



United States
Department of
Agriculture

Forest Service

Forest
Products
Laboratory

General
Technical
Report
FPL-GTR-222



Evaluation of the High-Heel Roof-to-Wall Connection with Extended OSB Wall Sheathing

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Abstract

A recently completed testing project conducted to evaluate optimized structural roof-to-wall attachment solutions demonstrated the effectiveness of wood structural panels in restraining high-heel trusses against rotation. This study was designed to further evaluate the performance of OSB wall sheathing panels extended over the high-heel truss in resisting combined uplift and shear forces. Five full-size roof-wall assemblies were constructed with extended OSB wall sheathing and each was tested under a different loading combination. Our test results indicate that using extended wood structural panel wall sheathing as the primary connection element at the roof-to-wall interface of energy trusses can provide a continuous load path in both the shear and uplift directions and can be considered a viable option for residential construction in most areas of the country.

Keywords: Lateral performance, high-heel truss, OSB wall sheathing, roof system, roof-to-wall connection

Acknowledgments

This project was funded through a cooperative research agreement between the National Association of Home Builders (NAHB) Research Center and the USDA Forest Service Forest Products Laboratory. This research was supported in part by funds provided by Norbord, Inc.

April 2013

DeRenzi, Andrew; Kochkin, Vladimir; Wang, Xiping. 2013. Evaluation of the high-heel roof-to-wall connection with extended OSB wall sheathing. General Technical Report FPL-GTR-222. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 11 p.

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Contents

Introduction.....	1
Objectives	1
Methods and Materials.....	1
General.....	1
Specimen Construction	1
Test Setup and Protocol.....	3
Results.....	6
Summary and Conclusions	10
References.....	10
Appendix—Summary of the Shear Load Versus Horizontal Displacement Curves for Tests B Through E	11

Conversion Table

English unit	Conversion factor	SI unit
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile (m)	1.6093	kilometer (km)
pound (lb)	0.45359	kilogram (kg)

Evaluation of the High-Heel Roof-to-Wall Connection with Extended OSB Wall Sheathing

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Introduction

This study responds to the new requirements for roof-to-wall connections in the 2012 International Residential Code (IRC 2012). The new IRC provisions specify complex details for attachment of rafters and trusses to the supporting walls. These new requirements, which apply to high-heel energy trusses even in the low-wind areas, are labor intensive and add cost to construction of light-frame wood buildings. A recently completed testing project (NAHBRC 2011; DeRenzis and others 2012) conducted to evaluate optimized structural roof-to-wall attachment solutions demonstrated the effectiveness of wood structural panels in restraining high heel (i.e., energy) trusses against rotation.

The current study was designed to evaluate the performance of oriented strandboard (OSB) wall sheathing panels extended over the high heel truss in resisting combined uplift and shear forces. No additional connecting hardware was installed to investigate the construction efficiencies in using extended wall sheathing to perform multiple structural functions: transfer of roof shear load, restraining high heel trusses from rotation, and resisting uplift forces. Testing was conducted on full-size roof specimens under combined uplift and shear loading. The structural limitations of this type of roof-to-wall connection were established. This method for connecting the roof to the walls complements the recently codified approach of providing a continuous combined uplift and shear load path using wood structural panels. In addition to the structural functions, the extended OSB serves as an insulation stop or baffle for the attic insulation at the heel.

The results of this study are expected to further expand prescriptive construction solutions optimized for structural and energy performance and constructability.

Objectives

The specific objective of this study is to develop an uplift–shear capacity interaction curve for extended wood

structural panel wall sheathing used as the primary connecting element at the roof-to-wall interface.

Methods and Materials

General

Testing was conducted at the National Association of Home Builders (NAHB) Research Center Laboratory Facility located in Upper Marlboro, Maryland. All specimens were constructed in the laboratory and all construction materials were purchased from local suppliers. The Norbord Tall Wall/Windstorm OSB product used for the wall sheathing was donated by Norbord Inc., Toronto, Canada.

Table 1 provides a test matrix summarizing the uplift–shear loading combinations. A total of five laboratory tests of a roof–wall assembly with extended OSB wall sheathing were conducted, each under a different loading combination. Test A (100% uplift, no shear load) and Test E (100% shear load, no uplift load) were conducted to anchor the combined loading interaction curve. The uplift load levels by percentage of total uplift capacity were selected to provide system performance at realistic load combinations (Tests B, C, and D). For typical house geometries and based on the IRC wind load coefficients, combined loading scenarios for residential construction are typically above a 2:1 uplift to shear loading ratio. Therefore, two of the three combined loading tests used the loading ratio above 2:1.

Specimen Construction

Each specimen was constructed with five 24-ft span wood trusses with a 1 $\frac{1}{4}$ -in. heel height, spaced at 24 in. on center. The overall size of the full roof system was 8 ft deep by 24 ft wide with additional 16-in. long overhangs on each side. Trusses were supported at the heel by 4-ft high by 8-ft long light-frame wood walls. The supporting walls were constructed with 24-in. on-center framing and sheathed on the exterior with 4-ft wide Norbord Tall Wall/Windstorm OSB panels extending up past the top plate of the wall to

Table 1—Test matrix

Test	Uplift load (as percentage of peak uplift capacity)	Uplift to shear ratio (assuming linear relationship)	Shear load	Purpose
A	100	NA	None	Establish peak uplift load capacity of the specimen
B	85	5.7:1	Max load at 85% uplift	Develop uplift versus shear load capacity interaction relationship at various combined loading ratios
C	70	2.3:1	Max load at 70% uplift	
D	40	0.7:1	Max load at 40% uplift	
E	0	NA	Max load at 0% uplift	

**Figure 1. Test specimen and set-up.**

capture the energy truss heel. Figure 1 shows a photo of a specimen and set-up.

Tables 2 and 3 provide summaries of materials and methods used in construction of the specimens and the fastening schedule for the wall sheathing, respectively. Figures 2 and 3 show the typical test specimen and fastening detail, respectively.

The Tall Wall OSB sheathing was fastened to the truss heel with five 8d common nails (2½ in. by 0.131 in.) (Fig. 3 and Table 3). This nailing schedule is based on the observed performance of OSB sheathing under lateral load only in Phase I testing where a total of three nails were used. For the current study, two additional nails were specified for added uplift resistance, for a total of five nails per truss heel (with a maximum of one of the five nails installed into the end grain of the bottom chord of the truss). Trusses were connected to the top plate with two toe-nails. No other connection hardware was installed.

Sheathing was attached to the wall framing (Fig. 3 and Table 3) in general accordance with the provisions approved by American Wood Council for the use of wood structural panels to resist combined uplift and shear (Coats and Douglas 2011). These provisions specify additional nails along

the top and bottom plates of a shear wall to resist the uplift component in a combined loading scenario. One exception in the tested specimen attachment details was the 4-in. on-center nail spacing at the top plate (increased from the 3-in. spacing specified in the AWC provisions) on account of the additional roof-to-wall load path through the sheathing-to-truss heel fasteners.

The supporting walls were anchored to the laboratory strong floor using ½-in. bolts at 16 in. on center with 3-in. by 3-in. by 0.229-in. square plate washers to prevent cross-grain bending failure of the bottom plates and Simpson HD hold downs at each wall's end.

The 7/16-in.-thick OSB roof sheathing was installed perpendicular to the truss top chord members with a staggered panel layout. Metal sheathing clips were installed on the unblocked edges of each panel at 24 in. on center between the framing members. A 2-in.-wide roof vent was provided at the ridge (1 in. each side of the ridge) such that bearing of panel edges did not occur during testing.

The ceiling gypsum panels were installed perpendicular to the truss bottom chord members and the first rows of fasteners was located approximately 8 in. from the supporting walls (i.e., floating edges) in accordance with the Gypsum Association's Application and Finishing of Gypsum Panel Products (GA-216-2010, Gypsum Association 2010). A single 2x boundary member and 2x nailing member were installed at the front and back trusses to simulate the presence of supporting gable-end walls by providing the bearing surface for the exterior edges of the gypsum as the gypsum panels rotate under the shear loading. Figure 4 shows a schematic of this boundary detail.

Because the objective of the study is to evaluate the performance of the wall sheathing to energy truss heel connection, gypsum panels were attached to bottom truss chords using screws at 8 in. on center to simulate the upper bound capacity of residential ceiling diaphragms. The ceiling diaphragm of the Test E specimen (100% shear load, no uplift load) was further reinforced with additional bracing of the truss bottom chord members to force the failure at the energy truss heel connection. All interior gypsum panel joints for all specimens were taped and mudded; however, no

Table 2—Construction of specimens

Materials	Methods
Roof dimensions	24-ft roof span (plus 1-ft, 4-in. overhang on each end) 8 ft deep (a total of 5 trusses)
Roof pitch	7:12
Roof framing members	Metal plate connected wood trusses fabricated with No. 2 SPF lumber; Heel heights 15¼ in.
Truss spacing	24 in. on center
Truss-to-wall plate connection	Trusses toe-nailed to top plate of wall with two 16d box (3¼-in. by 0.131-in.) nails
Fascia board	1- by 6-in. nominal lumber face-nailed to each truss end with two 8d common (2½-in. by 0.131-in.) nails
Roof sheathing materials	7/16-in-thick OSB sheathing installed perpendicular to framing member with steel edge clips and unblocked edges parallel to the ridge
Roof sheathing fasteners	8d common (2½-in. by 0.131-in.) at 6 in. on center on panel perimeter and 12 in. on center in the panel field
Ceiling material	1/2-in-thick gypsum panels installed perpendicular to truss bottom chord members, joints in the panel field taped and mudded
Ceiling fasteners	1-5/8-in. Type W drywall screws at 8 in. on center with first rows of fasteners 8 in. in from side walls (i.e., floating edges)
Supporting wall dimensions	8 ft long by 4 ft in height
Supporting wall framing (including top plates)	2 by 4 nominal SPF STUD or No. 2 grade lumber
Supporting wall sheathing	7/16-in-thick Norbord Tall Wall/Windstorm OSB sheathing attached with 8d common (2½-in. by 0.131-in.) nails

Table 3—Wall sheathing fastening schedule

Location/Connection	Fastener schedule
Extended wall sheathing to energy truss heel	Face-nailed with five (5) 8d common (2½ in. by 0.131 in.)
Wall sheathing to vertical wall framing (Nailing for shear)	8d common (2½ in. by 0.131 in.) at 6 in. on center around perimeter, 12 in. on center in the field
Wall sheathing to top plate (Increased nailing for additional uplift load)	8d common (2½ in. by 0.131 in.) at 4 in. on center along top plate
Wall sheathing to bottom plate (Increased nailing for additional uplift load)	8d common (2½ in. by 0.131 in.) at 3 in. on center along top plate

finishing was done at the interface of the ceiling and the bearing walls.

Test Setup and Protocol

Lateral load was applied to the specimen using the same methodology developed and used during Phase I testing of this project. Specifically, load was applied through permanent truss bracing (2 by 6 nominal Southern Pine, No. 2 Grade lumber) attached at the center vertical web member of each truss approximately 2 ft, 3 in. up from the bottom. The intent of using a pair of typical permanent truss braces was to minimize the restraints imposed on the specimen by the loading apparatus by applying the load through members that are typically present in truss roof assemblies.

Figure 5 provides the American Society of Civil Engineers (ASCE) 7 lateral wind load profile in the direction parallel to the ridge. Wind pressures are transferred to the roof and ceiling diaphragms through the framing of the gable-end wall and wall below the roof. Figure 6 illustrates the tributary area associated with each of the diaphragms, including the additional load into the ceiling diaphragm from the wall framing below. The location of the lateral loading brace was chosen to most closely replicate this ratio of forces in the roof and ceiling diaphragms by a wind load. Refer to DeRenzis and others (2012) for a more in-depth explanation of the distribution of forces and derivation of the loading methodology.

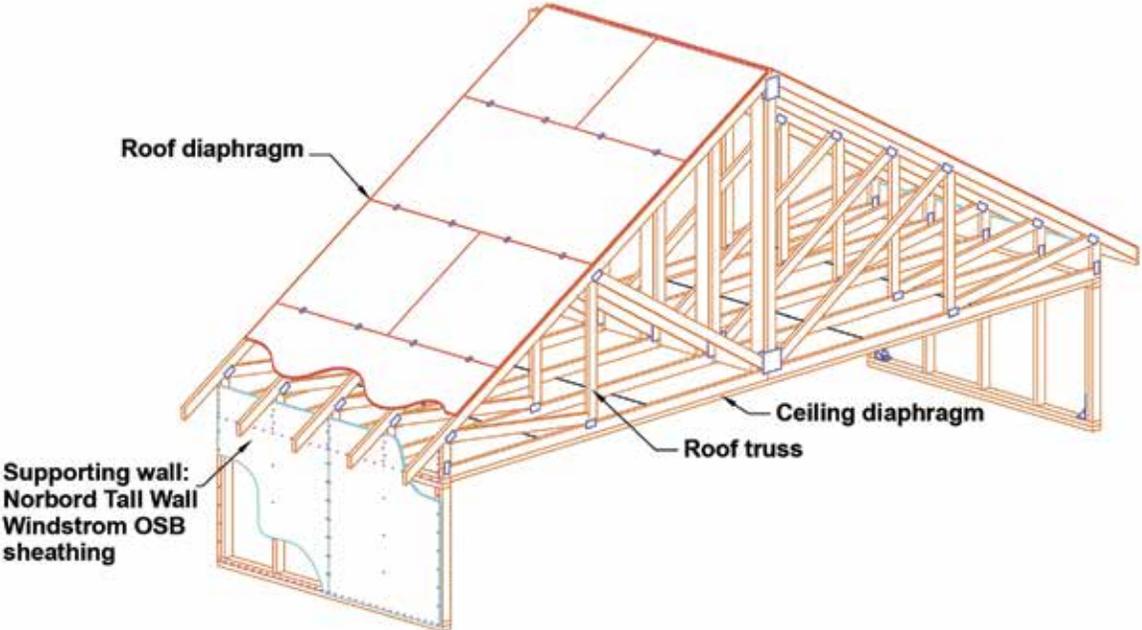


Figure 2. Specimen construction.

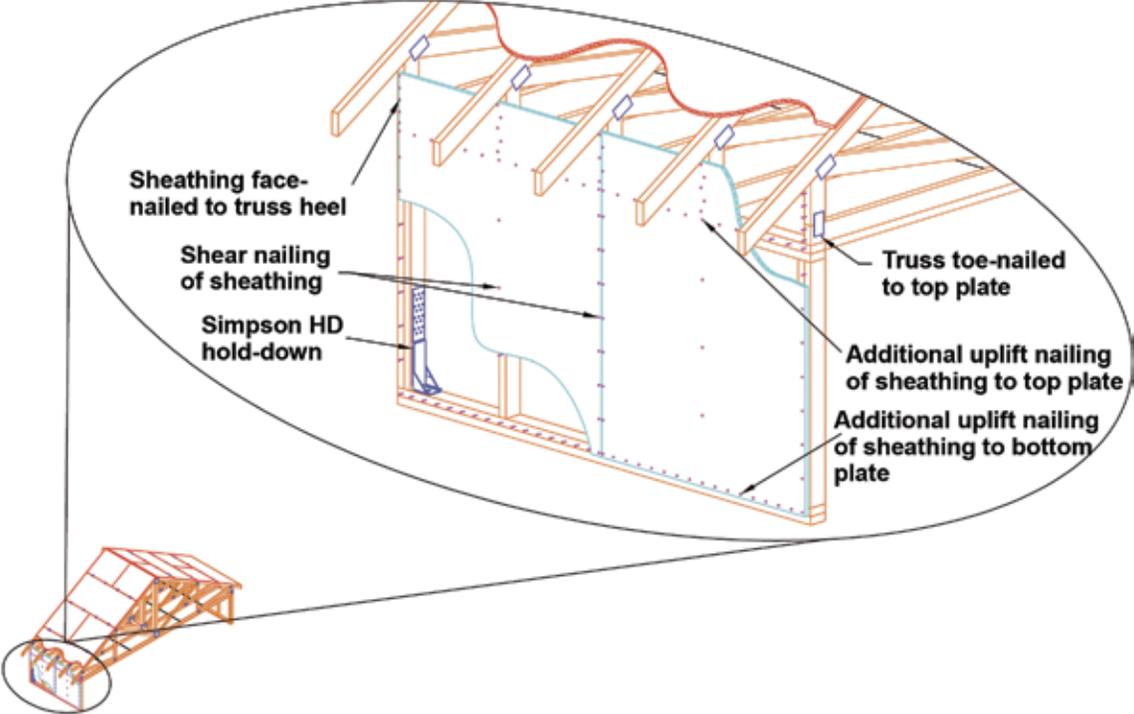


Figure 3. Tall Wall OSB sheathing connection detail.

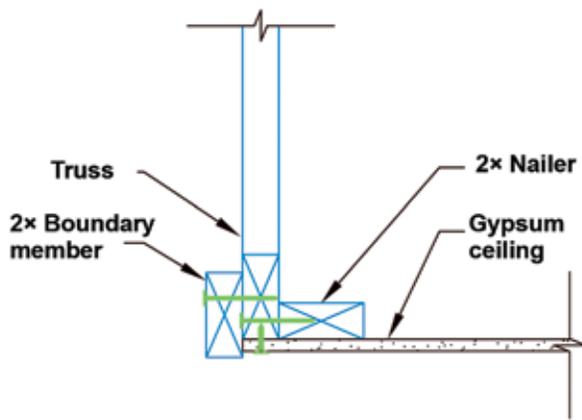


Figure 4. Ceiling diaphragm boundary detail at front and back trusses.

Each center vertical truss web member was reinforced with a double 2 by 8 vertical member to prevent weak-axis bending failure of the web. Each permanent bracing member was attached to the vertical reinforcing member with a single 4½-in. by ½-in. lag bolt to provide sufficient load transfer with minimal rotational restraint.

Figure 7 shows a schematic of the test set-up including the specimen, loading apparatus, and instrumentation and Figure 8 provides photos of the uplift–shear loading apparatus.

Uplift loading was applied uniformly to the ceiling diaphragm through a pressurized rectangular air bag. The air bag was constructed of flexible ethylene propylene diene monomer (EPDM) rubber and placed on a support structure underneath the test specimen (Figs. 8 and 9). The support structure also isolated the air bag from the test specimen walls to prevent any restraint of the walls caused by friction with the bag. The top of the air bag was isolated from the gypsum ceiling diaphragm by a translatable bearing surface constructed of a series of bearing plate rollers sandwiched

between two layers of OSB; the rollers allowed the specimen to move independently of the air bag in the lateral direction under combined loading. Figure 9 provides a detail of the translatable bearing surface.

The lateral load was applied in tension using a computer-controlled hydraulic cylinder mounted to a steel reaction frame. The reaction frame was attached to the laboratory structural floor. Uplift loading via inflation of the air bag was achieved using a computer-controlled pressure load actuator (PLA).

For Test A, uplift loading was applied continuously at a rate targeting failure to occur at approximately 5 minutes. For all subsequent combined loading tests, uplift load was ramped up to a value determined in accordance with Table 1 as a percentage of the peak uplift pressure achieved in Test A and then held constant throughout the remainder of the test. After the target uplift load was achieved, lateral load was ramped up monotonically at a constant displacement rate of 0.06 in. per minute to allow for sufficient visual observations throughout the test. Testing was continued until failure, defined as a 20% drop from the peak lateral load.

Several displacements were measured using electronic linear motion position transducers (LMPTs), including the following:

- Lateral displacement of the top of the heel (TOH) on the first/front truss (T1) at both ends
- Lateral displacement at the top of the supporting walls (TOW)
- Lateral displacement of the roof ridge
- Lateral displacement of the ceiling diaphragm
- Lateral slip at the bottom of the supporting walls
- Uplift at the end stud of the supporting walls, and

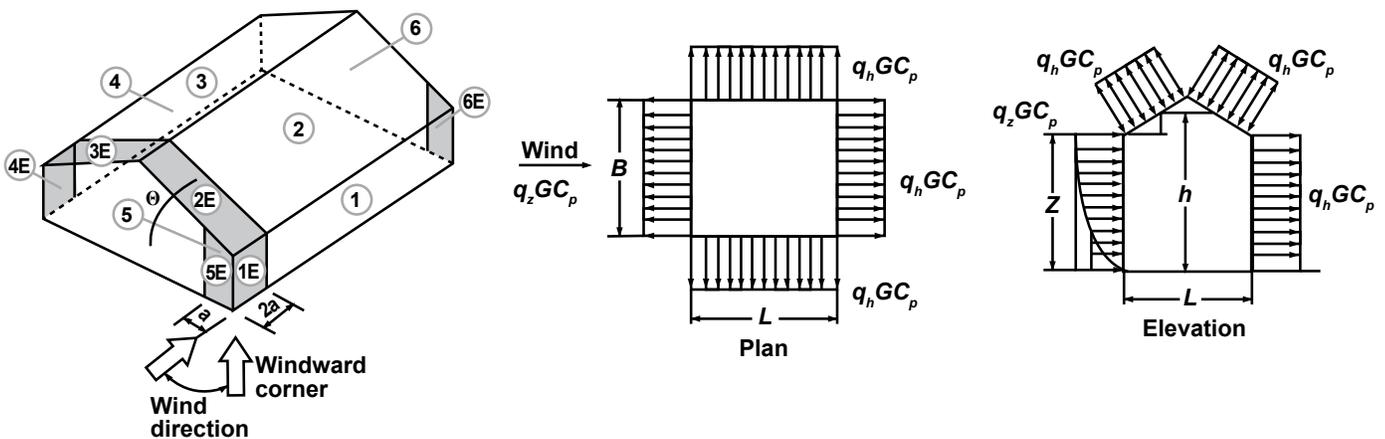


Figure 5. ASCE 7 wind-loading profile.

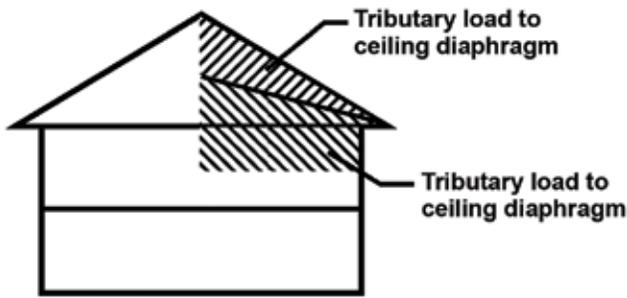


Figure 6. Tributary area of wind loads.

- Compression at the end stud of the supporting walls
- Relative uplift between the supporting wall and the heel of the first/front truss at both ends
- Relative uplift between the supporting wall and the heel of the last/back truss at both ends

Lateral loading was measured using an electronic load cell installed between the cylinder and the loading bracket. Uplift loading was measured via an electronic pressure transducer located within the pressurized bladder. All load and displacement measurements were recorded using an electronic data acquisition system.

Prior to testing, a series of calibrations were conducted on a full-size specimen to verify the correlation of the uplift load calculated from the pressure measurements to the uplift reaction load measured using load cells placed at the bottom of the supporting walls. Uplift load based on

pressure measurements was calculated as the pressure reading inside the air bladder multiplied by the area of loading (i.e., the translatable bearing surface) and then adjusted down to account for the weight of the specimen and the set-up. For direct measurement of uplift load (rather than pressure), each anchor bolt was fitted with a 5,000-lb load cell that measured the uplift force between the anchor bolt and the specimen. Oversized holes were drilled at each of the anchor bolt locations in the bottom plates of the supporting walls to prevent friction. The calibration indicated that the uplift load based on the pressure measurements and the reaction load measurements were within 6.5% of each other, with the pressure-based uplift loads being the higher of the two. Subsequent uplift capacities calculated from uniform pressure load measurements during testing were adjusted down accordingly.

Calibration was also conducted to quantify the amount of lateral resistance the translatable bearing surface imparted onto the specimen through friction effects. A full-size test specimen was constructed without roof or ceiling sheathing, and subjected to incrementally increased uplift load. At each load increment, the uplift force was held constant while the specimen was laterally loaded to a set displacement and then allowed to return to its original position. Lateral load measurements during this calibration showed that the contribution of the frictional forces to the lateral load was negligible.

Results

The results of the testing are summarized in Table 4 including the unit uplift capacity and corresponding unit shear

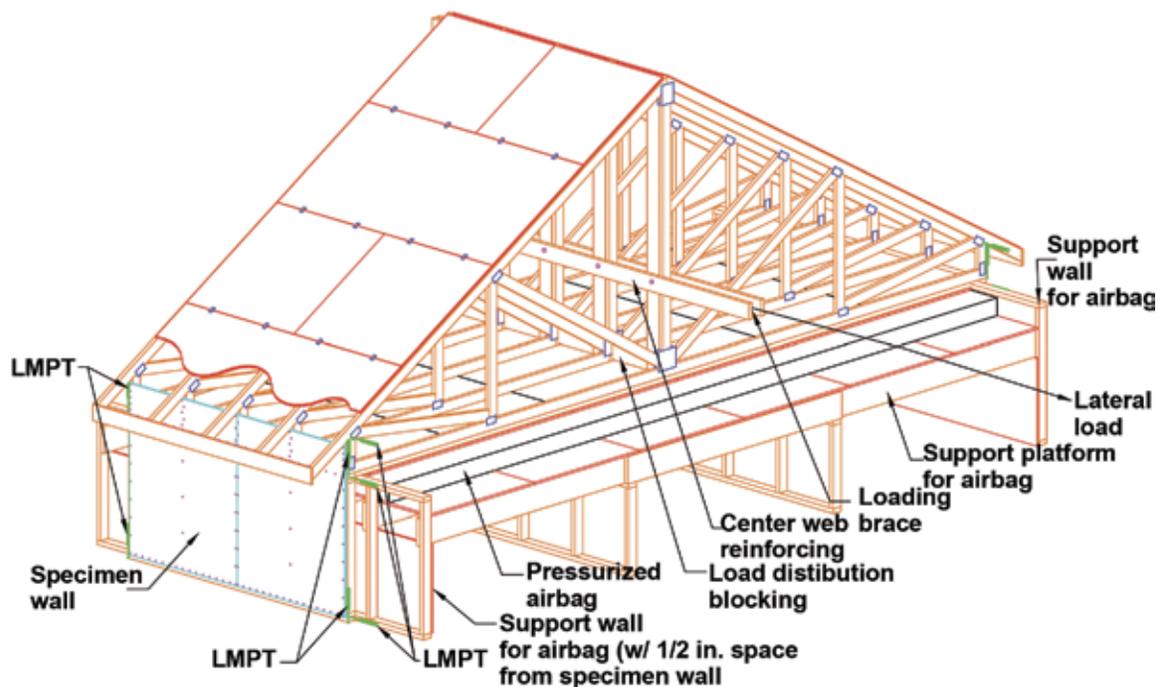


Figure 7. Test set-up and instrumentation.



Figure 8. Specimen uplift and lateral load apparatus.

capacity for each specimen. Figure 10 shows the uplift versus shear capacity interaction curve based on the test results. For comparison, Figure 10 also includes a simplified linear interaction. The Appendix provides a summary of the shear load versus horizontal displacement curves for Tests B through E, measured at various locations on the specimen, including the midpoint of the bottom chord of the first truss, the TOH of the first truss and the TOW at both ends of the specimen.

Test A (100% uplift) provided a benchmark uplift capacity for the energy truss heel connection system. Test A reached

Table 4—Test results

Test	Uplift load (as % of peak uplift capacity)	Unit uplift capacity (lb/ft)	Unit shear capacity (lb/ft)
A	100	525	—
B	85	435	143
C	70	360	236
D	40	195	393
E	0	—	592

a unit uplift capacity of 525 lb/ft. The primary failure mode in Test A was a Mode III_m yielding followed by withdrawal of the nails connecting the OSB wall sheathing and the truss heel and a withdrawal of the toe-nails from the top plate (Fig. 11).

Tests B, C, and D evaluated the shear load capacity of the connection system at various levels of uplift loading. The primary failure modes of Tests B (85% uplift) and C (70% uplift) were similar to those seen in Test A. The majority of the OSB to heel nail connections exhibited the Mode III_m Yield failure (Fig. 12), with the back trusses (trusses under tension caused by global overturning) reaching capacity first. A few of the nail connections at the center truss, where the two OSB sheets are joined, failed by nails pulling through the OSB edges (Fig. 13). This change in failure mode most likely is due to the effect of the smaller panel edge distances at that location in conjunction with the shear load component acting perpendicular to the OSB edge.

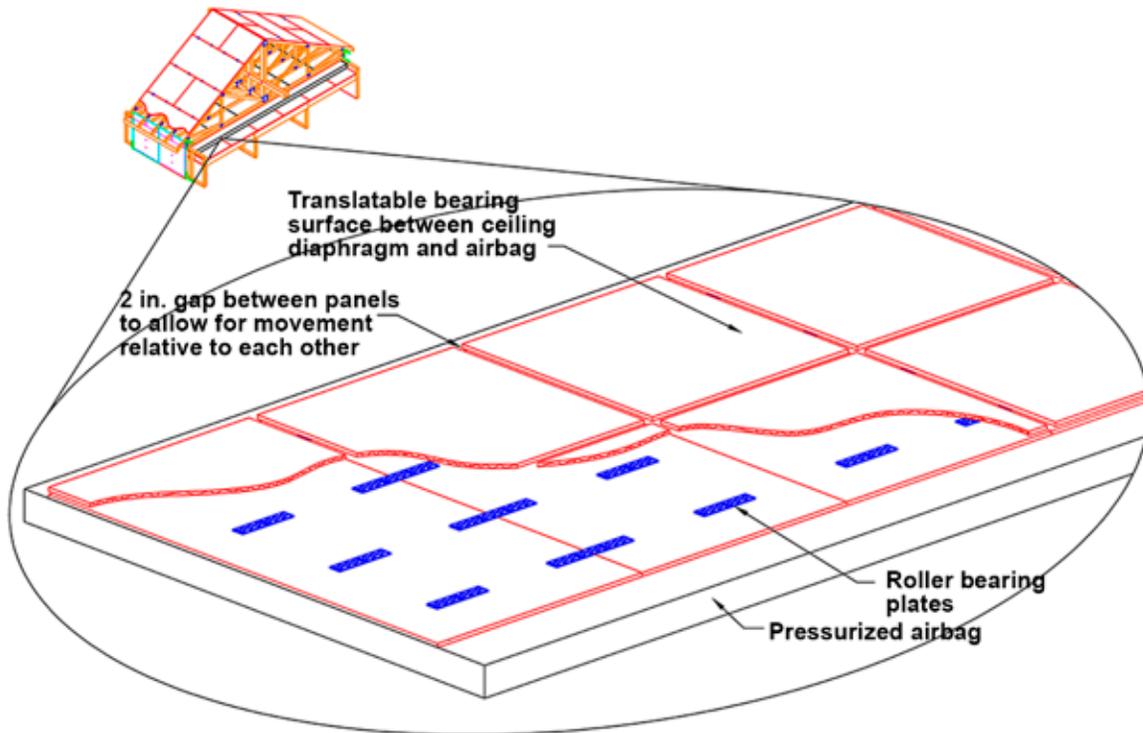


Figure 9. Translatable bearing surface.

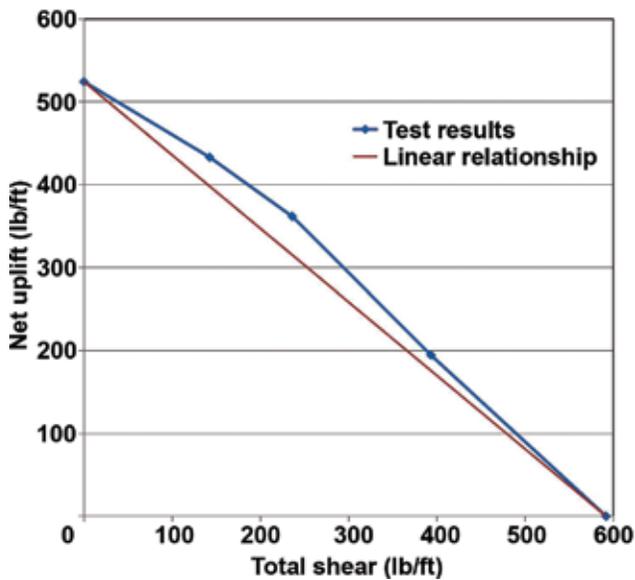


Figure 10. Uplift versus shear interaction curve from testing.



Figure 12. Fastener yield failure mode of Test C.

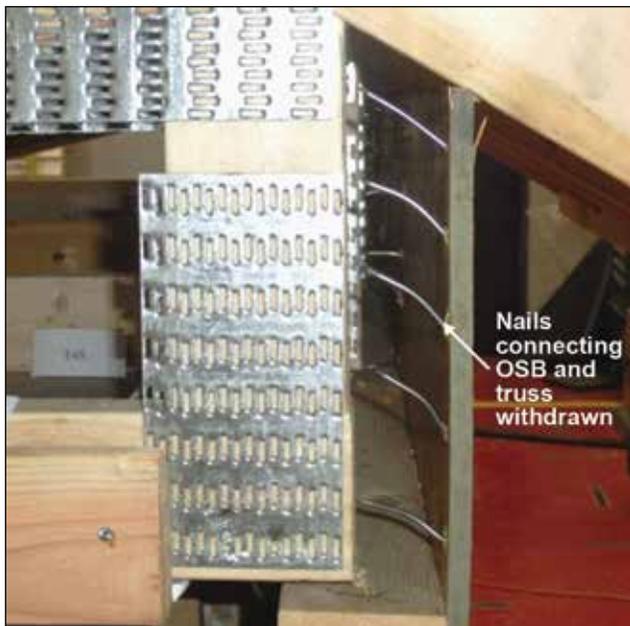


Figure 11. Primary failure mode of Test A.



Figure 13. Nail pull-through failure of Test C.

The primary failure mode of Test D (40% uplift) was a cross-grain bending failure of the wall bottom plates at the uplift ends of the supporting walls, initiated by a failure of the test set-up reaction anchors that secured the tension stud hold-downs (Fig. 14). However, significant deformations were observed at the heel connection (Fig. 15), suggesting that the roof-to-wall connection had reached or was near its capacity.

During Test D (40% uplift), the roof-to-wall connection had reached or was near its capacity as evidenced by the

significant deformations observed at the heel connection; however, the overall capacity of the specimen was limited by the failure of the uplift hold-downs and subsequent cross-grain bending failure of the bottom plates at the uplift ends of the supporting walls. Further inspection indicated that the hold-down anchors had not been fully engaged within the reaction frame of the testing set-up before the start of the test.



Figure 14. Set-up anchor failure of hold-down during Test D.



Figure 16. Fastener yield failure mode of Test E.



Figure 15. Deformation of OSB-to-heel connection at failure of Test D.

Test E (0% uplift load) benchmarked the shear capacity of the energy truss heel connection system. The ceiling diaphragm of the Test E specimen was reinforced with additional sheathing on the top face of the truss bottom chords to achieve failure at the energy truss heel connection. Test E reached a unit shear capacity of 592 lb/ft, and also exhibited a Mode III_m Yield failure at the OSB-to-heel connection as its primary failure mode, with the nail bending/yielding occurring in the horizontal direction (Fig. 16).

Evaluation of the combined uplift and shear capacity interaction curve indicates a slightly nonlinear relationship with all of the tested capacities greater than those predicted by a linear relationship. Therefore, a linear relationship may be a simplified and conservative representation of the response under combined loading for design of energy truss heel connections using extended wood structural panel sheathing.

Figures 17 and 18 compare the combined uplift and shear interaction curve based on testing to design wind load scenarios for several typical house roof sizes. The roof sizes (24 ft by 50 ft, 36 ft by 50 ft, and 48 ft by 50 ft) were selected to bracket the majority of the building dimensions and roof length to width aspect ratios present in residential construction. The wind loads were determined using tables 2.2A and 2.5B from *Wood Frame Construction Manual (WFCM) for One- and Two-Family Dwellings—2012 Edition* (AWC 2012) assuming a mean roof height of 30 ft, a 7:12 roof pitch, and the truss span in the short direction. Note that the 2012 edition of the WFCM is based upon *ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures* (ASCE 2010) basic wind speeds for a 3-s gust and 700-year return period. Design wind loads were adjusted up by assuming a factor of safety of 2.0 to provide a direct comparison to the uplift and shear capacities of the system measured from testing.

Analysis presented in Figures 17 and 18 shows that the capacity of the energy truss heel connection exceeds the adjusted design values in Exposure B wind regions for all analyzed building configurations at basic wind speeds up to 120 miles per hour (mph), and in the case of the shorter roof spans at basic wind speeds up to 150 mph. Accordingly, the analysis indicates that the tested energy truss heel connection can provide a factor of safety of 2.0 in Exposure B wind regions

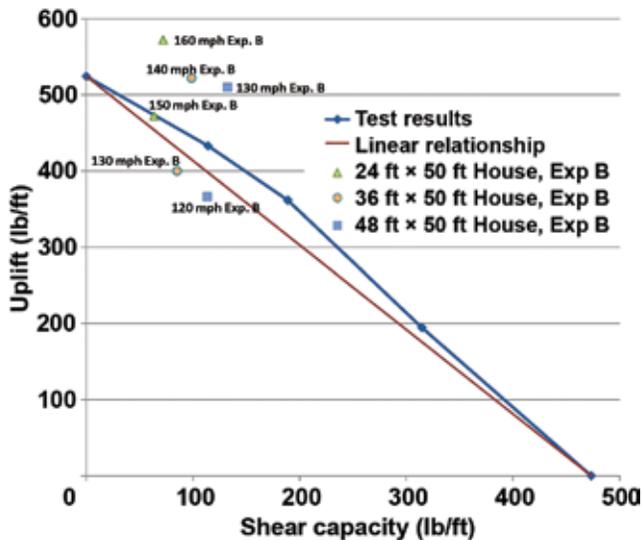


Figure 17. Comparison of tested capacity to typical wind-loading scenarios (Exposure B regions).

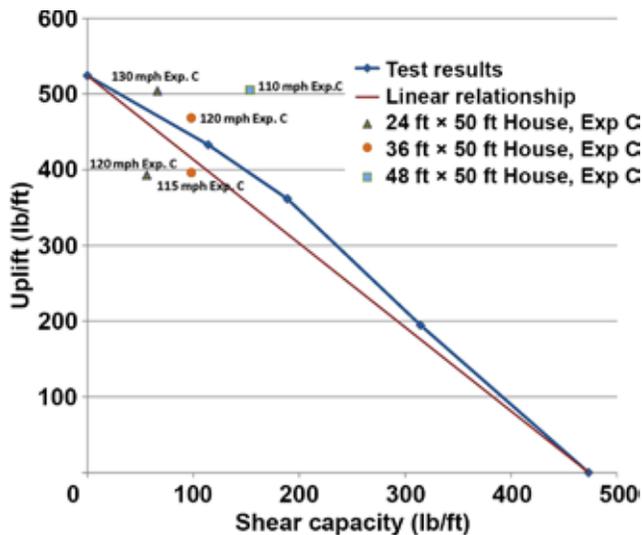


Figure 18. Comparison of tested capacity to typical wind-loading scenarios (Exposure C regions).

for nearly all typical building layouts across the majority of the country. The applicability of the tested system is less extensive when comparing peak capacities to design values in Exposure C wind regions (Fig. 18); the system capacity yields a factor of safety of 2.0 or greater at basic wind speeds of up to 115 mph in only two of the three layouts analyzed. A basic wind speed of 115 mph, however, encompasses the greater majority of the continental United States, excluding only the coastal regions in the eastern half of the country.

Summary and Conclusions

This testing program was designed to evaluate the performance of OSB wall sheathing panels extended over the

roof truss heel in resisting combined uplift and shear forces. The results of this study are expected to provide guidance towards further expanding prescriptive solutions optimized for structural and energy performance and constructability. The following is a summary of the conclusions that can be drawn from the results of this testing program:

1. The tested system using extended wood structural panel (Norbord Tall Wall/Windstorm OSB) wall sheathing as the primary connecting element (without additional connecting hardware) at the roof-to-wall interface of energy trusses can provide a continuous load path in both the shear and uplift directions and can be considered a viable option for residential construction in most areas of the country (the limitations include hurricane-prone areas and homes with large roof systems).
2. The uplift versus shear capacity interaction curve for the energy truss heel connection system is slightly nonlinear, with capacities for all uplift to shear ratios measured in this testing program exceeding the capacities predicted based on a linear interaction. In design applications, a linear relationship may be a simplified and conservative representation of the response under combined loading for energy truss heel connection system using extended wood structural panel sheathing.

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Appendix—Summary of the Shear Load Versus Horizontal Displacement Curves for Tests B Through E

