High Heel Roof-to-Wall Connection Testing

Phase III – Evaluation of extended wall OSB sheathing connection under combined uplift and shear loading for 24-inch-heel trusses

Prepared for
Forest Product Laboratory, Forest Service, U.S. Department of Agriculture
Norbord Inc., Toronto, Ontario, Canada

Prepared by
NAHB Research Center, Inc.
400 Prince George's Boulevard
Upper Marlboro, MD 20774

December 26, 2012

Acknowledgements

This research was supported in part by funds provided by the Forest Products Laboratory, Forest Service, USDA.

This research was supported in part by funds provided by Norbord, Inc.

Disclaimer

Table of Contents

List of Tables

List of Figures

INTRODUCTION

This study is Phase III of a test program that responds to the new requirements for roof-to-wall connections in 2012 International Residential Code (IRC) and expands upon the previous phases that evaluated innovative roof-to-wall connection systems.

The new IRC provisions specify complex details for attachment of rafters and trusses to the supporting walls (ICC 2012). 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 previous testing project (NAHBRC 2011) 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. Further testing conducted by the NAHBRC (NAHBRC 2012) confirmed the ability of OSB wall sheathing panels extended over the roof heel to resist combined uplift and shear forces without additional roof-to-wall hardware.

Phase III builds upon this previous testing by evaluating the performance of the extended wall structural panel connection in resisting combined uplift and shear forces at the roof-to-wall interface with a focus on a truss heel height of 24 inches to address the expected increases in the depth of attic insulation used in Climate Zones 5 and higher.

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

OBJECTIVES

The specific objective of Phase III of the testing program is to develop an uplift versus shear capacity interaction curve for extended wood structural panel wall sheathing used as the primary connecting element at the roof-to-wall interface with 24-inch high energy truss heels.

METHODS AND MATERIALS

General

Testing was conducted at the NAHB Research Center Laboratory Facility located in Upper Marlboro, MD. All specimens were constructed in the laboratory and all construction materials were purchased from local suppliers. The Norbord TallWall/Windstorm OSB product used for the wall sheathing was donated by Norbord Inc.

A total of five laboratory tests of a roof-wall assembly with extended OSB wall sheathing were conducted, each under a different loading combination. Table 1 provides a test matrix summarizing all uplift-shear loading combinations. A completely new specimen was constructed for each test.

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 by establishing capacities under a single direction loading. The uplift load levels by percent of total uplift capacity were selected to provide system performance at realistic combined load combinations (Tests B, C, and D). 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.

Table 1 – Test Matrix

Test	Uplift Load (as % 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	Establish peak shear load capacity of the specimen

Specimen Construction

Figure 1 shows a photo of a specimen and setup. Each specimen was constructed with five (5) 24-foot span wood trusses with a 24-inch heel height, spaced at 24 inches on center. The overall size of the full roof system was 8 feet deep by 24 feet wide with additional 16-inch long overhangs on each side. Trusses were supported at the heel by 4-foot high by 8-foot long light-frame wood walls. The supporting walls were constructed with 24-inch on center framing and sheathed on the exterior with 4-foot wide Norbord TallWall/Windstorm OSB panels extending up past the top plate of the wall to capture the energy truss heel. The heel height is the distance from the top of the wall to the top of the rafters (underside of the roof sheathing) measured along the line of the wall framing exterior.

Figure 1 – Photo of test setup

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

The TallWall/Windstorm OSB sheathing was fastened to the truss heel with seven (7) 8d common nails (2½” x 0.131”) (Figure 3 and Table 3). A total of seven nails were used for the current study (with only one (1) of the seven (7) nails installed into the end grain of the bottom chord of the truss). This number of nails is based on approximately the same nail spacing that was used in Phase II of this testing program. It is also the minimum number of nails necessary based on preliminary analysis to provide a shear/rotational performance similar to that exhibited in the Phase II testing. The shear/rotational restraint limit state was used as the basis because a higher heel height creates higher demand on the shear/rotational resistance of the connection (without changing the uplift demand). Therefore, maintaining the same nail spacing was expected to provide the resistance to combined loading equal or better than observed in Phase II. No other connection hardware was installed between the wall and the roof.

Sheathing was attached to the wall framing (Figure 3 and Table 3) in general accordance with the provisions developed by American Wood Council for the use of wood structural panels to resist combined uplift and shear (Coats and Douglas 2011) that 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-inch on center nail spacing at the top plate (increased from the 3 inch 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-inch-diameter bolts at 16 inches on center with 3-inch by 3-inch by 0.229-inch square plate washers and Simpson HD hold downs at each wall’s end.

7/16-inch-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 inches on center between the framing members. A 2-inch wide roof vent was provided at the ridge (1 inch each side of the ridge) such that bearing of panel edges did not occur during testing.

Table 2 – Specimen Materials and Construction

Roof Dimensions:	24 foot roof span (plus 1 foot 4 inch overhang on each end) 8 feet deep (a total of 5 trusses)
Truss heel height:	24 inches - measured from the top of wall to the top of the truss (underside of roof sheathing). Measured along the exterior of the wall framing.
Roof Pitch:	7/12
Roof Framing Members	Metal plate connected wood trusses fabricated with No. 2 SPF lumber; Heel heights 15¼ inches
Truss Spacing:	24 inches on center
Truss-to-Wall Plate Connection:	Trusses toe-nailed to top plate of wall w/ two (2) 16d box (3¼" x 0.131") nails
Fascia Board:	1x6 nominal lumber face-nailed to each truss end w/ two (2) 8d common (2½" x 0.131") nails
Roof Sheathing Materials:	7/16-inch-thick OSB sheathing installed perpendicular to framing member w/ steel edge clips and unblocked edges parallel to the ridge
Roof Sheathing Fasteners:	8d common (2½" x 0.131") at 6 inches on center on panel perimeter and 12 inches on center in the panel field
Ceiling Material:	1/2-inch-thick gypsum panels installed perpendicular to truss bottom chord members, joints in the panel field taped and mudded
Ceiling Fasteners:	1-5/8 inch Type W drywall screws at 8 inches on center w/ first rows of fasteners 8 inches in from side walls (i.e., floating edges)
Supporting Wall Dimensions:	8 feet long by 4 feet in height
Supporting Wall Framing (including top plates):	2x4 nominal SPF STUD or No. 2 grade lumber
Supporting Wall Sheathing:	7/16-inch thick Norbord TallWall/Windstorm OSB sheathing attached with 8d common (2½" x 0.131") nails

Table 3 – Wall Sheathing Fastening Schedule

Location / Connection	Fastener Schedule
Extended Wall Sheathing to Energy Truss Heel	Face-nailed with seven (7) 8d common (2½" x 0.131")
Wall Sheathing to Vertical Wall Framing (Nailing for shear)	8d common (2½" x 0.131") at 6 inches on center around perimeter, 12 inches on center in the field
Wall Sheathing to Top Plate (Increased nailing for additional uplift load)	8d common (2½" x 0.131") at 4 inches on center along top plate
Wall Sheathing to Bottom Plate (Increased nailing for additional uplift load)	8d common (2½" x 0.131") at 3 inches on center along top plate

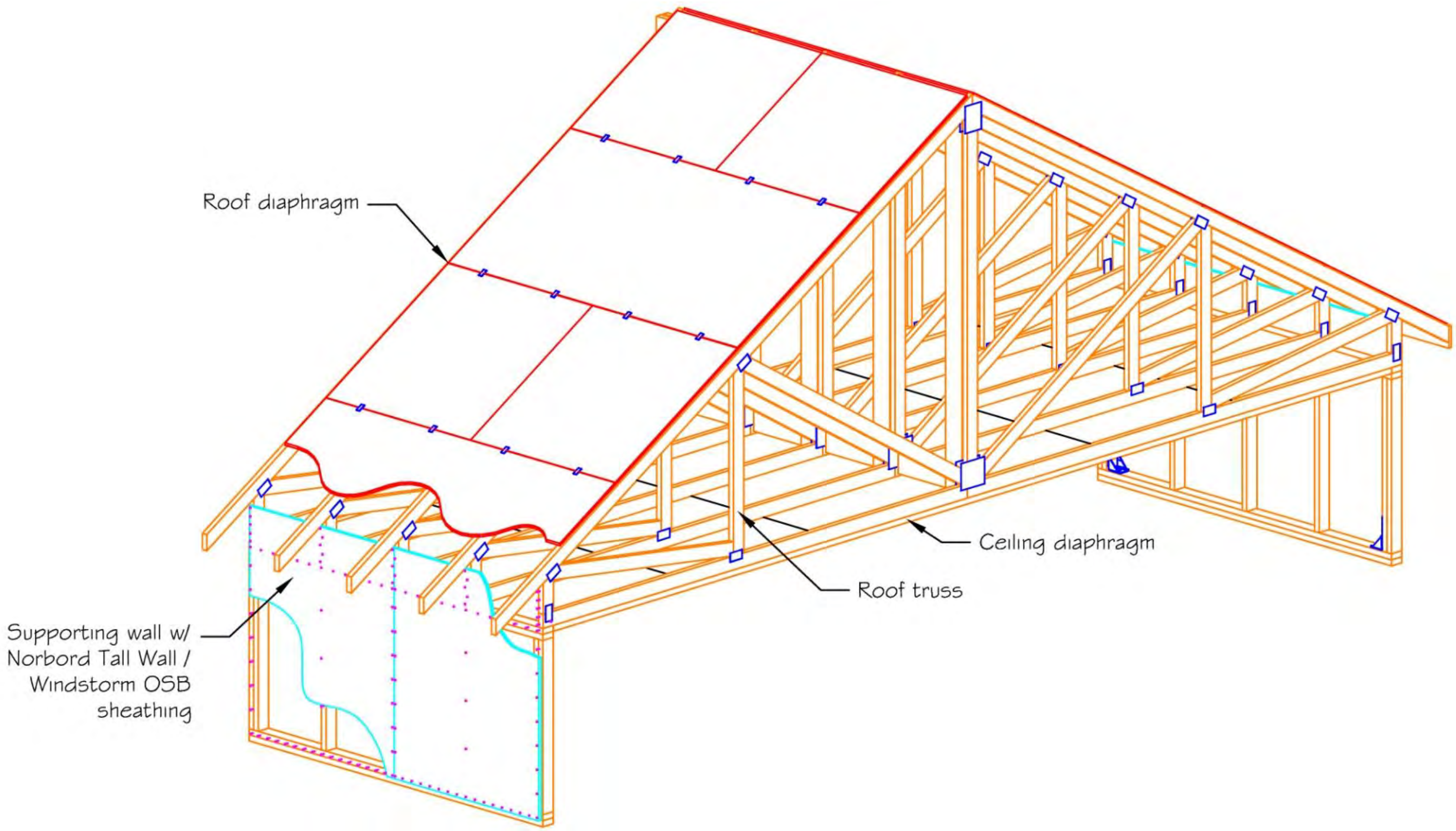


Figure 2 – Specimen construction schematic

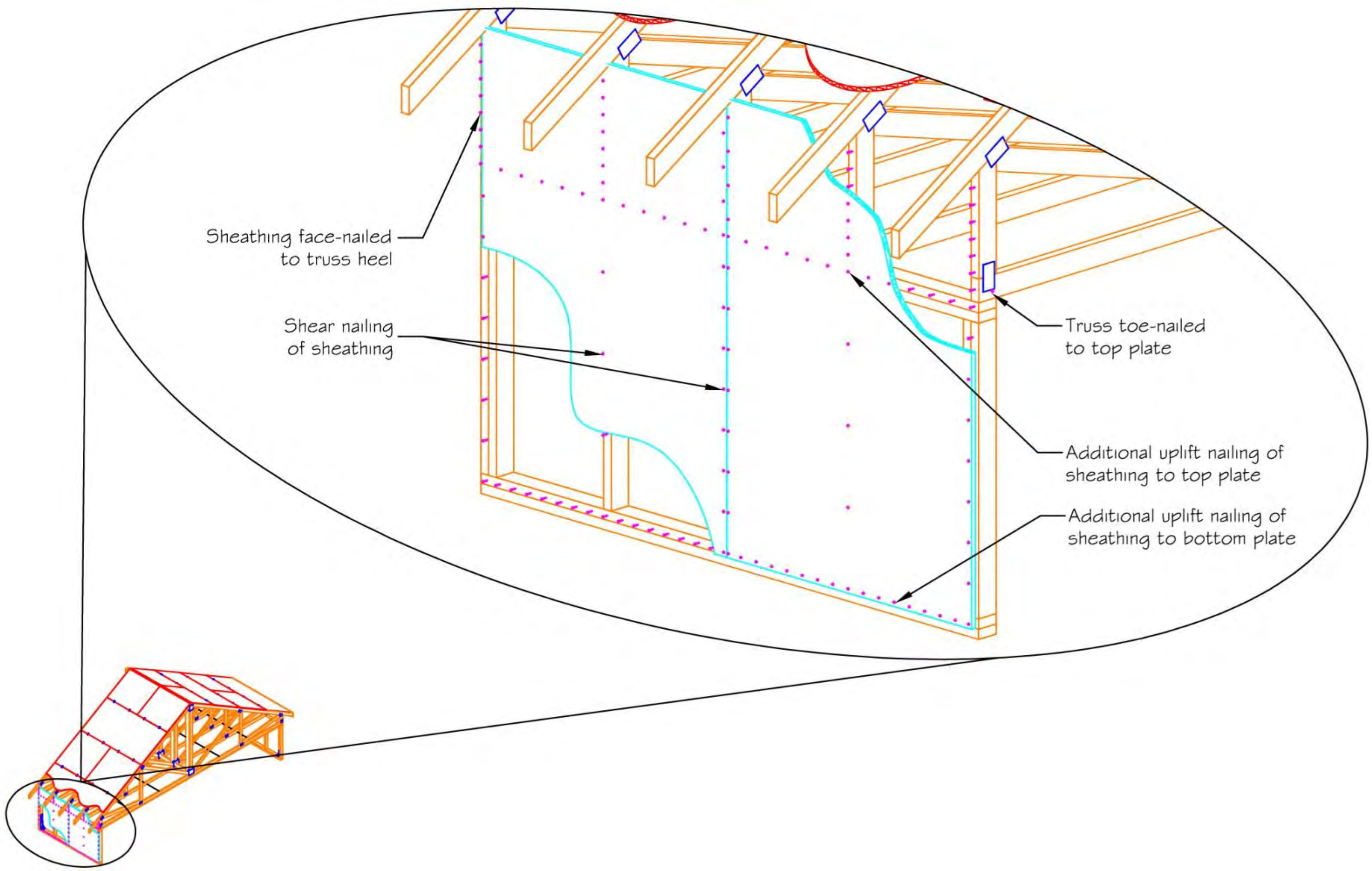


Figure 3 – TallWall/Windstorm OSB sheathing connection detail

The ceiling gypsum panels were installed perpendicular to the truss bottom chord members and the first rows of fasteners was located approximately 8 inches 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). 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.

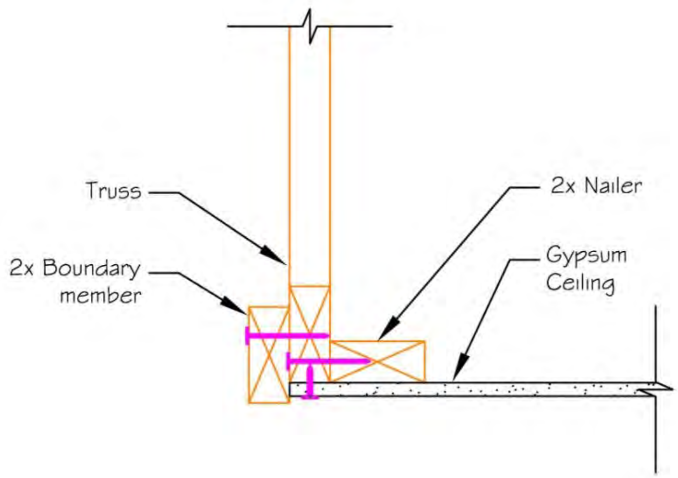
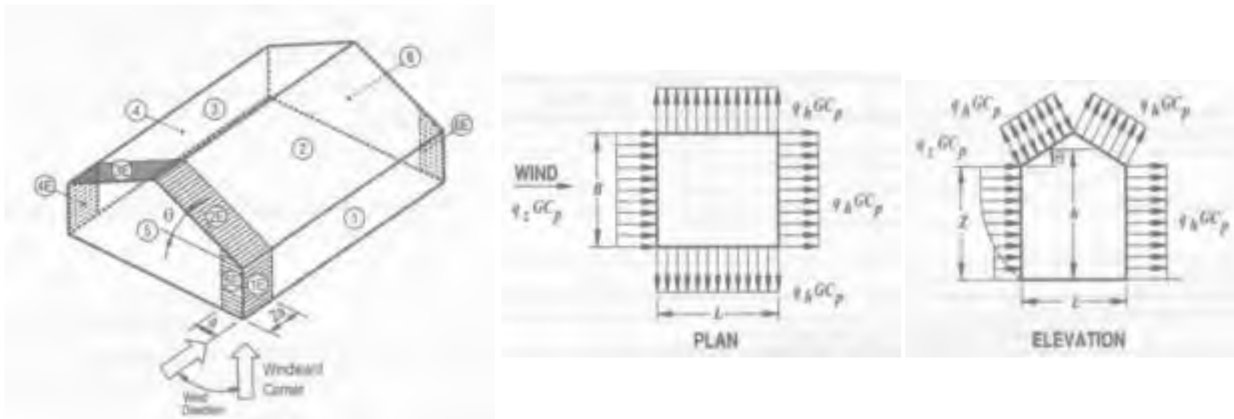


Figure 4 – Ceiling diaphragm boundary detail at front and back trusses

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

Test Setup and Protocol

Figure 5 provides the 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 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 *Evaluation of the Lateral Performance of Roof Truss-to-Wall Connections in Light-Frame Wood Systems* (NAHBRC 2011) for a more in-depth explanation of the distribution of forces and derivation of the loading methodology.



(a) Directional Procedure (b) Envelope Procedure

Figure 5 – ASCE 7 wind loading profile

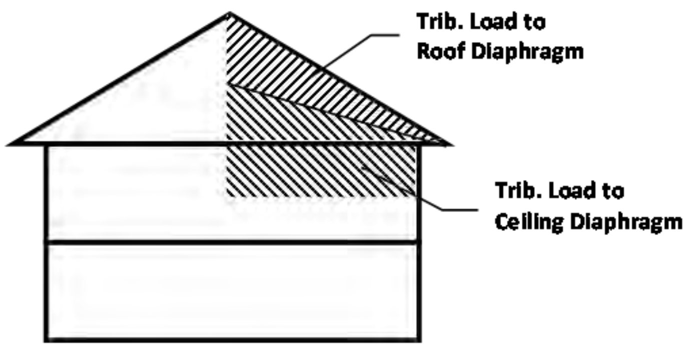


Figure 6 – Tributary area of wind loads

Figure 7 shows a schematic of the test set-up including the specimen, loading apparatus, and instrumentation and Figure 9 provides photos of the uplift/shear loading apparatus.

Lateral load was applied to the specimen using the same methodology developed and utilized during Phase I and Phase II testing of this project. Specifically, load was applied through permanent truss bracing (2x6 nominal Southern Pine, No. 2 Grade lumber) attached at the center vertical web member of each truss approximately 2 feet 8 inches 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.

Each center vertical truss web member was reinforced with a double 2x8 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-inch by ½-inch lag bolt to provide sufficient load transfer with minimal rotational restraint.

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 (Figure 7 and Figure 8). The support structure also isolated the air bag from the test specimen walls to prevent any restraint of the walls due to 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 8 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 inches 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.

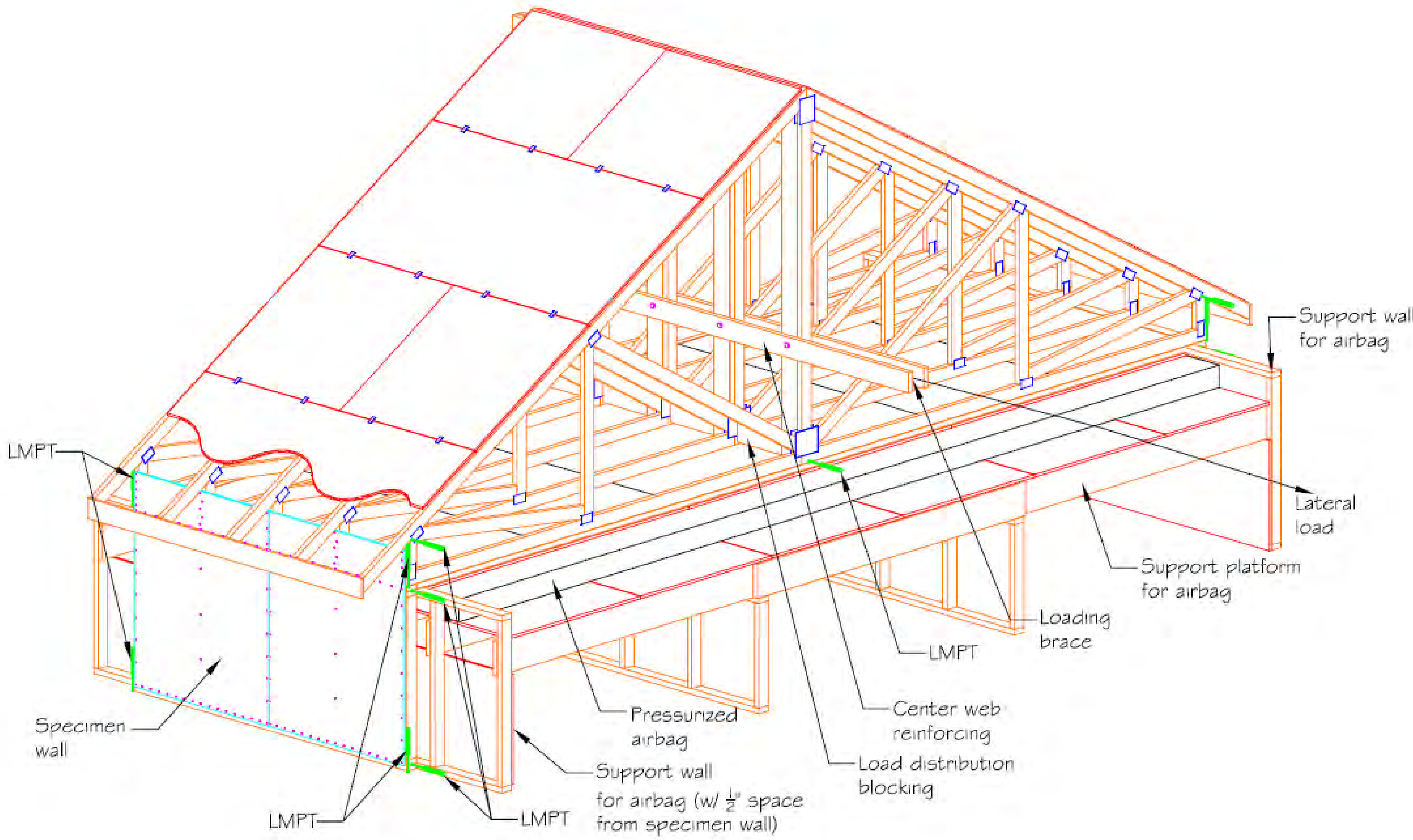


Figure 7 – Test set-up and instrumentation

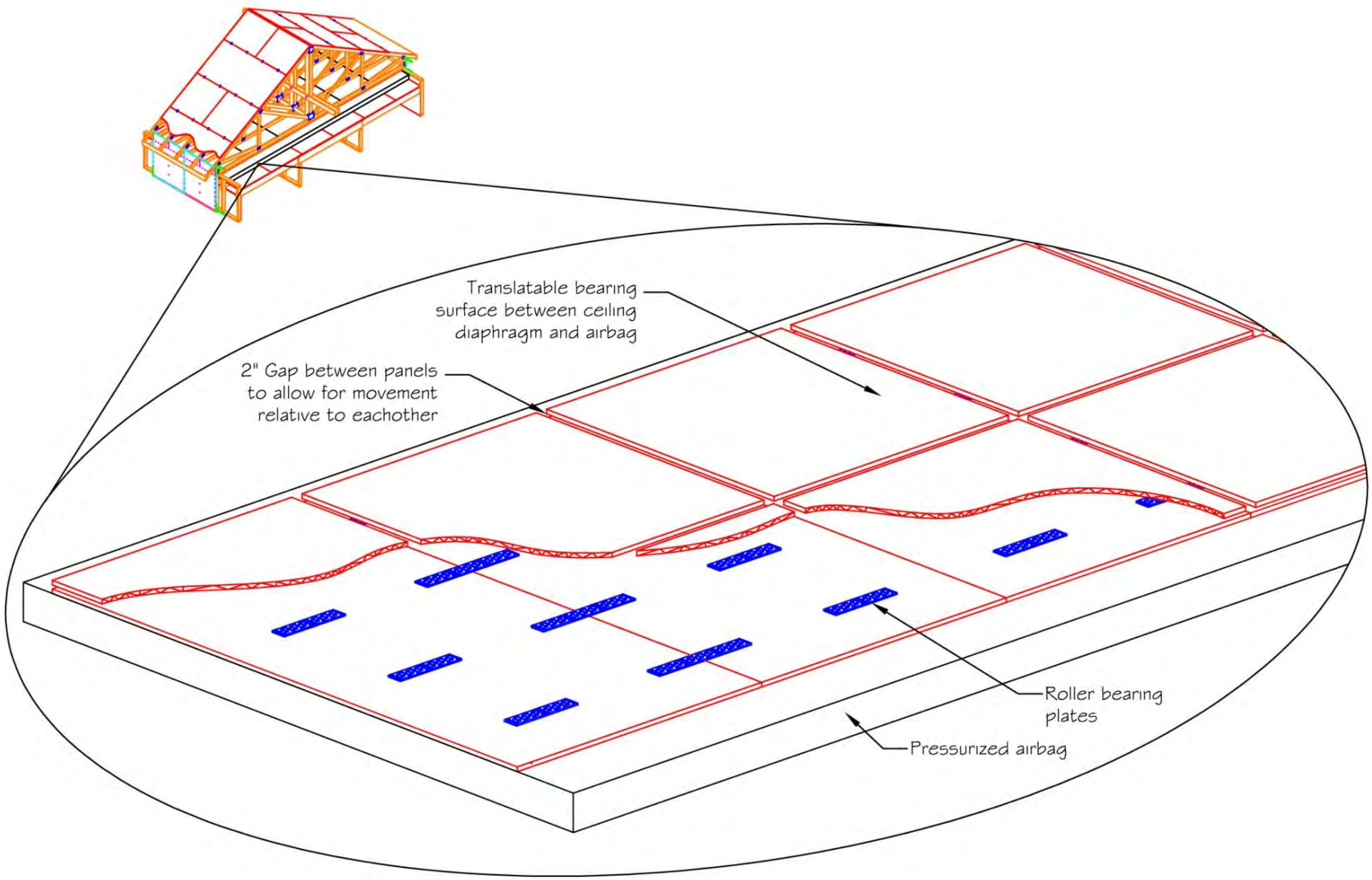


Figure 8 – Translatable bearing surface

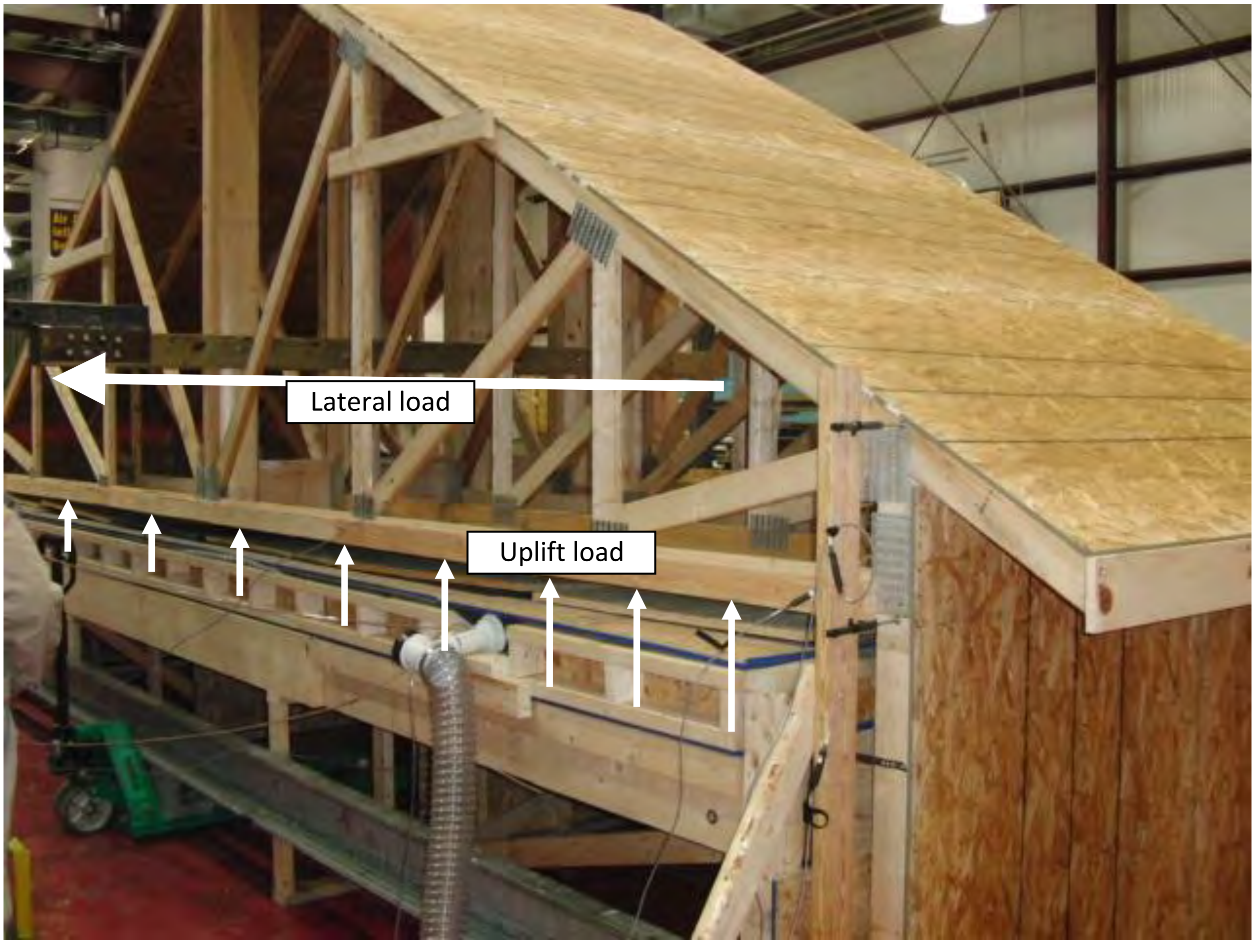


Figure 9 – Specimen uplift and lateral load apparatus

Several displacements were measured using electronic Linear Motion Position Transducers (LMPT's) as shown in Figure 7, including:

- 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 ceiling diaphragm;
- Lateral slip at the bottom of the supporting walls;
- Uplift at the end stud of the supporting walls, and;
- 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, and;
- 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.

A series of calibrations were conducted as part of Phase II testing 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 setup. 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 results of the calibration showed 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 as part of Phase II testing to quantify the amount 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 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 linear interaction. Appendix A 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.

Table 4 – Test Results

Test	Uplift Load (as % of peak uplift capacity)	Unit Uplift Capacity (lb/ft)	Unit Shear Capacity (lb/ft)
A	100%	890	---
B	85%	760	390
C	72%	640	385
D	45%	396	555
E	0%	---	620

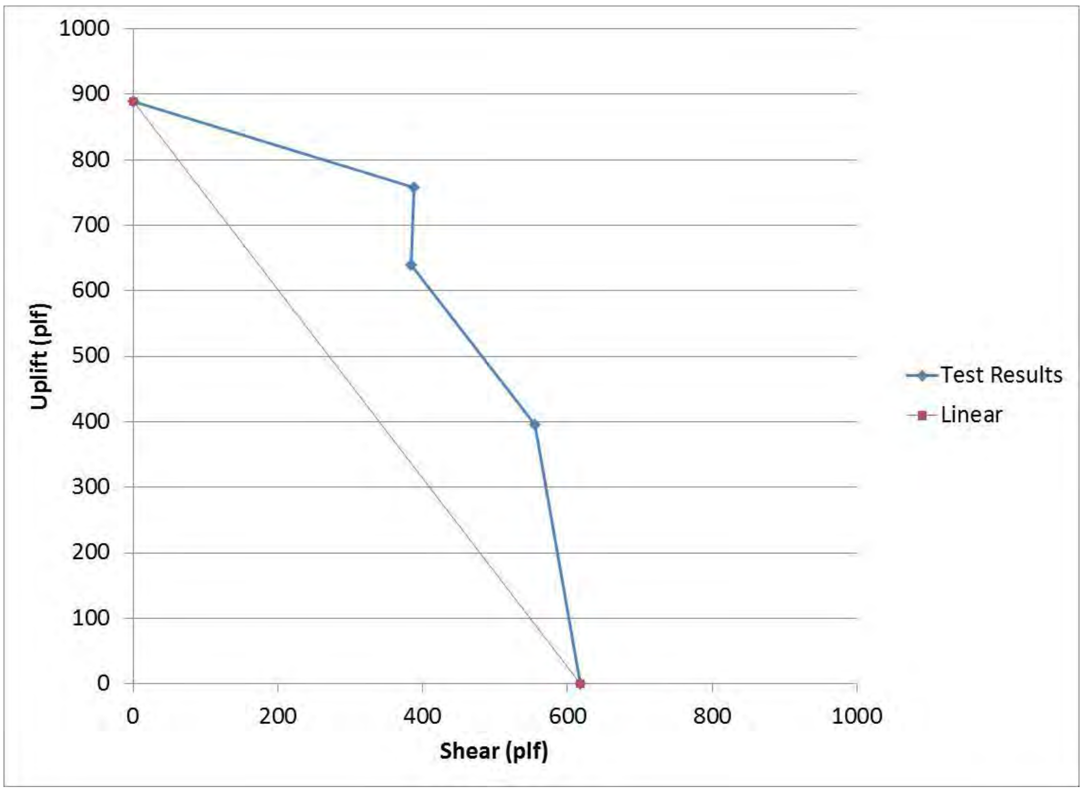


Figure 10 – Uplift versus shear interaction curve from testing

Test A (100% uplift) provided a benchmark uplift capacity for the 24-inch energy truss heel connection system. Test A reached a unit uplift capacity of 890 lb/ft. The primary failure mode in Test A was a Mode III_m yielding followed by nail head pull-through at the OSB wall sheathing–to-truss heel connection and a withdrawal of the toe-nails from the top plate (Figure 11).

Figure 11 – Primary failure mode of Test A

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 Test B (85% 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 followed by nail pull-through (Figure 12), with the back trusses (trusses under tension due to global overturning) reaching capacity first.

Figure 12 – Fastener yield and pull-through failure mode of Test B

The primary failure mode of Test C (70% uplift) and Test D (45% uplift) was nail pull-through at the connection of the OSB wall sheathing to the wall bottom plates at the uplift ends of the supporting walls (Figure 13).

Figure 13 – Fastener pull-through failure mode of Test C and 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 620 lb/ft, and also exhibited nail pull-through at the OSB-to-wall bottom plate connection as its primary failure mode (Figure 14).

Fig

Evaluation of the combined uplift and shear capacity interaction curve indicates a 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 24-inch energy truss heel connection systems using extended wood structural panel sheathing.

Figure 15 provides a comparison between the results of Phase II and Phase III of this testing program.

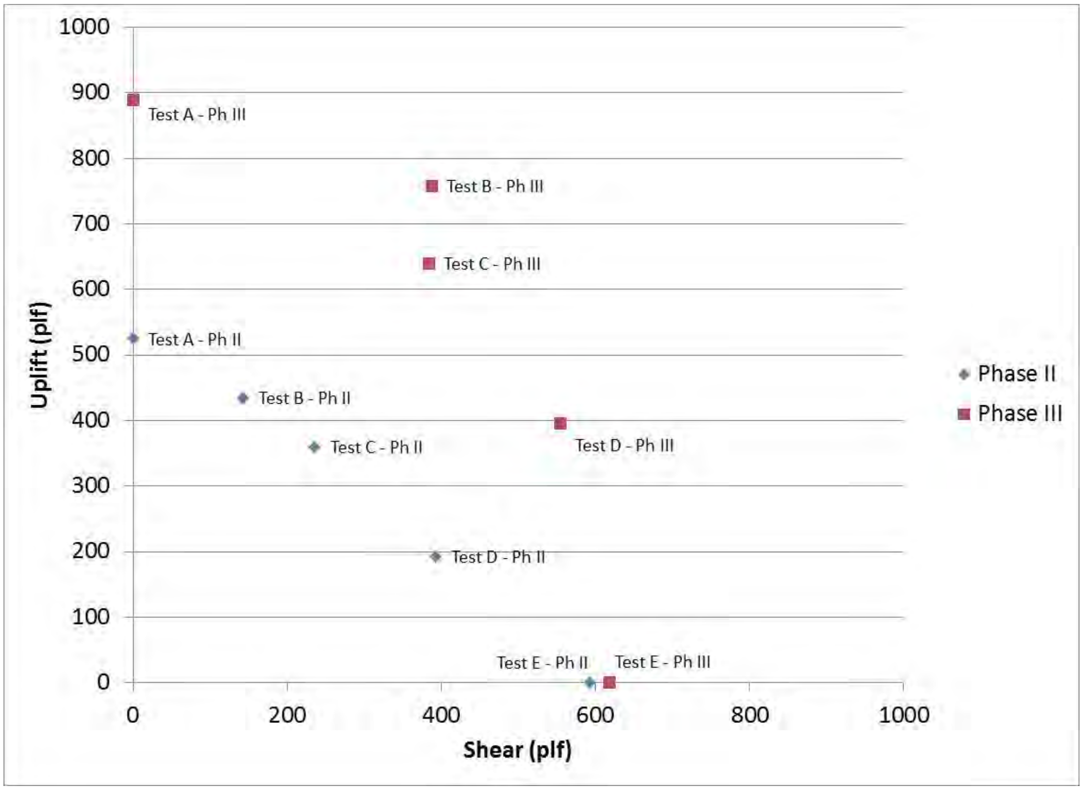


Figure 15 – Comparison of Phase II and Phase III results

The uplift capacity in Test A of the Phase III testing was approximately 60% greater than the capacity of the similar uplift test from Phase II. This increase of capacity over the Phase II results was expected due to the increased number of nails in the wall sheathing-to-truss heel connection (7 nails in Phase III testing vs. 5 nails in Phase II testing). The magnitude of the increase, however, is slightly greater than expected, with only a 40% increase in capacity predicted by the ratio of nails used in the two phases. This greater percent increase is attributed to the variability in the density of the lumber used in the high-heel energy truss construction.

The shear capacities from the lateral load only scenario of the two phases of testing were nearly equal, indicating that the additional nails offsets the greater overturning effects caused by the increased truss heel height. The controlling failure mode in the lateral load only tests changed from a roof-to-wall connection failure in Phase II to a bottom of wall sheathing-to- plate connection failure in Phase III. Both of these facts indicate a balanced system that is not adversely affected by the increase in heel height.

The increased capacity of Phase III Tests B, C, and D, along with the change in failure location from the roof-to-wall connection to the sheathing-to-bottom plate connection further indicates that the additional nailing in the truss heel connection offsets the greater overturning moment caused by the increase in heel height. Figure 16 and Figure 17 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 feet by 50 feet, 36 feet by 50 feet, and 48 feet by 50 feet) 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 feet, 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-second 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.

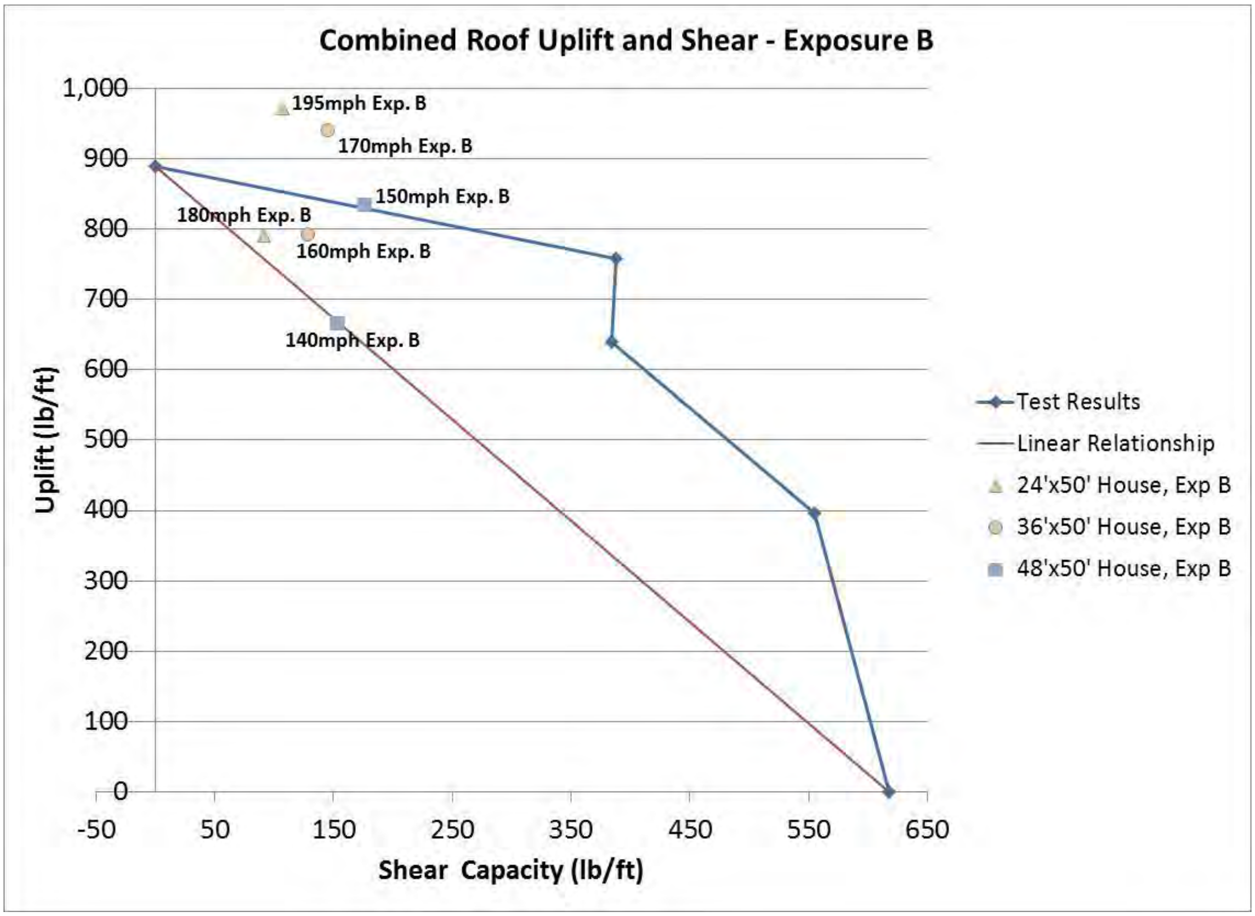


Figure 16 – Comparison of tested capacity to typical wind loading scenarios (Exposure B regions)

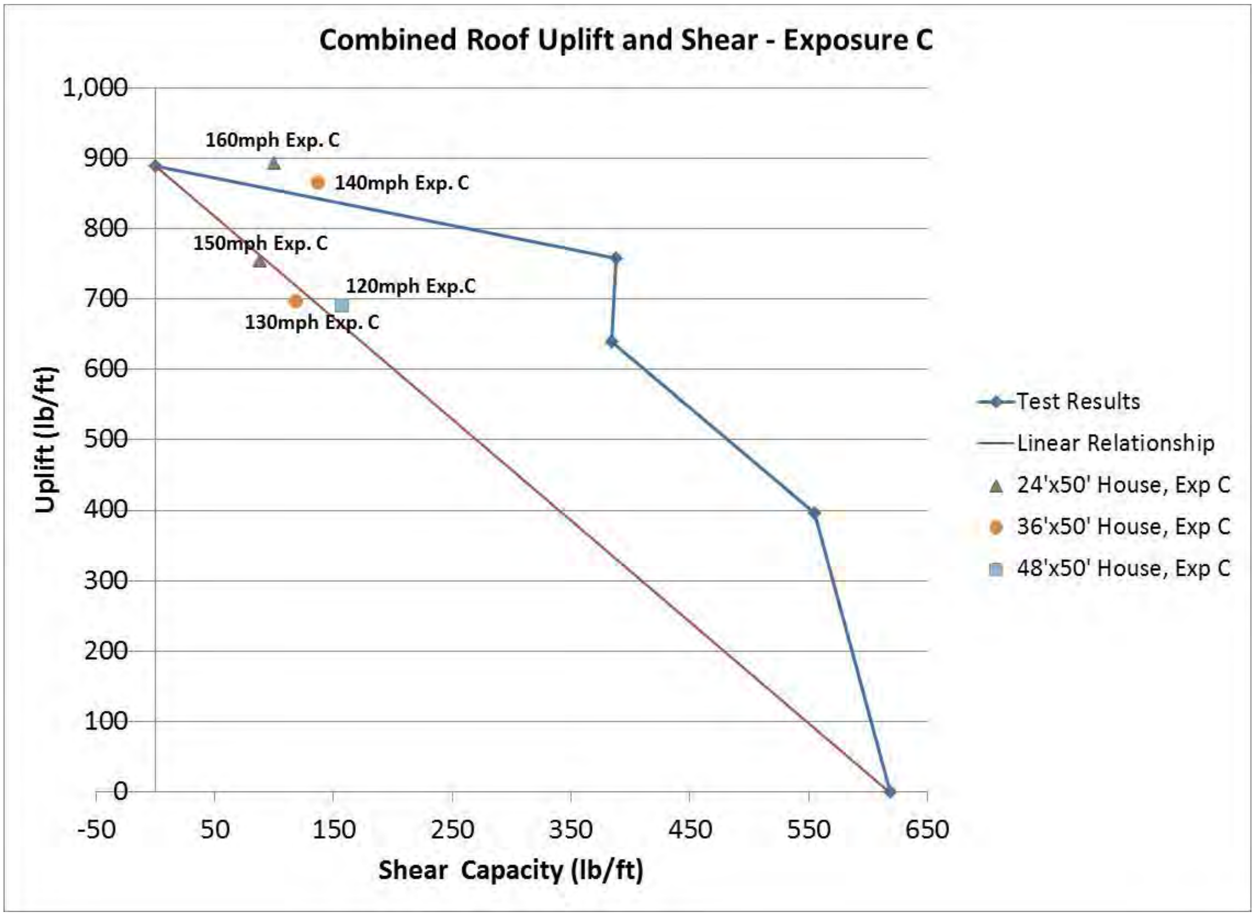


Figure 17 – Comparison of tested capacity to typical wind loading scenarios (Exposure C regions)

Analysis presented in Figure 16 and Figure 17 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 140 mph, and in the case of the shorter roof spans at basic wind speeds up to 180 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 for nearly all typical building layouts across the majority of the country. The applicability of the tested system is also extensive when comparing peak capacities to design values in Exposure C wind regions (Figure 17); the system capacity yields a factor of safety of 2.0 or greater at basic wind speeds of up to 120 mph. A basic wind speed of 120 mph encompasses the greater majority of the continental US, excluding only the coastal regions in the eastern half of the country.

SUMMARY AND CONCLUSIONS

This testing program was designed to further evaluate the performance of OSB wall sheathing panels extended over the roof heel in resisting combined uplift and shear forces. The results of this study provide guidance towards further expanding prescriptive solutions for high-heel truss attachment optimized for performance from the structural, energy, and constructability standpoint. The following is a summary of the conclusions based on the results of this testing program:

1) The tested system using extended wood structural panel (Norbord TallWall/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.

2) The overturning effects caused by increased energy truss heel heights can be offset by additional face nails attaching the extended wood structural panel wall sheathing to energy truss heel. This conclusion has been validated for trusses with heel heights up to 24 inches. (Results of Phase II testing validated performance for trusses with heel heights up to 15¼ inches). The number of nails used in these testing programs for attaching wall sheathing to the truss heel is based on a maximum nail spacing of 4 inches on center at the heel member.

3) The uplift-shear capacity interaction curve for the energy truss heel connection system is 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.

REFERENCES

American Society of Civil Engineers 2010. *Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-10)*. ASCE, Reston, VA.

American Wood Council 2011. *Woodframe Construction Manual for One and Two Family Dwellings (2012 Edition)*. AWC, Leesburg, VA.

Coats, P. & Douglas, B. 2011.*Use of Wood Structural Panels to Resist Combined Shear and Uplift from Wind*. Structure Magazine. Reedsburg, WI.

Gypsum Association. 2010. *Application and Finishing of Gypsum Panel Products (GA-216-2010)*. Gypsum Association, Hyattsville, MD.

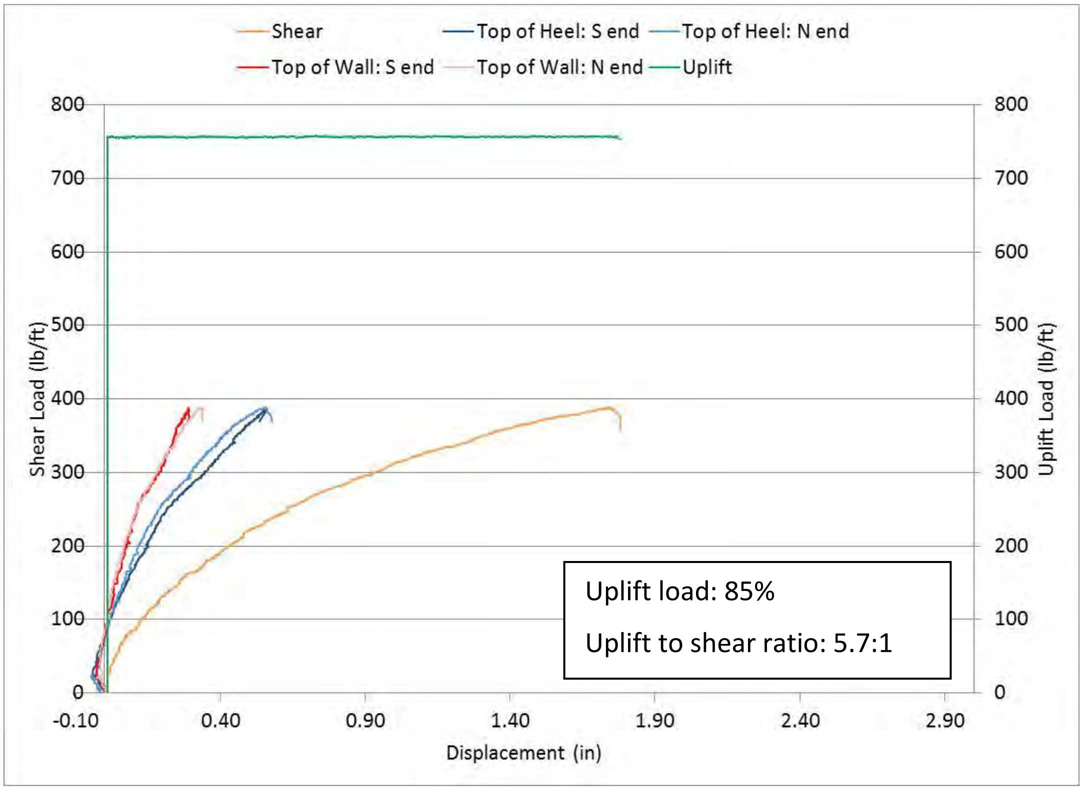
International Code Council. 2012. *International Residential Code for One and Two Family Dwellings*. ICC, Country Club Hills, IL.

NAHB Research Center. 2011. *Evaluation of the Lateral Performance of Roof Truss-to-Wall Connections in Light-Frame Wood Systems*. NAHBRC. Upper Marlboro, MD.

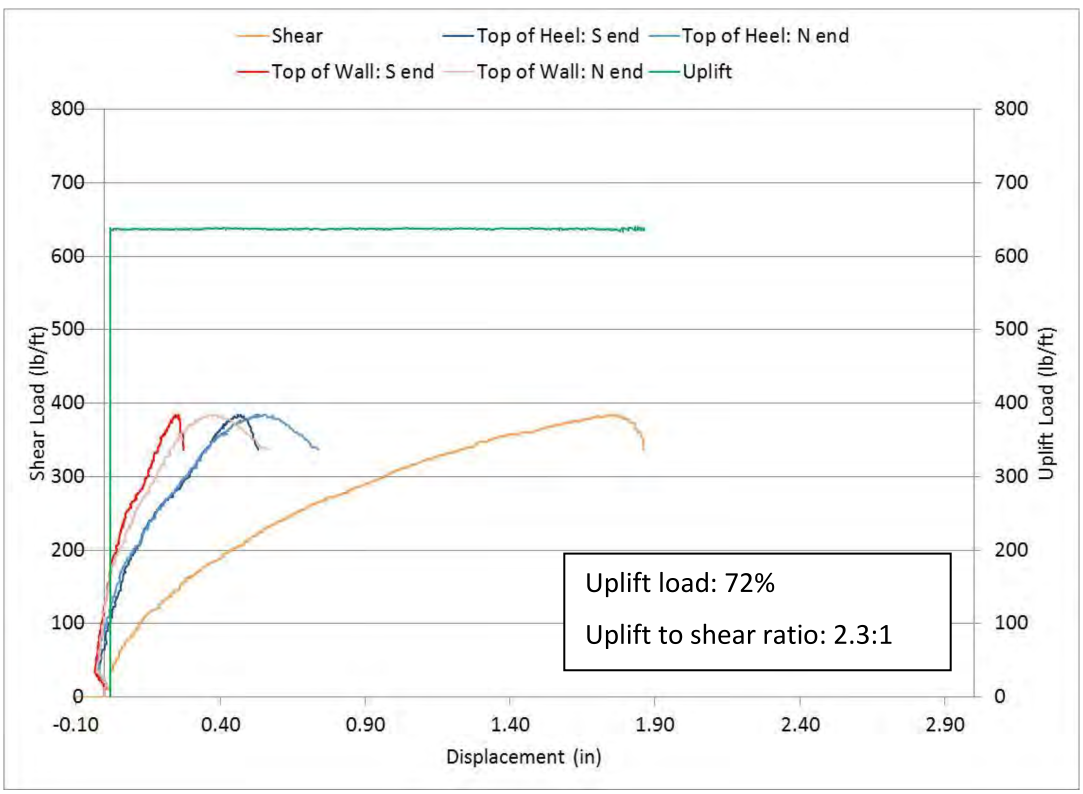
NAHB Research Center. 2012. *High Heel Roof-to-Wall Connection Testing – Phase II – Evaluation of extended wall OSB sheathing connection under combined uplift and shear loading.* NAHBRC. Upper Marlboro, MD.

APPENDIX A

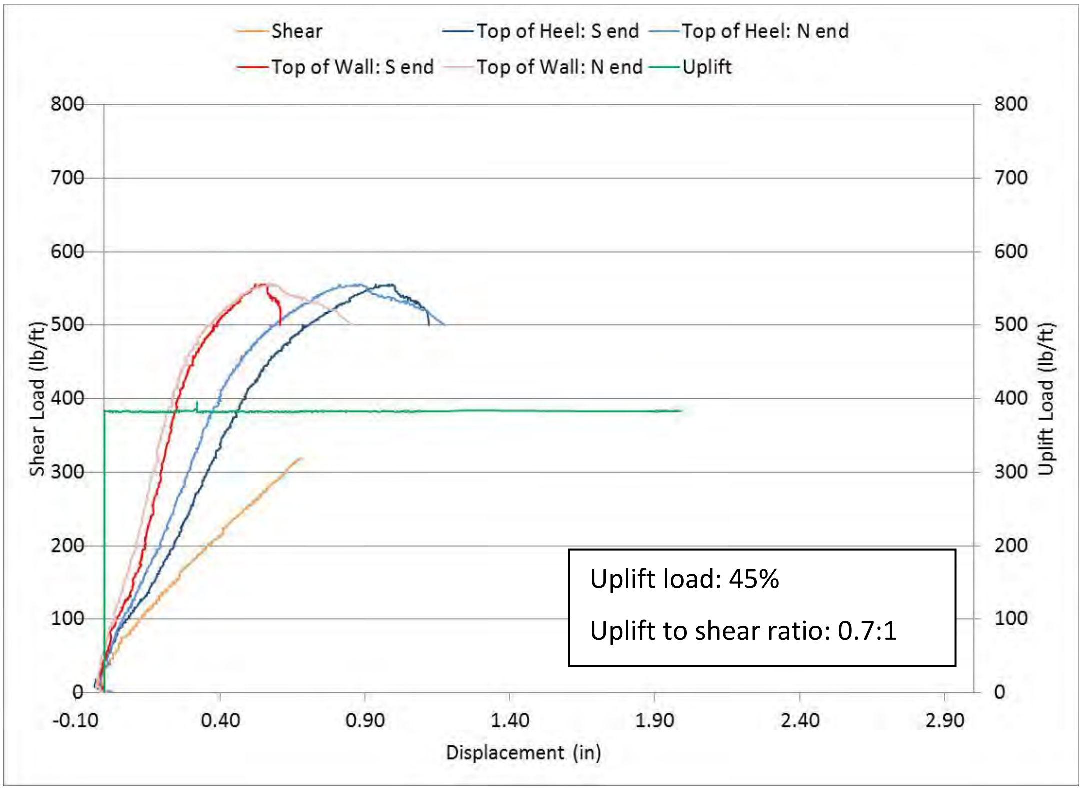
Specimen Load vs. Deflection Curves



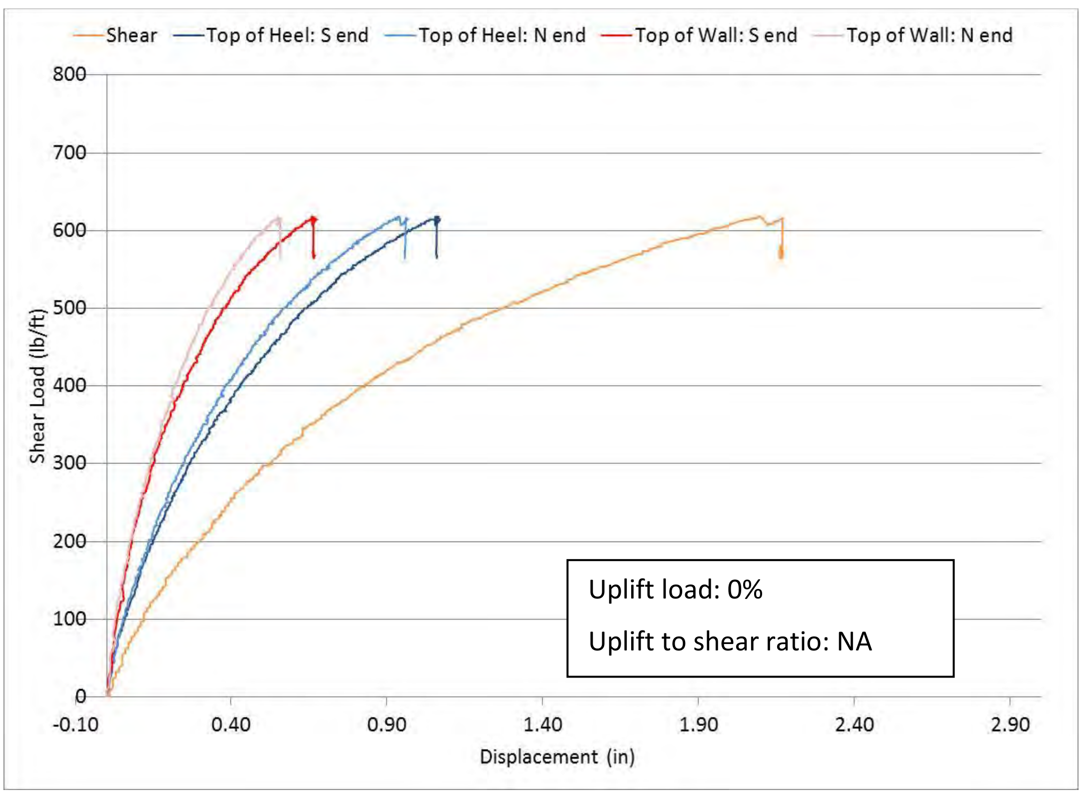
Test B



Test C


Shear
Top of Heel: S end
Top of Heel: N end
Top of Wall: S end
Top of Wall: N end
Uplift
Shear Load (lb/ft)
Uplift Load (lb/ft)
Displacement (in)
Uplift load: 45%
Uplift to shear ratio: 0.7:1

Test D


Shear
Top of Heel: S end
Top of Heel: N end
Top of Wall: S end
Top of Wall: N end
Shear Load (lb/ft)
Displacement (in)
Uplift load: 0%
Uplift to shear ratio: NA

Test E