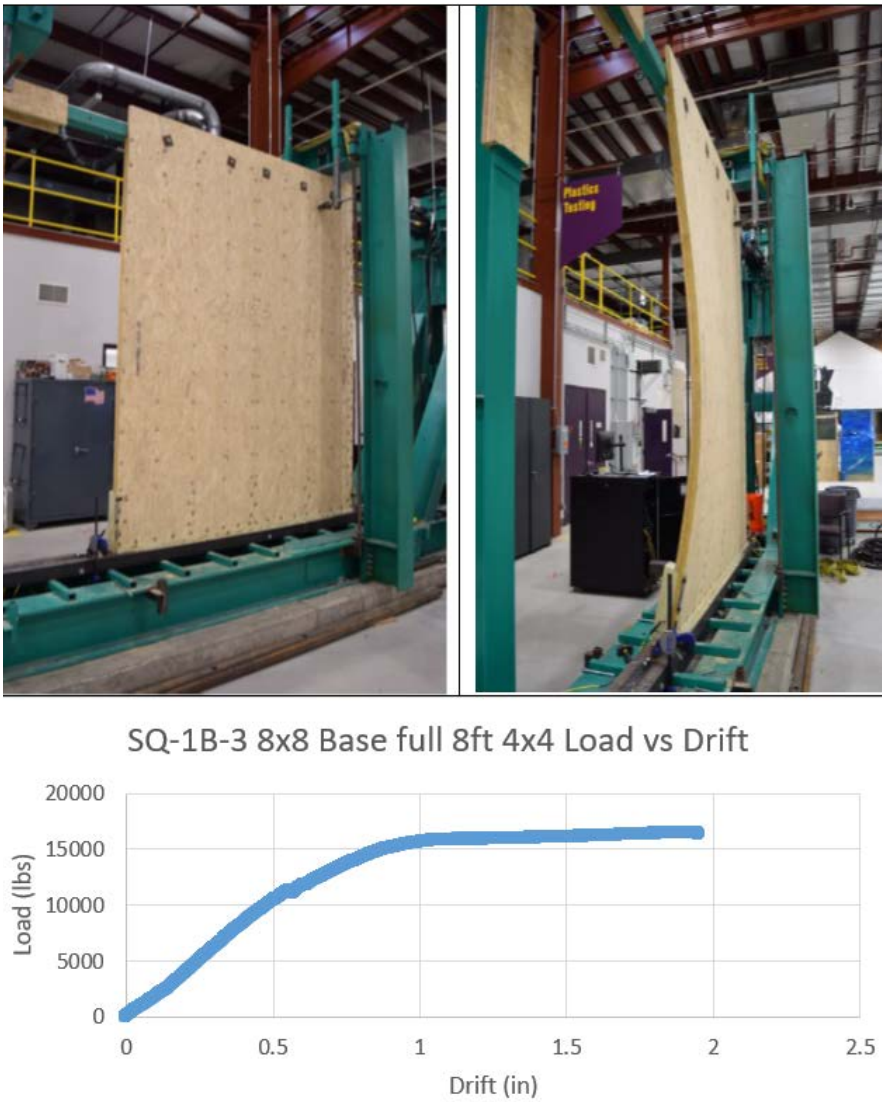

Figure B-12. Shear test of SPS Wall D

Baseline 8’ x 8’ SPS shear wall test showing failure mode of outward panel buckling at compression end of SPS wall specimen (same as Figure B-5).

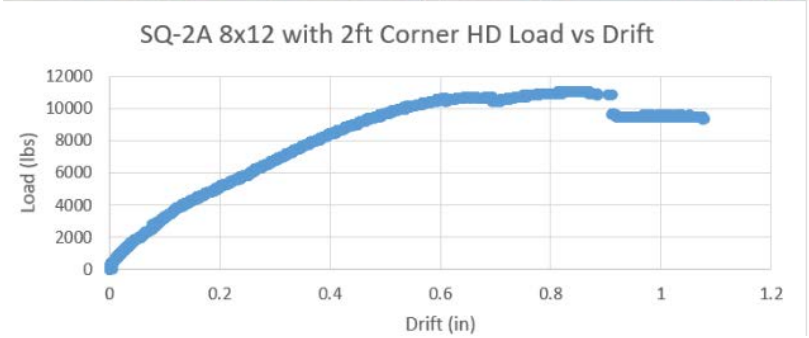

Figure B-13. Shear test of SPS Wall E

8’ x 12’ SPS shear wall test with 2-ft corner return in lieu of hold-down to determine effect of partial overturn restraint provided by SPS wall corner construction. Failure mode was splitting of corner return receiver plate along fastener line at base of outer SPS panel.

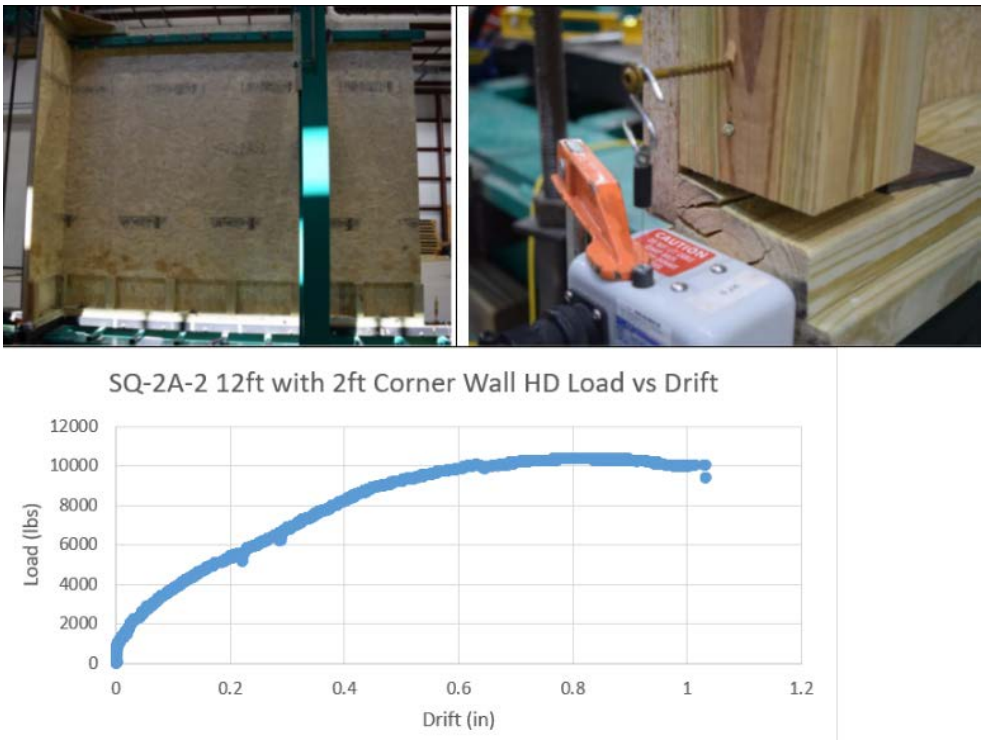

Figure B-14. Shear test of SPS Wall F

8’ x 12’ SPS shear wall test with 2-ft corner return in lieu of hold-down to determine effect of partial overturn restraint provided by SPS wall corner construction. Same failure mode as in Figure B-7 with splitting at end of receiver plate at 2-ft corner return shown above.

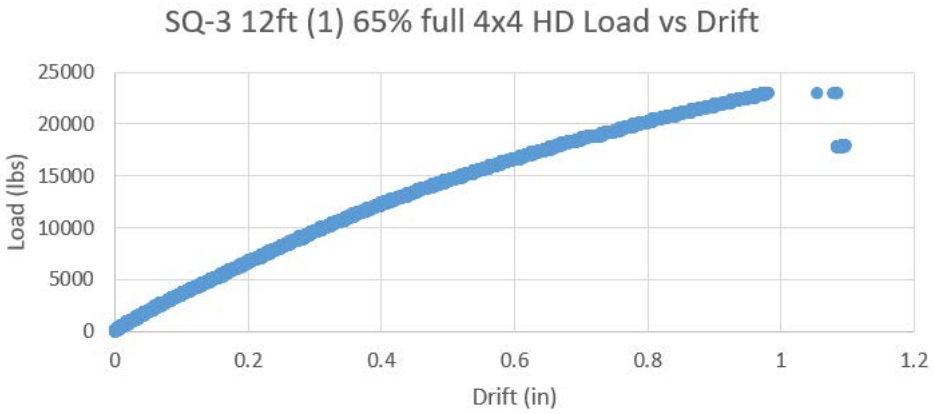

Figure B-15. Shear test of SPS Wall G

8’ x 12’ SPS shear wall test with 48” x 62.5” window opening size (opening height 65% of SPS wall’s 8-ft floor-to-ceiling height) resulting in full-height wall segments with an aspect ratio of 2:1 (height:width). Failure mode was fracturing of both panel plies at top window opening corner at the uplift end of wall and the inner panel at the lower window opening corner at the compression end of wall where the outer panel had a vertical splice aligned with the jamb of the window opening. These fractures were due to tension stress concentrations.

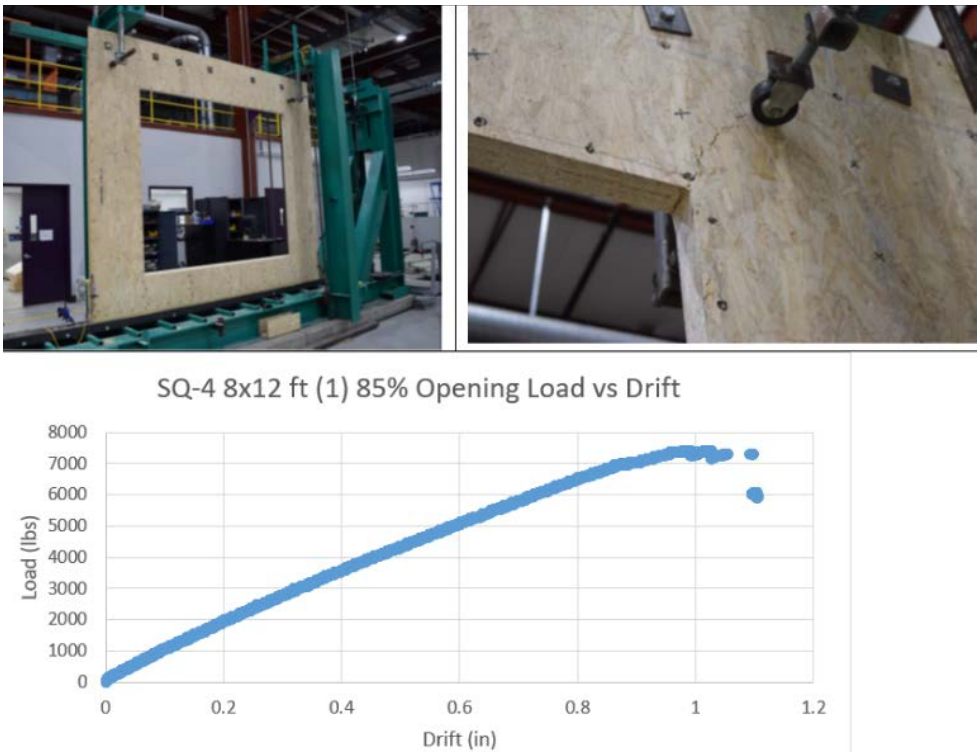

Figure B-16. Shear test of SPS Wall H

8’ x 12’ SPS shear wall test with 8’ x 82” large door opening size (opening height 85% of SPS wall’s 8-ft floor-to-ceiling height) resulting in full-height wall segments with an aspect ratio of 4:1 (height:width). Failure mode was fracturing of both plies at the upper corner of opening at the uplift end of the wall due to tension stress concentration.
NOTE: The SPS panel height below the opening represents the depth of a floor system.

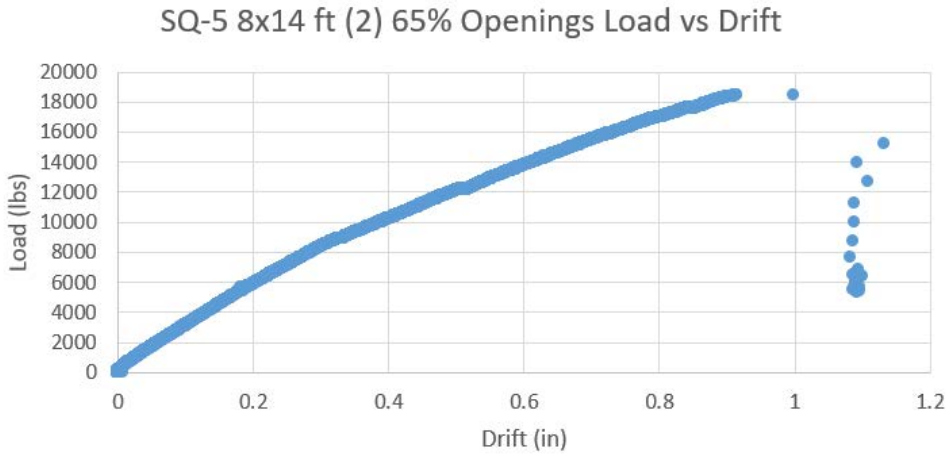

Figure B-17. Shear test of SPS Wall I

8’ x 14’ SPS shear wall test with two 48” x 62.4” window openings (opening height 65% of SPS wall’s 8-ft floor-to-ceiling height) resulting in full-height wall segments with an aspect ratio of 4:1 (height:width). Failure mode was fracturing of the outer SPS ply just above the hold-down bracket which was fastened to a shorter 4x4 blocking member (also see Figure B-3). This test was not repeated with a longer 8’ 4x4 blocking member as the load capacity was considered near the capacity that would have been otherwise limited by fracturing at corners of the window openings (see Figure B-11).

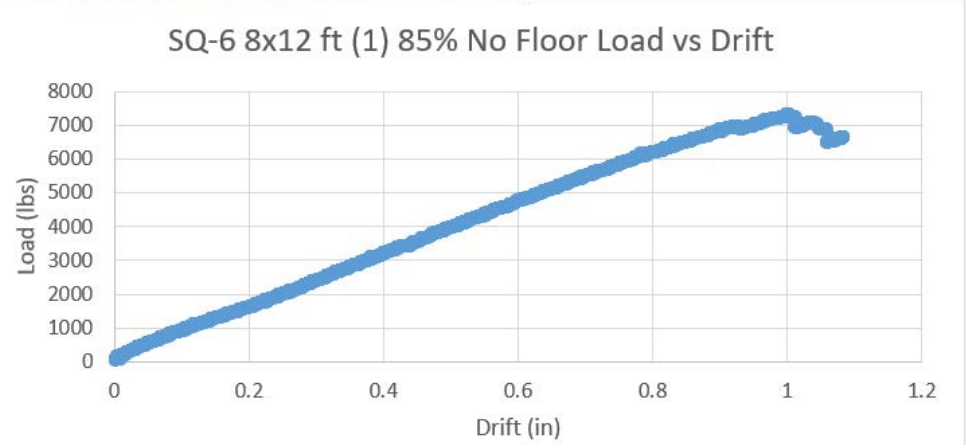

Figure B-18. Shear test of SPS Wall J

8’ x 12’ SPS shear wall test with 8’ x 7’ garage door opening size (opening height 85% of SPS wall’s 8-ft floor-to-ceiling height) resulting in full-height wall segments with an aspect ratio of 4:1 (height:width) where the segment height is measured from floor to ceiling. Failure mode was fracturing of both plies at the upper corner of opening at the uplift end of the wall due to tension stress concentration. This test was similar to that in Figure B-10, except without a floor system and the associated height of the SPS outer ply of OSB beneath the opening. This test simulated use of the SPS system for a garage opening with a slab floor.

Appendix C. 1-Ply and 2-Ply SPS Panel (1-1/8” OSB) Two-Point Bending Load Tests

Tests and data reported in this appendix were provided by Home Innovation Research Labs with additional analysis and explanatory information provided by Jay H. Crandell, PE of ARES Consulting. Test specimen construction drawings were provided by University of Minnesota.

C.1 SPS Bending Load Test Apparatus and Specimen Design

The following tests provided a basis for assessing the bending stiffness and composite action of 2-ply SPS wall panel constructions. It is noteworthy that the 2-ply tests had stiffness essentially identical in both directions of bending as should be expected.

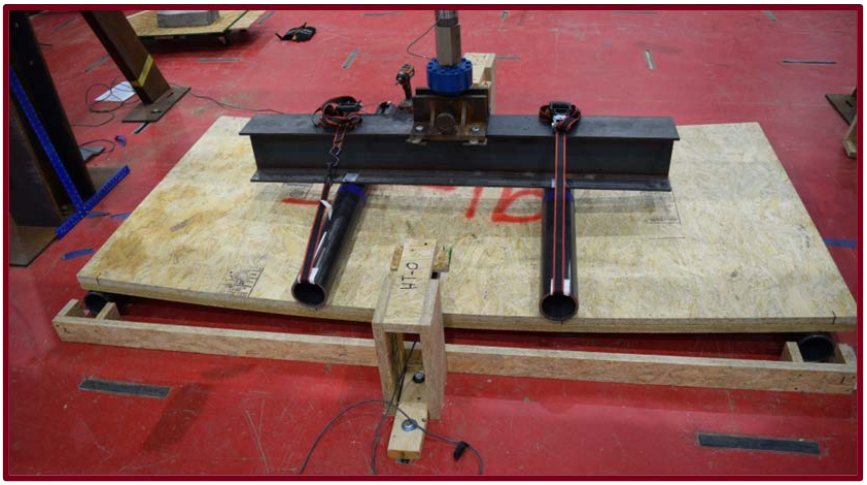

Figure C-1. Bending load test apparatus (for single and 2-ply)

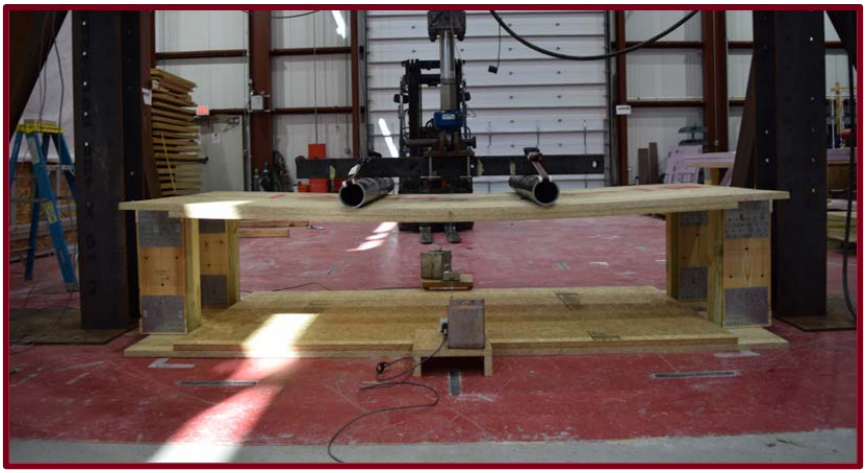

Figure C-2. Bending load test apparatus (inward bending with fixity)

Figure C-3. Bending load test apparatus (outward bending with fixity)

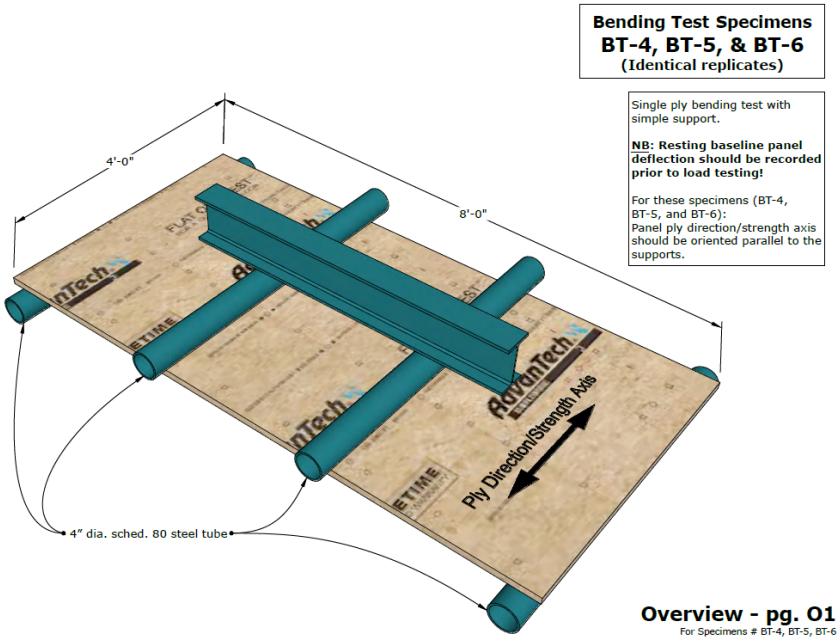

Figure C-4. Test specimen drawing for bending (1-ply)

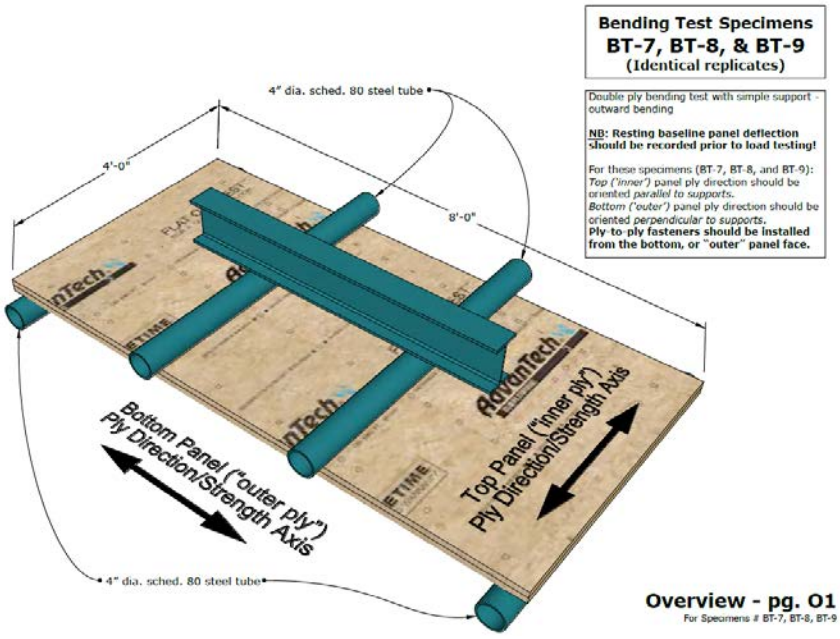

Figure C-5. Test specimen drawing for bending (2-ply)

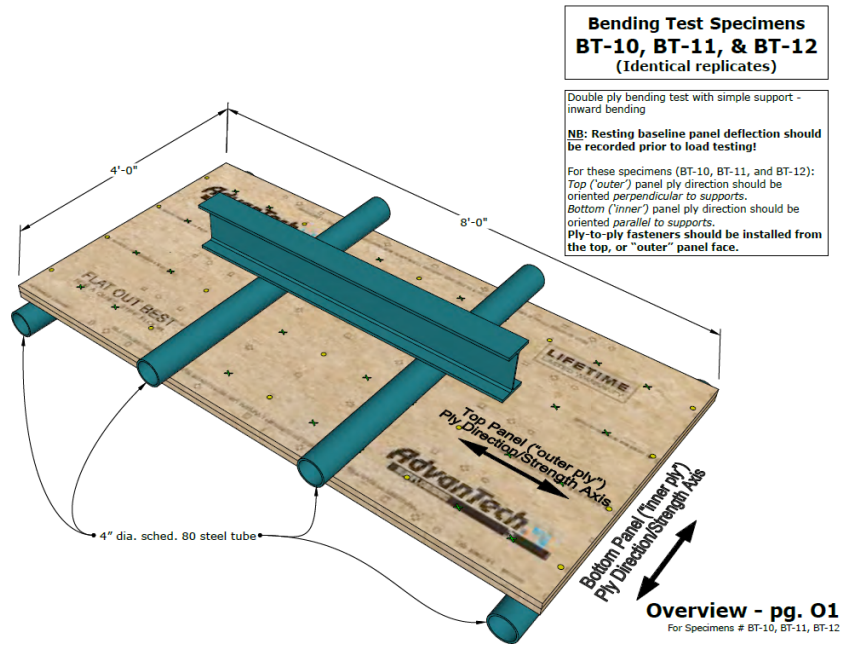

Figure C-6. Test specimen drawing for bending (2-ply—reversed)

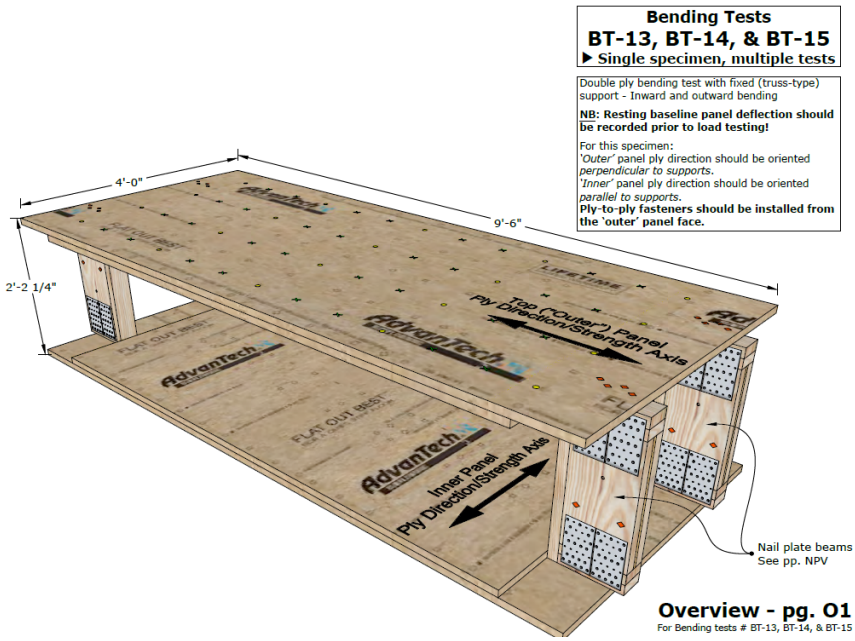

Figure C-7. Test specimen drawing for bending with fixity

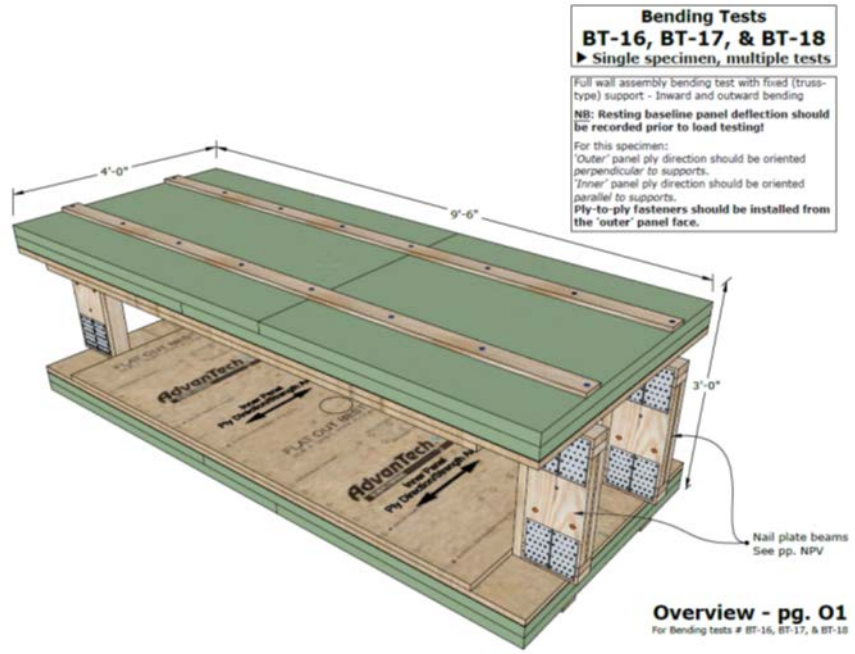

Figure C-8. Test specimen drawing for bending with fixity (including exterior foam and batten)

C.2 SPS Bending Load Test Specimens and Results

Table C-1. Two-Point Bending Load Results

	Test Name	Peak Load (lbs)	Avg Defl. @ Peak Load (in)	Load @ 0.5" Avg. Defl (lbs)	Avg Length (in)	Avg Width (in)	Avg. Panel Thickness (in)	Span (in)	Panel MOI (in^4)	EI Stiffness ($lb\text{-}in^2$)	MOE (psi)
1-ply Tests - Panel bending stress parallel to length of original panel	1-1	1249.8	3.249	112.1	95.979	47.875	1.144	90	5.97	5,353,269	896,218
	1-2	1313.4	3.265	142.5	95.875	47.938	1.144	90	5.98	5,479,467	916,149
	1-3	1303.6	3.276	114.4	95.979	47.875	1.144	90	5.97	5,542,601	927,915
									Average	**5,458,445**	**913,427**
									COV	**0.018**	**0.018**
1-ply Tests - Panel bending stress perpendicular to length of original panel	2-1	466.6	3.181	154.9	96.042	48.021	1.144	90	5.99	1,503,940	251,018
	2-2	501.6	3.208	140.6	96.000	47.833	1.144	90	5.97	1,724,662	288,986
	2-3	495.4	3.212	141.4	96.021	47.979	1.144	90	5.99	1,688,320	282,036
									Average	**1,638,974**	**274,013**
									COV	**0.072**	**0.074**
2-ply Tests - Outward Bending (outer panel in tension stress parallel to original panel length)	3-1	2148.0	3.156	130.8	95.979	48.000	1.144	90	n/a	9,824,602	
	3-2	2208.9	3.263	168.3	96.000	48.167	1.144	90	n/a	9,555,679	
	3-3	2103.7	3.235	138.2	95.875	48.104	1.144	90	n/a	9,296,855	
									Average	**9,559,045**	
									COV	**0.03**	
2-ply Tests - Inward Bending (inner panel in tension stress perpendicular to original panel length)	4-1	2024.0	3.160	149.1	95.875	47.875	1.144	90	n/a	9,119,085	
	4-2	2236.6	3.254	151.1	96.063	48.000	1.144	90	n/a	9,797,690	
	4-3	2072.9	3.236	121.8	95.938	48.125	1.144	90	n/a	9,227,818	
									Average	**9,381,531**	
									COV	**0.04**	
2-ply Tests - All (average for all six tests 3-1 through 4-3; both sets within 1% of average for all)	All (average)	2132.3	3.217	143.2	95.955	48.045	1.144	90	n/a	9,470,288	
									COV	**0.032**	

1. Avg. panel thickness is based on measurements during ASTM D3042 tests (see Appendix F)
2. Panel EI determined using beam equation to derive EI = $23PL^3/648xDefl$ where deflection is measured at center span, L=span, and P=1/2 applied load.
3. EI is determined for elastic range of deflection from 0.5" to a deflection of ~3" max with "peak load" reported.
4. Two-point load applied at third-points of span of simply supported beam.
5. Deflections are average of two panel edge measurements at center span.

C.3 Analysis of SPS Bending Load Tests

Figures C-9 and C-10 show the bending load deflection plots for test specimens 1-1 through 4-3.

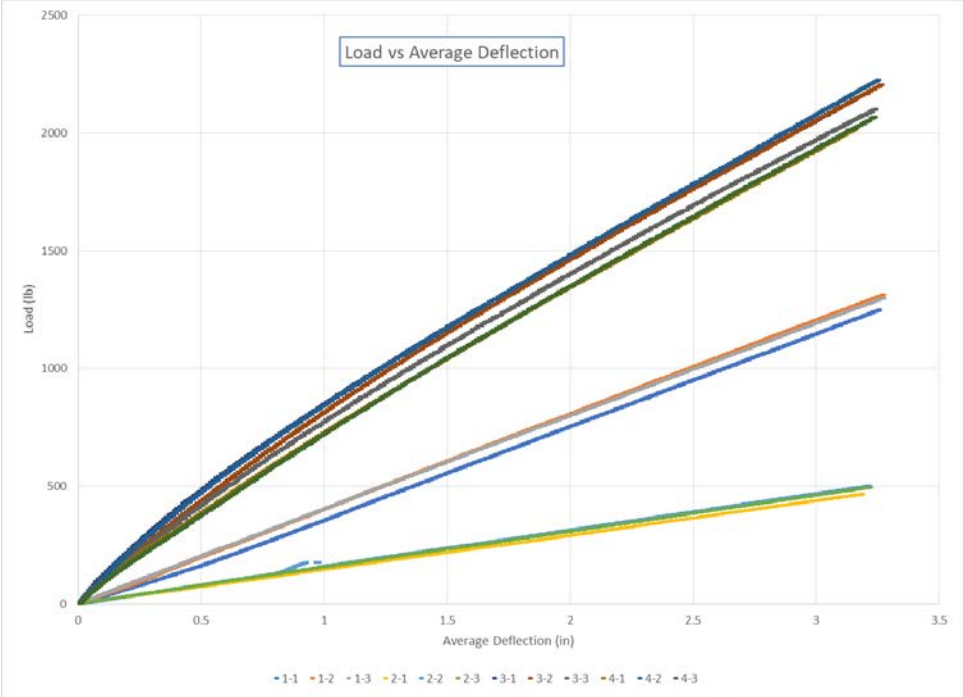

Figure C-9. Bending load deflection plots

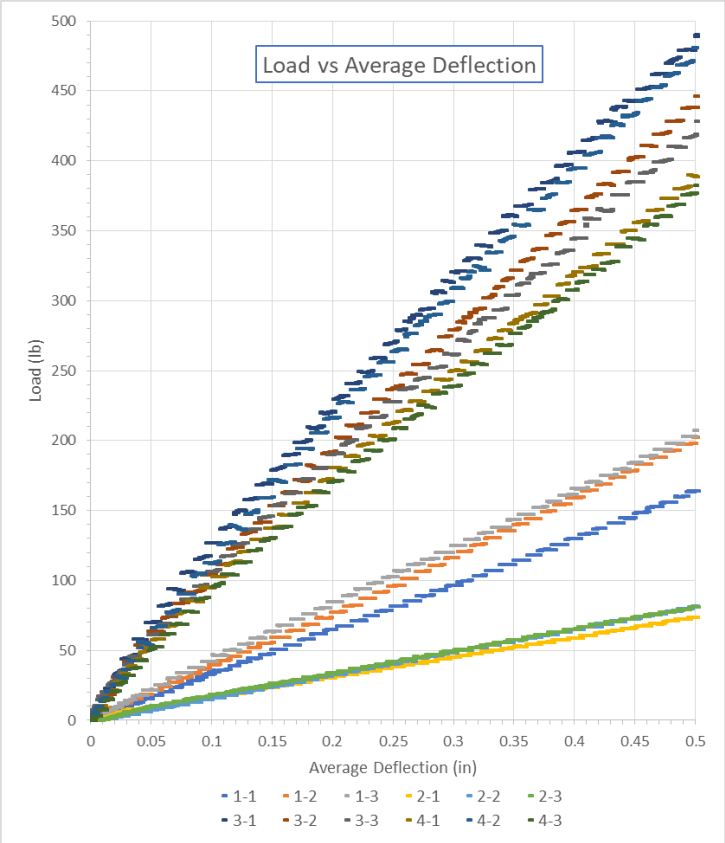

Figure C-10. Bending load deflection plots

For comparison with the above test data, the following idealized fully composite section properties for the 2-ply panels were determined using the transformed section method and using the MOE data from the 1-ply tests above. Also determined was an idealized non-composite section property where the two panels are assumed to behave completely independently as based also on the individual 1-ply panel tests. The analysis shows that the mechanical fastener lamination approach for 2-ply SPS panels results in about 15.6% of fully composite capability with regard to bending stiffness (EI).

Table C-2. Properties of the Specimens

Fully Composite Section Properties (EI and I) Using Transformed Section Method for 2-Ply Panel and Eperp and Epara Values from 1-Ply Test Results		
Eperp =	274,013	psi (average from manufacturer test data is 315,300 psi and ASTM D3042 Method C tests per Appendix F is 350,389 psi)
Epara =	913,427	psi (average from manufacturer test data is 800,080 psi and ASTM D3042 Method C tests per Appendix F is 822,077 psi)
b =	48.045	in (width of panels)
t_avg =	1.144	in (average thickness of panels, each ply)
Calculated Values		
n =	0.300	(ratio Eperp/Epara for section transformation to material with homogenous MOE = Epara)
b'-perp =	14.413	in (adjusted width of transformed panel with MOE = Eperp)
y_bar =	0.836	in (location of neutral axis of composite section from outer face of Epara panel)
MOI_trans =	24.392	in^4 (moment of inertia, I, of transformed section with MOE = Epara)
EI_fc =	22,280,088	$lb\text{-}in^2$ (stiffness of fully composite 2-ply panel)
Non-Composite 2-Ply Panel EI summing MOI x E for each Panel Bending Independently [i.e., $I\_nc = 1/12(b)(t\_avg)^3$ x (Epara) + $1/12(t\_avg)(t\_avg)^3$ x (Eperp)]		
I_nc = 7,117,979 in^4		
Non-Composite 2-Ply Panel EI (each panel bending independently)		
EI_perp	1,638,974	lb-in2 (avg from 1-ply test data above)
EI_para	5,458,445	lb-in2 (avg from 1-ply test data above)
EI_nc	7,097,420	lb-in2 (stiffness of non-composite 2-ply panel with individual panel tested EI)
% Composite =	**15.6%**	**percentage of full composite action realized where % Composite Action = [((tested avg EI 2-ply) - EI_nc) / (EI_fc - EI_nc)] x 100%**

The following results are for 2-ply SPS wall assembly bending tests (also using third point loading), but with restraint provided at the 2-ply panel ends by connection to floor/roof members as shown in the construction drawings and as would typically occur in an actual SPS building application. These tests provide a basis for assessing the degree of end-moment fixity provided by attachment of the SPS wall panels to floor and roof assemblies. These same specimens, because the bending tests were conducted within the elastic bending range, were also used to conduct axial load tests included in Appendix E.

Table C-3. Three-Point Bending Test Load Results

								Pinned End Assumed	Fixed End Assumed	
Test Name	**Load 1 (lbs)**	**Avg Defl. @ Load 1 (in)**	**Load 2 (lbs)**	**Avg Defl. @ Load 2 (in)**	**Nom. Panel Width (in)**	**Avg. Panel Thickness (in)**	**Span (in)**	**EI-Simple Stiffness (lb-in²)**	**EI-Fixed Stiffness (lb-in²)**	
SPS 2-ply panels tested "bare" as in tests 3-1 through 4-3										
13-1 Inward	260	0.2	1296	1.15	48	1.144	96	17,122,717	3,722,330	
14-1 Outward	292	0.21	1345	1.18	48	1.144	96	17,044,849	3,705,402	**Degree of End Fixity (%)**
							Average	**17,083,783**	**3,713,866**	**56.9%**
SPS 2-ply panels tested with added 4" thick foam sheathing and two 1x4 pine wood furring at 24"oc centered on face of panels										
16-1 Inward	411	0.2	1076	0.68	48	1.144	96	21,752,889	4,728,889	
17-1 Outward	322	0.2	1075	0.88	48	1.144	96	17,386,918	3,779,765	**Added Stiffness Factor**
							Average	**19,569,903**	**4,254,327**	**1.15**

1. Bending load was applied as two point loads at third points of span. Reported loads in table are total applied loads by UTM.
2. Degree of end fixity is unknown so EI is determined using a pinned end beam and a fixed end beam assumption to compare with the actual EI of 9,470,288 lb-in² from previously tested 2-ply panels using a simply supported beam with pinned ends (see bending test data and analysis for specimens 3-1 through 4-3). EI-simple= $23PL^3/648xDefl$ and EI-fixed= $5PL^3/648xDefl$ where deflection is measured at center span, L=span, and P=1/2 applied load. The calculated degree of end fixity indicates the extent to which the SPS wall panel assembly behaves as a fixed beam relative to that of a simply supported beam.
3. The “added stiffness” factor is the average ratio of EI determined for the bare panel tests (13-1 and 14-1) and the tests with added two layers of 2” thick foam sheathing and two 1x4 pine wood furring at 24”oc installed centered on face of panels. See construction drawings for attachment of furring. The foam sheathing was specified and labeled with a minimum compressive resistance of 15 psi. For the two tests (16-1 and 17-1) the added stiffness factor ranged from 1.02 to 1.27 with outward bending resulting in the lower added stiffness factor.

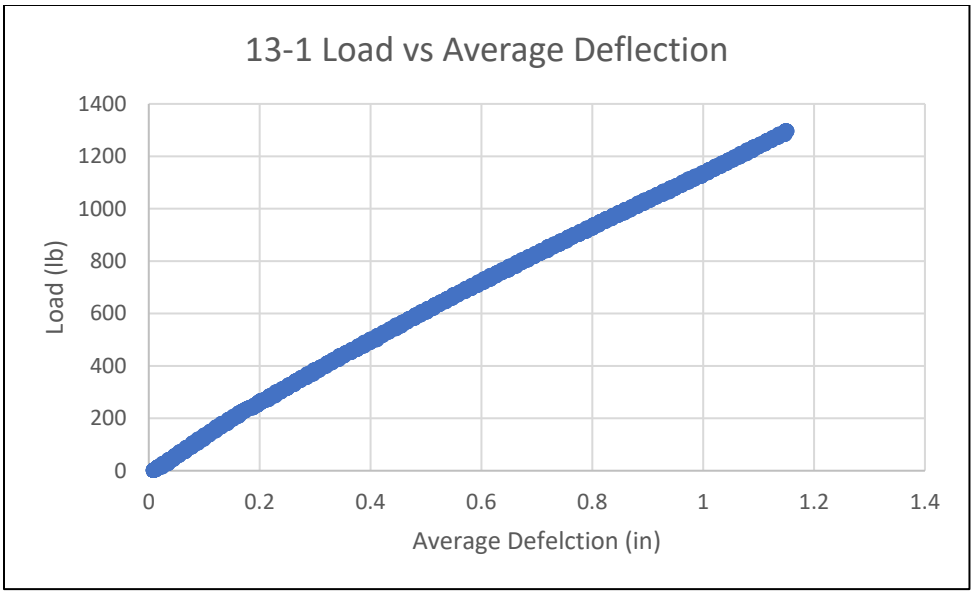

Figure C-11. Full 2-wall specimen no foam - inward bending

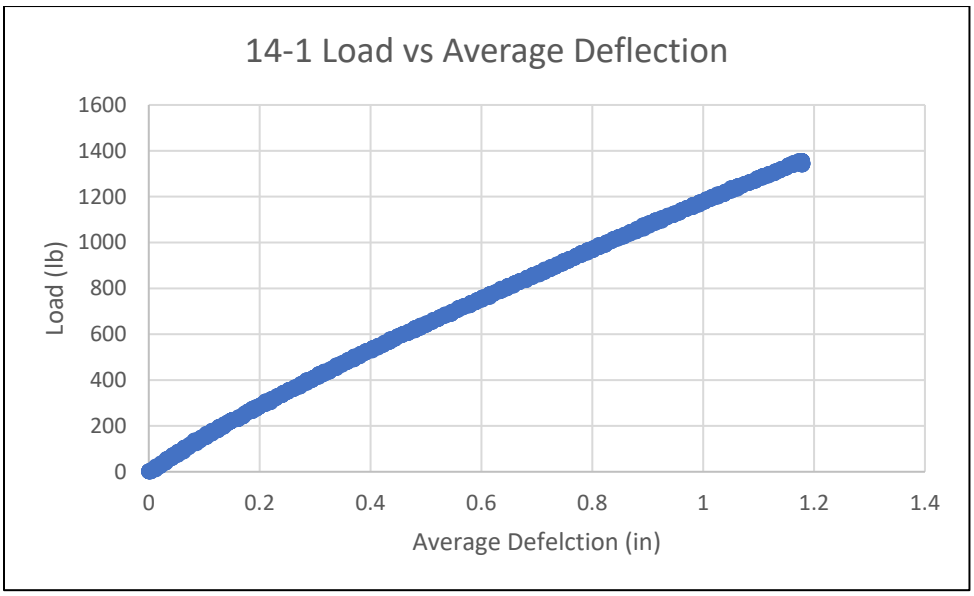

Figure C-12. Full 2-wall specimen no foam - outward bending

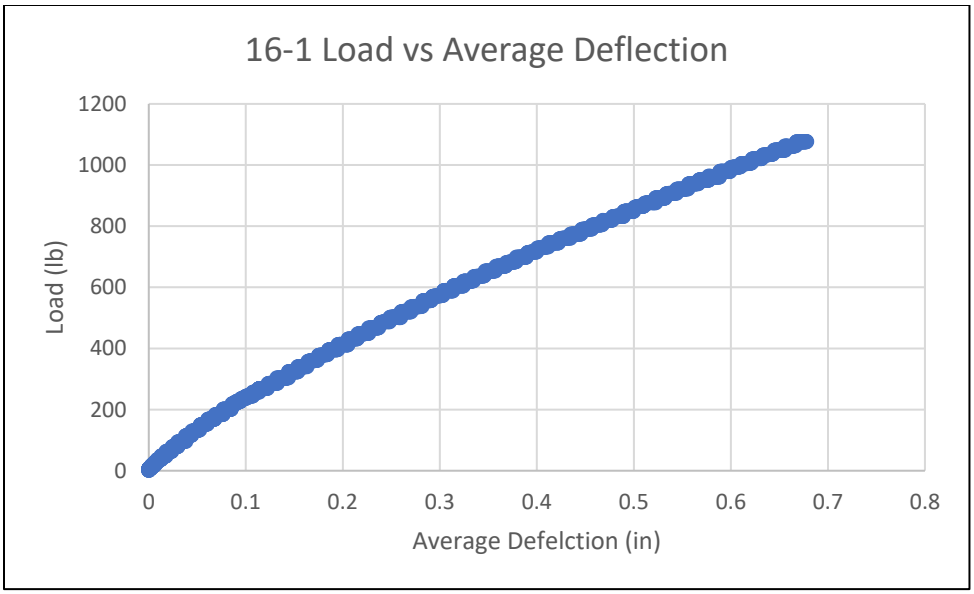

Figure C-13. Full 2-wall specimen with foam - inward bending

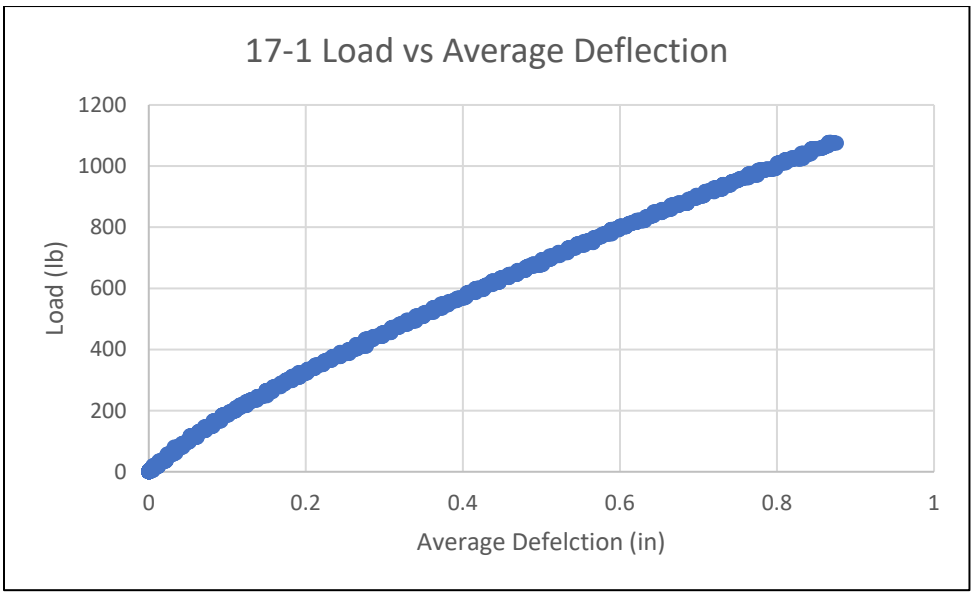

Figure C-14. Full 2-wall specimen with foam - outward bending

C.4 Selected Photos of Bending Load Tests

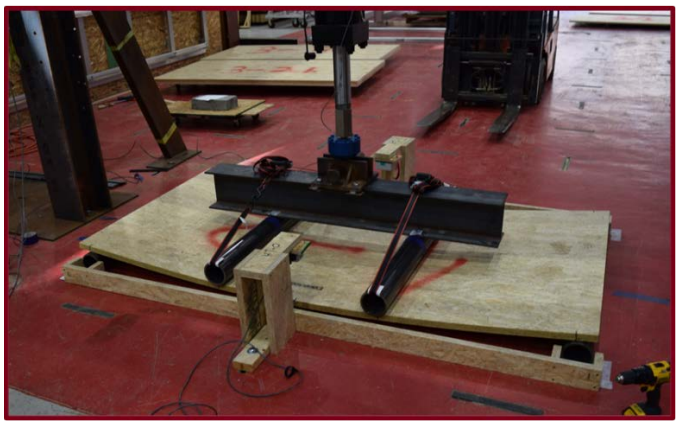

Figure C-15. 1-ply bending test (perpendicular to strength axis)

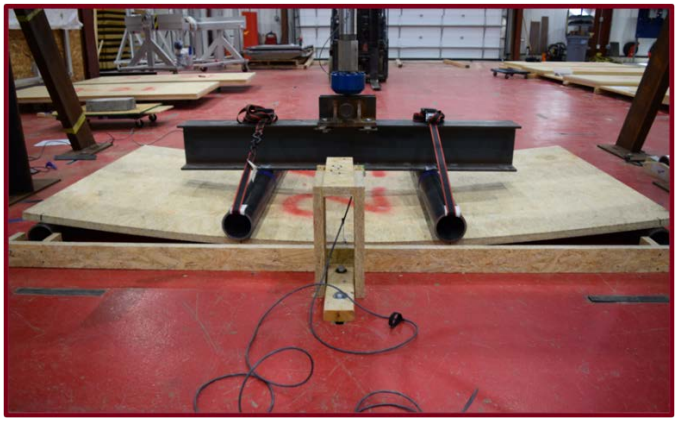

Figure C-16. 1-ply bending test (parallel to strength axis)

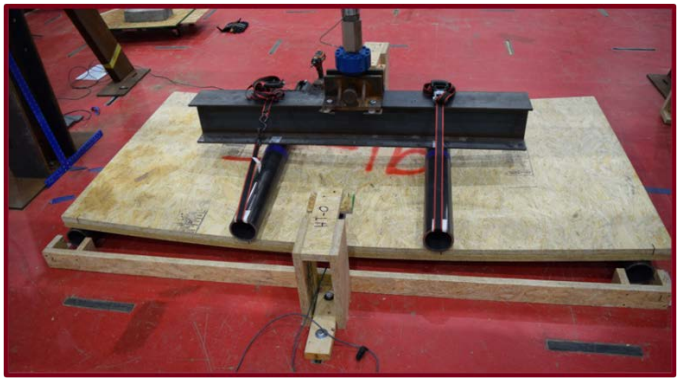

Figure C-17. 2-ply bending test (inward)

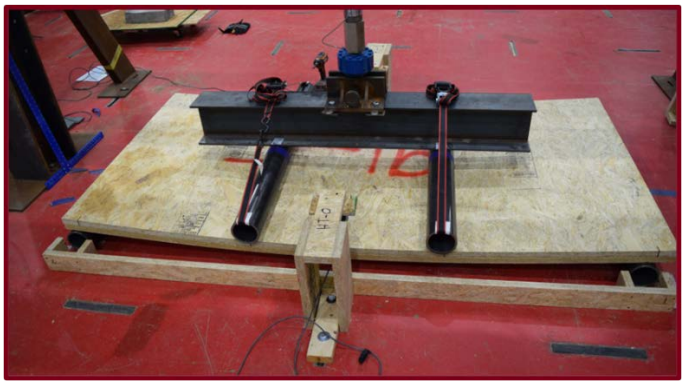

Figure C-18. 2-ply bending test (outward)

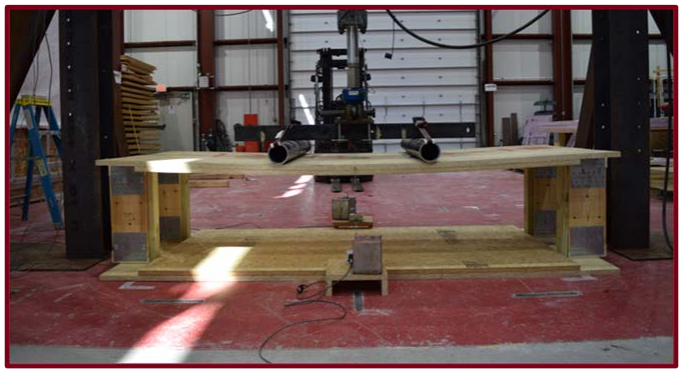

Figure C-19. SPS bending test w/ fixity (inward)

Figure C-20. SPS bending test w/ fixity (outward)

Appendix D. SPS Wall-Floor-Roof Assembly Axial Load (Compression Buckling) Test

Tests and data reported in this appendix were provided by Home Innovation Research Labs with additional explanatory information provided by Jay H. Crandell, PE of ARES Consulting. Test specimen construction drawings were provided by University of Minnesota.

D.1 Axial Load Test Apparatus and Specimen Designs

These two compressive buckling load tests use the same two paired SPS wall assembly specimens used for bending tests reported as SPS wall assembly panel test specimens 13-1/14-1 and 16-1/17-1 in Appendix D, noted here as specimens 15-1 and 18-1, respectively. For these paired SPS wall panel test specimens, end restraint is provided by attachment to floor/roof members at the top and bottom of the panels. One specimen had “bare” panels (specimen 15-1) and the other with 2” thick 15 psi foam sheathing and furring installed (specimen 18-1).

Figure D-1. Axial load test apparatus

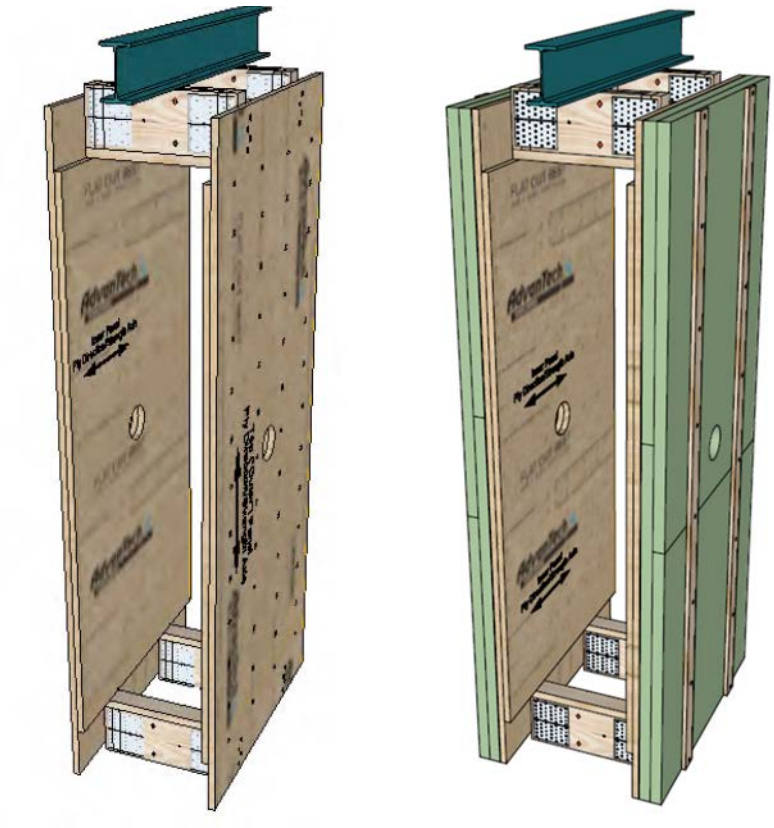

Figure D-2. Test specimen drawings for axial with fixity

D.2 Axial Load Test Specimens and Results

Peak load shown is total load on two SPS 2-ply wall panels (load applied to center span of two floor/roof beams attached at each end to top of each SPS wall assembly; the two floor/roof beams were located at 1 ft in from each edge of SPS panels at 2 ft o.c. spacing). Failure mode was panel buckling in both tests. Test 15-1 was terminated before reaching peak load to remove the string potentiometers (deflection measurement) and restarted with final test-to-maximum peak load noted as test “15-1 SP Removed.”

Table D-1. Axial Load Test Results

Test Name	Peak Load
15-1	30460.36
15-1 SP removed	37051.97
18-1 with foam	38014.15

D.3 Analysis of Axial Load Tests

The following load-deflection plot is only for specimen 18-1. As indicated in the table above for specimen 15-1, the test was terminated before reaching the maximum peak load capacity and restarted with the string potentiometer removed so there are no deflection measurements at center span of the wall. The lateral buckling deflections of the two paired SPS wall panels in Test 18-1 as shown in the plot below are the average of two measurements at center span at each outer edge of both 2-ply SPS wall panels.

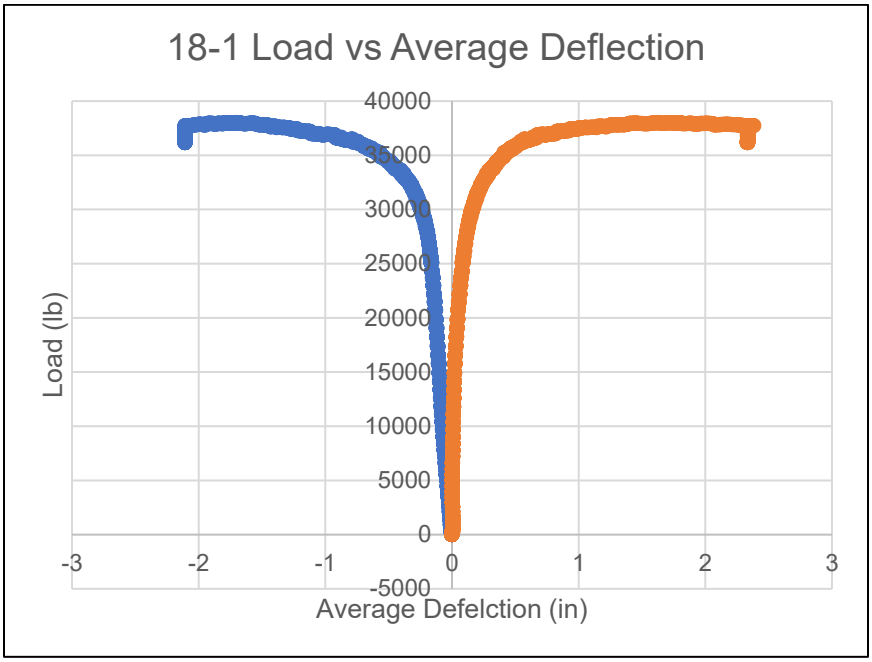

Figure D-3. Full 2-wall specimen with foam – compression

D.4 Selected Photos of Axial Load Tests

Figure D-4. Axial test (SPS only)

Figure D-5. Axial test (SPS with exterior foam and batten)

Appendix E. OSB Material Property Tests

E.1 OSB Testing

The OSB panel that was used throughout this project was fully characterized to get key material properties. This was done to both confirm earlier assumptions and validate the experimental test results. These detailed material properties will allow the findings of this study to be extended using accepted engineering calculations and techniques.

E.2.1 OSB Bending Flatwise, Two-Point Loading

Table E-1. ASTM D4761, Section 8 (simply supported beam with two-point loading at third points of span)

Bending Test Specimen Size and Span			**Specimen Section Properties (calculated)**		
Thickness (d)	1.125	inches	Moment of Inertia ($I = bd^3 / 12$):	1.42	in^4
Width (b)	12	inches	Section Modulus ($S = bd^2 / 6$):	2.53	in^3
Length	18	inches	E-apparent = $5PL^3$ / [324 (I)(disp at load points)] where P is total UTM load and displacement at load points "disp at load points" is the UTM cross head deflection less initial slack displacement of ~0.014" as shown in data and L-D plots		
Span (L):	16	inches			
NOTE: Center span deflection (not measured) is 2.3 times greater than deflection at the load points.					

Table E-2. Specimen Bending Stress Perpendicular to Length of Sampled OSB Panels (cross panel bending)

Specimen ID	UTM Total Max Load (lbs)	UTM Crosshead Deflection at Max Load (in)	Max Moment (in-lbs)	Modulus of Rupture (MOR) $F_{b,ult,perp}$ (psi)	UTM Total Load at 0.25" Crosshead Defl. (lbs)	E-apparent*	Notes
B1	2360	0.511	6293	2486	1060	199398	Speed was 0.1 in/min
B2	2400	0.456	6400	2528	1125	211625	Speed changed to 0.2 in/min.
B3	2330	0.488	6213	2455	1160	218209	Broke outside of rollers.
B4	2420	0.505	6453	2549	1120	210685	
B5	2220	0.454	5920	2339	1090	205042	
B6	2140	0.464	5707	2254	1100	206923	
B7	2130	0.439	5680	2244	1060	199398	
B8	2270	0.453	6053	2391	1120	210685	
B9	2040	0.462	5440	2149	1010	189993	
B10	2150	0.475	5733	2265	1040	195636	Broke slightly outside rollers on one side.
Average	2246	0.4707		**2366**		**204759**	
std-dev				136.7		8575.2	
COV				**0.058**		**0.042**	
5th-%ile				2142		190696	

Table E-3. Specimen Bending Stress Parallel to Length of Sampled OSB Panels (along panel bending)

Perpendicular Bending	UTM Total Max Load (lbs)	UTM Crosshead Deflection at Max Load (in)	Max Moment (in-lbs)	Modulus of Rupture (MOR) $F_{b,ult,para}$ (psi)	UTM Total Load at 0.25" Crosshead Defl. (lbs)	E-apparent*	Notes
D1	4610	0.41	12293	4857	2385	448646	
D2	4190	0.428	11173	4414	2115	397856	
D3	4360	0.424	11627	4593	2205	414786	
D4	4320	0.422	11520	4551	2185	411024	
D5	3560	0.384	9493	3750	1995	375282	
D6	4000	0.394	10667	4214	2190	411964	Broke under roller on one side.
D7	4870	0.445	12987	5131	2310	434538	
D8	5210	0.45	13893	5489	2465	463695	
D9	4630	0.455	12347	4878	2165	407261	
D10	4310	0.427	11493	4541	2110	396915	At each edge the break is between rollers; crack propagated outside of rollers in center of specimen.
Average	4406	0.4239		**4642**		**416197**	
std-dev				484.0		26149.3	
COV				0.104		**0.063**	
5th-%ile				3848		373312	

* These E-apparent measurements are low estimates of a pure bending MOE because of the low span-to-depth ratio used for the small specimen property testing. Therefore, these E-apparent values should not be used for design purposes.

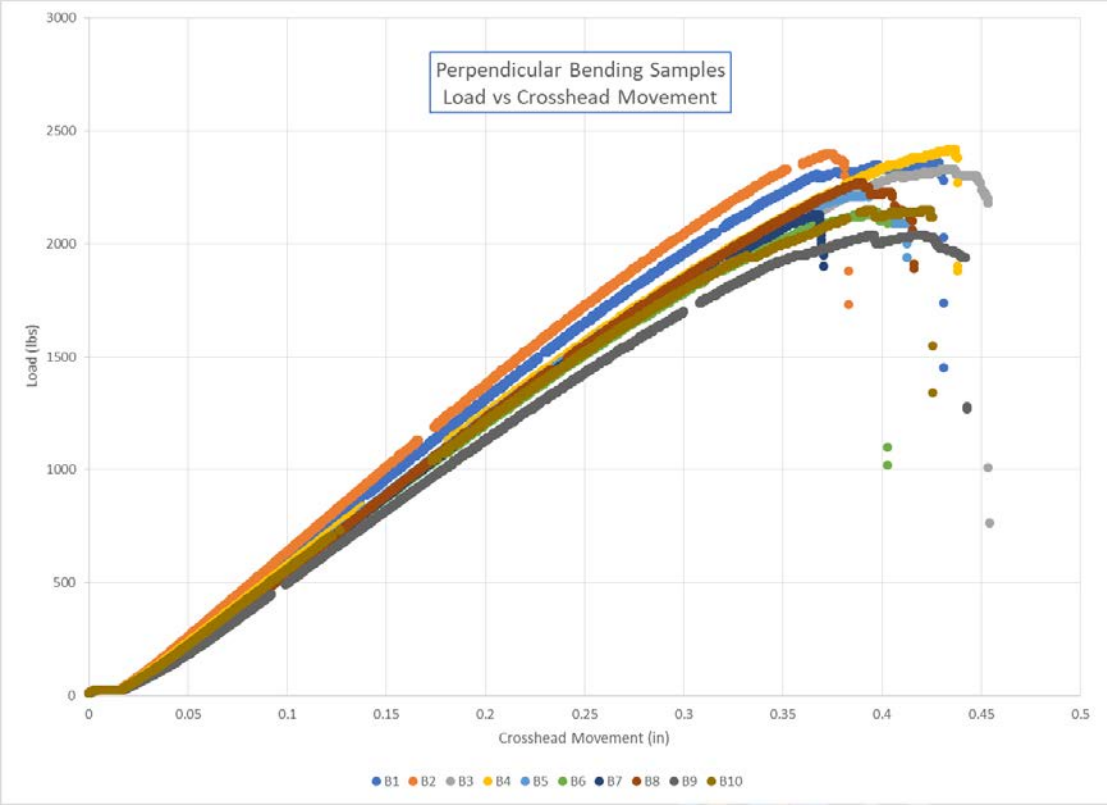

Figure E-1. Perpendicular bending samples

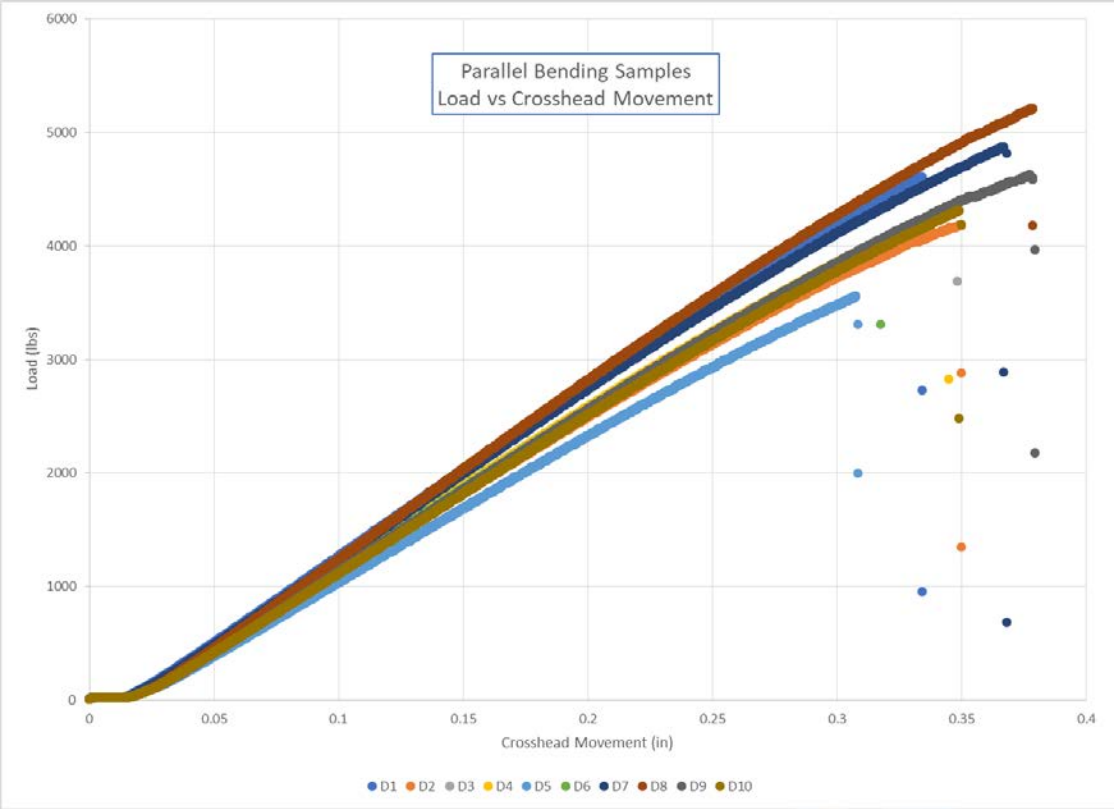

Figure E-2. Parallel bending samples

E.2.2 Pure Moment Test (Bending Flexure)

Table E-4. ASTM D3042 Method C - Pure Bending Tests

Unit		Lbs	inches											lbs-in^2/ft		lbs-in/ft
	Specimen	Weight	Thickness 1	Thickness 2	Thickness 3	Thickness 4	Thickness 5	Length 1	Length 2	Length 3	Height 1	Height 2	Height 3	EI (Para)	EI (Perp)	Bending Strength
Parallel (done first)	1	64.725	1.14	1.14	1.14	1.142	1.14	48.125	48.125	48.125	48	48	48	1133216	550455	567.3
	2	63.735	1.144	1.138	1.14	1.142	1.145	48	48	48.125	48	48	48	1202260	546867	564.4
	3	63.31	1.15	1.148	1.15	1.155	1.154	48.125	48.25	48.125	48	48	47.875	1301083	540781	563.7
	4	62.48	1.14	1.146	1.144	1.144	1.14	48.125	48	48	48	48	48	1272192	500352	565.6
	5	62.475	1.146	1.145	1.151	1.14	1.142	48	48	48	48	48	47.875	1254140	517993	565.5
Perpendicular (done first)	6	63.29	1.142	1.142	1.14	1.146	1.145	48	48	48	47.875	47.875	47.875	1348479	507977	569.9
	7	63.265	1.142	1.143	1.144	1.146	1.146	48	48	48	48	48	48	1196241	507149	564.2
	8	62.845	1.147	1.145	1.151	1.153	1.15	48	48	48	48	48	48	1239998	508260	568.0
	9	62.86	1.14	1.14	1.142	1.141	1.137	48	48	48	48	48	47.938	1113511	535681	564.4
	10	63.285	1.141	1.146	1.147	1.146	1.148	48	48	48	48	48	48	1260137	534339	566.9

NOTE: Bending strength is irrelevant because specimens were tests for EI only in elastic range and not tested to failure (bending rupture).

Table E-5. Analysis of Above Test Data to Determine E(parallel) and E(perp)

	Specimen	Avg Thickness (inches)	MOI per ft (in^4/ft)	E(para) (psi)	E(perp) (psi)
Parallel (done first)	1	1.140	1.48	764084	371151
	2	1.142	1.49	807659	367377
	3	1.151	1.53	852366	354277
	4	1.143	1.49	852397	335247
	5	1.145	1.50	835905	345251
Perpendicular (done first)	6	1.143	1.49	903037	340177
	7	1.144	1.50	798570	338555
	8	1.149	1.52	817023	334888
	9	1.140	1.48	751588	361569
	10	1.146	1.50	838144	355400
	Average	1.144	Average	822077	350389
			Std Dev	44670	13438
			COV	0.054	0.038
			5th-%ile	748819	328351

E.2.3 ASTM D4761 Section 10 – Axial Strength in Compression

Table E-6. ASTM D4761, Section 10 Specimen Size

Compression Test Specimen Size		
Thickness	1.125	inches
Width	6	inches
Height	12	inches

Table E-7. Horizontal (Cross-Panel Width) Compressive Stress

Horizontal (Cross-Panel Width) Compression	Max Load (lbs)	Deflection at Max Load (in)	Max Comp. Stress, Fc,ult (psi)	Notes
A1	21187	0.1603	3139	Crack and delamination approx. 1/3 from top of specimen. Loaded at 0.5 in/min
A2	17426	0.1732	2582	Crushing at bottom of specimen. Loaded at 0.5 in/min
A3	17153	0.166	2541	Crushing at bottom of specimen. Loaded at 0.25 in/min
A4	14794	0.1085	2192	Crushing and delamination at top. Loaded at 0.2 in/min
A5	17155	0.1442	2541	Delamination and face crack. Loaded at 0.2 in/min
A6	19011	0.1485	2816	Delamination on one edge with buckling. Loaded at 0.2 in/min
A7	20733	0.1605	3072	Delamination and buckling in center. Loaded at 0.2 in/min
A8	18957	0.1497	2808	Delamination at top. Loaded at 0.2 in/min
A9	18858	0.1587	2794	Delamination, mostly on one edge. Loaded at 0.2 in/min
A10	18607	0.1492	2757	Delamination approx. 1/3 from bottom. Loaded at 0.2 in/min
Average	18388.1	0.1519	**2724**	
Std Dev			276	
COV			0.10	
5th-%ile			2272	

Table E-8. Vertical (Along Panel Length) Compressive Stress

Vertical (Along Panel Length) Compression	Max Load (lbs)	Deflection at Max Load (in)	Max Comp. Stress, Fc,ult (psi)	Notes
C1	23028	0.1215	3412	Delamination on bottom. Loaded at 0.2 in/min
C2	23212	0.1188	3439	Delamination on bottom. Loaded at 0.2 in/min
C3	20232	0.0965	2997	Delamination at center. Loaded at 0.15 in/min
C4	20236	0.12	2998	Delamination on one edge. Loaded at 0.1 in/min
C5	20368	0.1482	3017	Delamination at bottom. Loaded at 0.1 in/min
C6	19381	0.1155	2871	Delamination and face crack approx. 1/3 from top. Loaded at 0.1.in/min
C7	22362	0.1285	3313	Delamination and face crack at center. Loaded at 0.1 in/min
C8	18636	0.1165	2761	Delamination at bottom with face crack in center. Loaded at 0.1 in/min
C9	20770	0.1215	3077	Very minor delamination on one edge with face crack. Loaded at 0.1 in/min
C10	17647	0.1045	2614	Very minor delamination on one edge with face crack. Loaded at 0.1 in/min
Average	20587	0.1192	**3050**	
Std Dev			272	
COV			0.09	
5th-%ile			2605	

The compression moduli shown in Table E-9 was estimated from elastic range of load-displacement plots.

Table E-9. Compression Modulus Data and Analysis

	Displ 1 (in)	Load 1 (lbs)	Stress 1 (psi)	Displ 2 (in)	Load 2 (lbs)	Stress 2 (psi)	Strain (in/in)	Stress Delta (psi)	Comp Mod (psi)
Horizontal (cross panel width)	0.0295	2987	442.5	0.0862	13512	2001.8	0.00473	1559.3	**330002**
Vertical (along panel length)	0.038	4754	704.3	0.082	15558	2304.9	0.00367	1600.6	**436525**

Estimated from Central Tendency (Average) of compression load-deflection plots

Compression Modulus (perpendicular) = 330,002 psi

Compression Modulus (parallel) = 436,525 psi

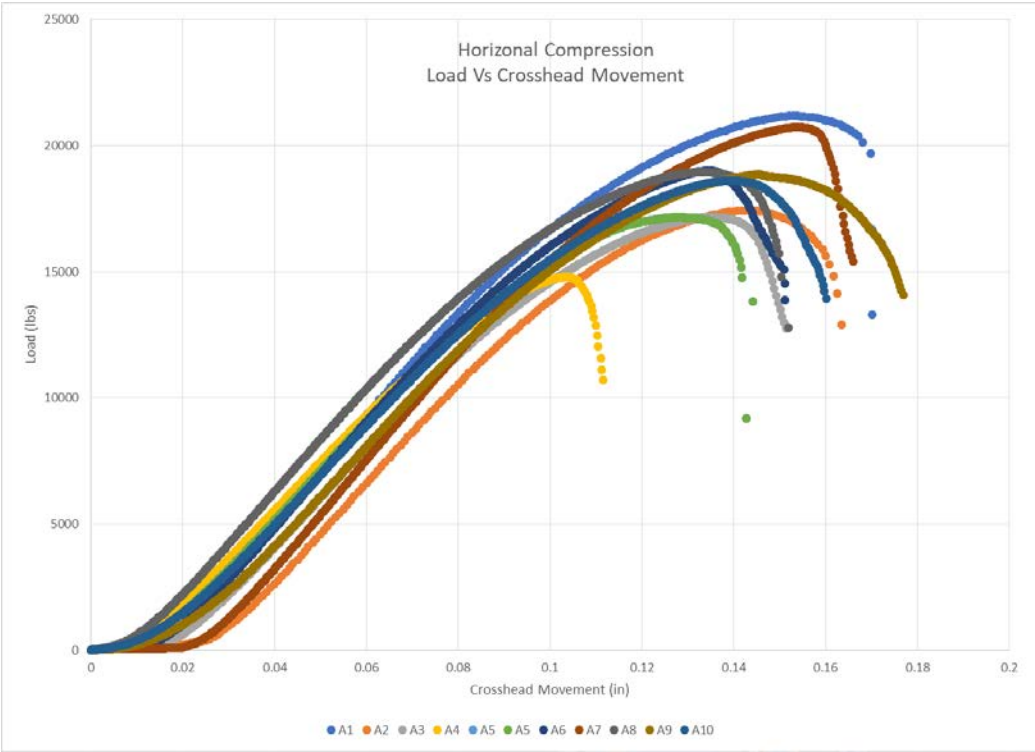

Figure E-3. Horizontal compression

Compression load vs. UTM crosshead displacement for 1-1/8" x 6" x 12" OSB specimens with weak axis oriented in the vertical direction (perpendicular to the original OSB panel length).

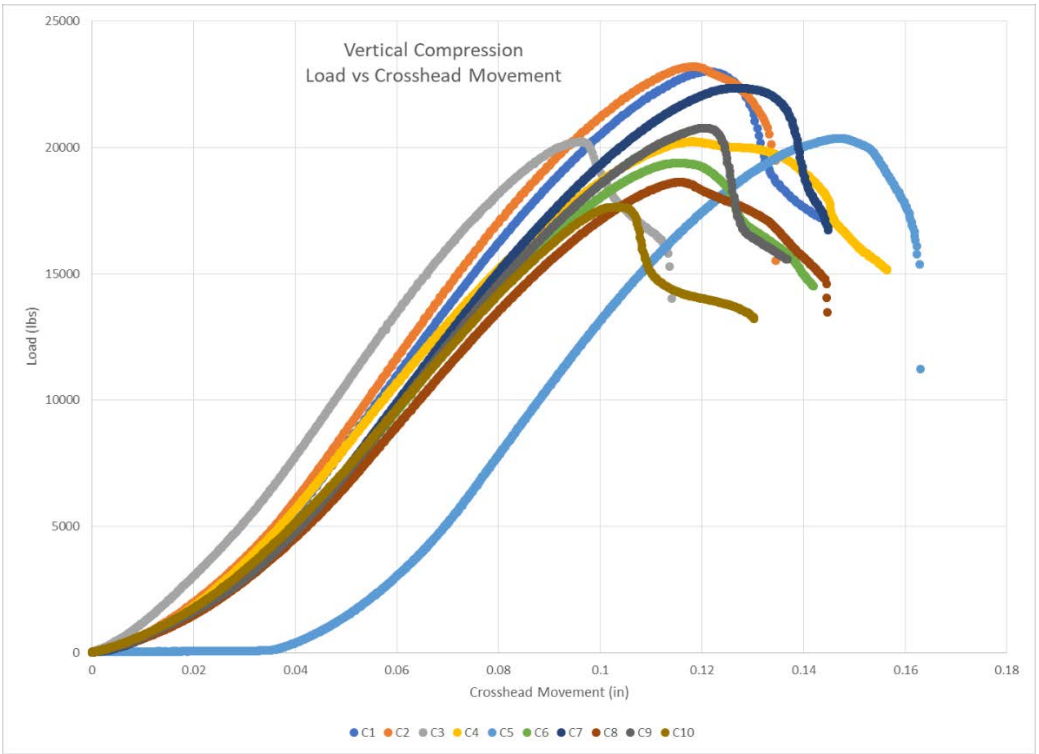

Figure E-4. Vertical compression

Compression load vs. UTM crosshead displacement for 1-1/8” x 6” x 12” OSB specimens with strong axis oriented in the vertical direction (parallel to the original OSB panel length).

Appendix F. Industry Professional Survey Form

F.1 Screening Tool for Builder/Contractor Recruiting

Which of the following services does your company offer? ***Select all that apply only.***

New SFD home builder primarily, but also offers remodeling services (Builder/remodeler)	1	Minimum of 2, but up to 4 is OK
New SFD home builder primary occupation, does not offer remodeling services	2	Up to 6, no minimum
Architect of Single-Family Homes (must be primary occupation)	3	Minimum of 2
Plumbing installations in new homes (must be primary occupation) 50/50? Should be More than half of work in new homes—exclude 50/50	4	Only 1
Electrical installations in new homes (must be primary occupation) 50/50? Should be More than half of work in new homes—exclude 50/50	5	Only 1
None of the above	6	Terminate

(if Builder) Approximately how many single-family detached homes has your company built in the past 12 months? Write-in_____(half 5 to 20 homes/year, half >20 homes/year)

(if Builder or Architect) Which of the following phrases best describes how much influence you have when it comes to selecting structural framing for new single-family detached homes your company builds or designs? ***Select one response.***

I am the sole decision maker of which materials we use	1	
I have authority to specify which materials we use	2	
I strongly influence the selection of which materials we use	3	
I have some influence over the selection of which materials we use	4	
I have no influence over the selection of which materials we use	5	Terminate

In which state(s) has your construction-related company operated in the past 12 months? If multiple states, have respondent choose where they operate locally or where they have most experience. (Recruiter—mix from across U.S. regions, approximately 3 in South, 1 in Northeast, 2 in Midwest, and 2 in West. None from Florida)

(if Plumber & Electrician) How much of your current work involves plumbing or electrical installations in new homes: All of it; More than half of it; Some of it (terminate it); None of it (terminate).

About how many years of experience do you have in installing electrical/plumbing in new homes? _____ (must be 10 years +)

(if Builder) What are the primary issues you have with lumber wall framing (2 x 4 or 2 x 6) in the construction of new homes?

[RECRUITER: Any answer is acceptable, but participant should be engaged in the subject to some degree, articulate, and willing to share insights. Please record verbatim response on the recruitment grid]

_____
_____
_____

(if Plumbers & Electricians) What are the primary issues you have with running plumbing/electrical in homes with solid masonry or concrete walls, either above-grade or basements?

[RECRUITER: Any answer is acceptable, but participant should be engaged in the subject to some degree, articulate, and willing to share insights. Please record verbatim response on the recruitment grid]

_____
_____
_____

(if Builders) Are you willing to participate in a 45-minute videoconference call on the viability of a new structural wall material made of engineered wood that would be a substitute for traditional?

[PROGRAMMER: Invite to participate, and let them know there will be follow up shortly]

Yes ___
No ____

(if Plumber & electrician) Are you willing to participate in a 45-minute videoconference call to discuss the pros and cons of running electrical / plumbing in a new solid (no wall cavity) engineered wood wall system? **[PROGRAMMER: Invite to participate, and let them know there will be follow up shortly]**

Yes ___
No ____

F.2 Discussion Guide for SPS

F.2.1. Overview and Objectives

Home Innovation Research Labs will conduct 12 individual in-depth interviews via Zoom among a mix of New Home Builders/remodelers, Architects, Plumbers, and Electricians to obtain feedback on the use and adoption of the Solid Panel Structure (SPS) engineered wood frameless wall system. These one-on-one interviews will be conducted prior to July 24, 2023 via Zoom Webinar meetings.

The primary objectives of the qualitative research are to:

- Understand construction considerations when using the SPS exterior walls, particularly in respect to erecting the SPS walls, home design and features, insulating walls, appropriate exterior/interior finishes, and running plumbing and electrical utilities.
- Assess how the approach compares to traditional construction methods to identify impact on labor, other materials used, construction schedules and overall productivity.
- Identify opportunities and challenges that may influence the adoption of SPS wall system in single-family residential construction.

F.2.2 Interview/Discussion Outline

Welcome and Introduction (3 Minutes)

- Moderator introduction points:

- Qualitative market researcher for Home Innovation Research Labs
- HI is conducting this research to better understand constructability considerations for homes built using a new “frameless” wall system made of engineered wood, called the Solid Panel Structural (SPS) wall system.
- Moderator is a neutral party whose role is to facilitate the discussion and gather market insights; there’s no right or wrong answer.
- Interviews are being conducted with 12 construction professionals nationwide to get various perspectives. Participant identities will be kept confidential.

- The goal of our discussion today is to get a real-world perspective of construction considerations when building a home using the new SPS wall system and to identify potential needs for different or new solutions for its successful market introduction.

Current Practices (10 Minutes)

- Tell me a little bit about the type of homes you build/design (e.g. style, square footage, Entry/Move-up/Luxury). How many homes do you typically build/design in a year?
- Let’s discuss your method for structural above-grade walls.
  - What materials primarily?
  - Who assembles?
  - Why do you use that approach?
- Does speed of construction play a role in your choice? Please explain. Probe if needed…
  - Cost of materials?
  - Labor availability, or labor skills?
  - Choice of interior & exterior finishes?
  - Choice of insulation?
- What aspects of your current exterior wall assembly can present challenges, or would like to see better solutions?
- Are you considering any other exterior wall systems as an alternative to what you currently use? Why? Probe as needed, but only into categories they show an interest.
  - Above-grade poured concrete or block?

- Light gauge steel?
- Panelization—any materials?
- Modular homes?
- Mass timber/cross-laminated timber walls?

Overview of SPS Wall System and Initial Impressions (20 Minutes)

- Here is a brief description of the SPS wall system to give you an overview of the approach and some context for what we will be discussing today. (Share a 5-page presentation of the SPS system on the screen, then 3-minute time lapsed video)
  - What are your initial impressions of this method of construction?
- In what ways might this approach offer advantages over traditional construction approaches? (probe as needed on the following)
- What do you see as being the greatest benefit?
- In what ways could this potentially save time?
- Would this result in any cost savings? How?
- How about home performance?
- What are some of the key questions or concerns you have about the system and process? What are some drawbacks obvious to you?

Construction Considerations (20 Minutes)

- We will now discuss areas where the new wall system might affect construction, and what changes you might need to make if adopting the SPS wall system.

Construction Process

- How would this affect how you build homes? (probe on positive and negative changes)
  - Contractor selection?
  - Sequencing of construction phases?
  - On-site equipment?
  - Do you see potential labor savings or productivity benefits?
- Would your current framing crew be a good candidate for installing the SPS wall system? Why or why not?

- Would you want this to be supplied to you as a turn-key framing system (materials and installation are provided by the same supplier & contractor?) Why or why not?

Foundation, Floor, and Roof Connections

- What specifically would be important for a builder or architect to understand about how SPS walls connect to the foundation? Please explain why.
- What other foundation-related considerations would this system give rise to?
- What about connecting floors? Roofs? What generally needs to be considered for each?
- Do you see any drawbacks or potential challenges? (If yes) In what ways?

WRB, Windows and Doors

- The primary weather-resistant layer on the SPS wall is usually the exterior side of the structural wall panel and under the exterior insulation. Does this present opportunities or problems with constructability or building performance?
- Because the thickness of the SPS structural wall material is 2-1/4," a typical door jamb will extend beyond the surface of the interior wall. How would you adjust for this? (note that one developer "boxes out" at the jamb to provide the necessary depth).

Insulation

- Overall, what do you think about using foam board insulation on the exterior of the home?
- What specifically would be important for a builder or architect to understand about how to insulate these frameless walls? Please explain
- Do you see any drawbacks or potential challenges? (If yes) In what ways?
- What about connecting windows and doors with exterior foam insulation?
- If issues are brought up, how would you solve these issues? What would you do differently?
    - For example, would you try to insulate another way, or use another insulating strategy?

Electrical and Plumbing

- The initial homes built with the SPS frameless wall panel used baseboard raceways to accommodate electrical wiring and plugs. What are your impressions of that? (probe on perceived benefit vs drawback)

- What would you do about electrical outlet and switch boxes?
- Do you foresee any problems with running plumbing or HVAC utilities in a home with the SPS frameless wall? (e.g. sequencing of construction stages, sealing exterior penetrations, location of outlet/switch planning, etc.)
- How would this process impact sequencing of construction steps? Would this approach offer any labor savings or productivity benefits?

Exterior & Interior Finish

- What are your thoughts on how the SPS frameless wall would impact the exterior finishes you choose? Do you have issues or concerns with how your finish materials would perform in the long term? Ease of installation?
- How about interior finishes? Any concerns or limitations? Please explain

Overall Home Quality, Durability, and Energy Performance

- Thinking about everything overall, what are some reasons a builder would want to use this exterior wall system?
- What are the primary reasons a builder would not want to use it?
- What recommendations do you have to the manufacturer?

Building Code Approvals, Inspections

- Any benefits or concerns here?

Close (2 Minutes)

- Any last comments or thoughts on how the SPS wall system could be improved?
- Thank you again for taking the time to come out here today and provide your insights. It is greatly appreciated!

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