Evaluation of the Lateral Performance of Roof Truss-to-Wall Connections in Light-Frame Wood Systems

Prepared for
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National Association of Home Builders

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INTRODUCTION

The 2009 International Residential Code (IRC) includes new requirements for roof-to-wall connections (Section R602.10.6) at braced wall panels. These new requirements were introduced in a related effort to the work of the ICC Ad Hoc Committee on Wall Bracing[1] with the intent to clarify or, where appropriate, reinforce the lateral load path between the roof and the braced wall panels of the supporting walls below. Particularly, significant changes were introduced for homes with high-heel trusses or deep rafters and for homes located in high hazard areas (wind speeds of 100 mph or higher or Seismic Design Category D0, D1, or D2). The requirements include provisions for additional fastening, blocking, knee walls, sheathing, or a combination of those elements. These requirements were further clarified and refined for the 2012 IRC.

The new requirements are labor intensive and have implications on the cost and time of construction, particularly as high-heel roof configurations become more common as a solution for meeting increasing energy efficiency standards. The proposed testing program is designed to benchmark the performance of traditional roof systems and incrementally-improved roof-to-wall systems with the goal of developing connection solutions that are optimized for performance and constructability.

OBJECTIVES

With the general focus on the lateral capacity of the roof-to-wall connections in the direction parallel to ridge, the specific objectives of this test study are to:

1) Establish performance-based limitations on traditional low-heel roof-to-wall connections using hurricane ties and without blocking, with specific intent to:
   a) Benchmark the capacities of the unblocked roof diaphragm and unblocked ceiling diaphragms tested as part of a roof assembly
   b) Benchmark the rotational response of the unblocked roof-to-wall connections
   c) Understand the system response of the overall roof assembly including the interaction between roof and ceiling diaphragms
2) Establish performance-based limitations for unblocked high-heel roof trusses attached with hurricane ties
3) Measure the performance of high-heel truss systems with intermittent blocking
4) Measure the performance of high-heel truss systems braced against rotation with wood structural panel sheathing attached to the vertical heel member of the truss

CODE REQUIREMENTS

The new IRC provisions are intended to increase the capacity of the heel joint in (1) resisting lateral forces between the roof and the wall and (2) resisting local rotation of the roof members at supports. Table 1

[1] The Ad Hoc Committee on Wall Bracing was established by the International Code Council (ICC) to review the provisions of the International Residential Code (IRC) related to wall bracing.

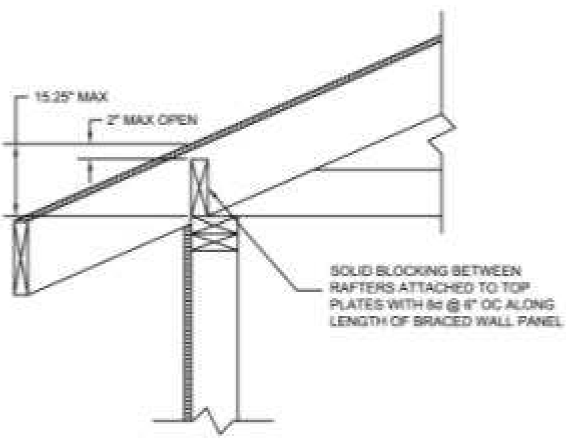
summarizes the 2012 IRC provisions for attachment of roof to walls. Figure 1 illustrates the IRC prescriptive detailing options. The special detailing requirements are triggered based on the following:

- Wind speed of 100 mph or greater;
- Seismic design category D_0 and higher, and,
- Heel heights of 9¼” and 15¼”.

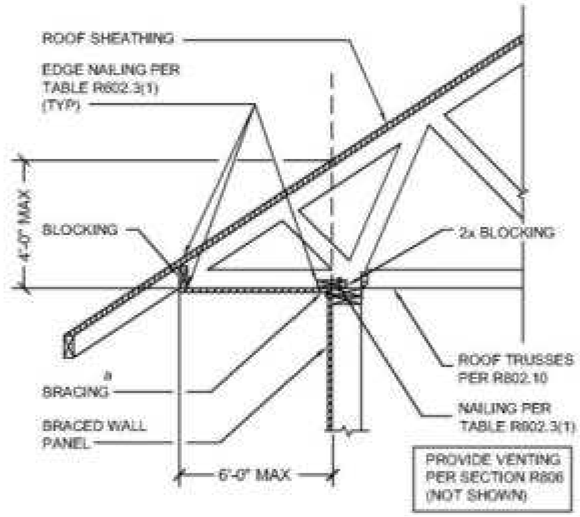
Table 1 – 2012 IRC Provisions for Roof-to-Wall Attachment

<table>
<tr><th colspan="2">Triggers</th><th rowspan="2">Minimum Requirements</th><th rowspan="2">Notes</th></tr>
<tr><th>Wind/Seismic</th><th>Roof Configuration</th></tr>
<tr><td rowspan="3">Wind less than 100 mph and SDC A, B, C</td><td>Rafter or truss heel joint 9¼ or less</td><td>Nailed connection per IRC Table R602.3(1)</td><td>At each heel joint along the length of the entire wall</td></tr>
<tr><td>Rafter heel joint height 9¼”—15¼”</td><td>Nailed connection per IRC Table R602.3(1)
AND
Partial height blocking nailed to wall top plate</td><td rowspan="4">Blocking is only at the braced wall panel</td></tr>
<tr><td>Truss heel joint height 9¼”—15¼”</td><td>Trusses attached per IRC Sections R802.10 and R802.11
AND
Partial height blocking nailed to wall top plate</td></tr>
<tr><td>Wind 100 mph or greater and SDC D_0, D_1, D_2</td><td>Rafter or truss heel joint height up to 15¼”</td><td>Nailed connection per IRC Table R602.3(1)
AND
Partial height blocking nailed to wall top plate</td></tr>
<tr><td>All wind speeds and all SDCs</td><td>Rafter or truss heel joint exceeds 15¼”</td><td>Options:
(1) Blocking at overhang and at top plate per Figure R602.10.6.2(2)
(2) Partial height blocking with wall panels per Figure R602.10.6.2(3)
(3) Engineered full-height blocking panels nailed to roof sheathing (blocked diaphragm)
(4) Other engineered methods</td></tr>
</table>

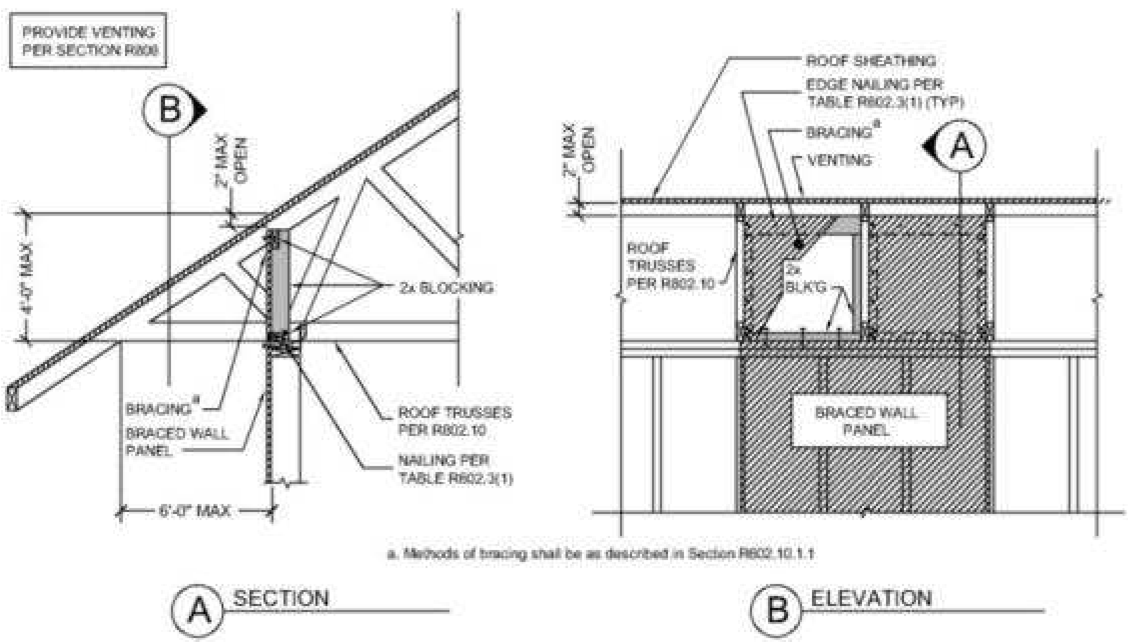
In the 2012 IRC, blocking is the primary method to increase the lateral capacity of the heel joint. The blocking members transfer shear load to the top plate of the wall below through face nailing or toe-nailing and restrict local rotation of the roof framing members caused by the eccentricity of the heel joint. The load from the roof diaphragm is transferred into the blocking either through end bearing or, if constructed as a blocked diaphragm with the roof sheathing nails penetrating the blocking, through the sheathing fasteners.



(a) Partial Height Solid Blocking



(b) Eave Bracing



(c) Blocking Panels

Figure 1 – Prescriptive detailing options in IRC

LOADING CONSIDERATIONS

The heel joint at the roof-to-wall interface is subject to wind and seismic forces including:

- Lateral forces (wind or seismic);
- Uplift forces (wind; roof uplift forces due to vertical seismic accelerations are not considered in residential design), and,
- Rotational (overturning) forces (secondary forces due to eccentricity of lateral force).

The scope of this project is limited to investigating the lateral load path and the connections and detailing in the IRC for resisting lateral forces in the direction perpendicular to the roof framing members (i.e., parallel to the ridge of the roof). Figure 2 shows the loads and the forces in the direction of interest of this study (note that uplift and orthogonal components are not shown intentionally for clarity). The perpendicular to ridge direction is not included as it does not contribute to the lateral load or overturning moment in the direction perpendicular to the trusses.

The impact of the uplift component on the response of roof systems with toe nails or hurricane ties under combined loading has been extensively studied by others (Riley & Sadek 2003, Scoville 2005, Kopp 2010, and Simpson Strong-Tie 2010) and, therefore, is not included in this testing program. In addition, the wind uplift forces do not have direct effect on the unblocked diaphragm action, cross-grain bending of roof members, truss rotation, and blocking performance -- the primary areas of this study. Furthermore, significant spatial variations of wind uplift pressure exist across the roof surface and the wind profiles developed specifically for design purposes (i.e. ASCE 7) may not be directly applicable to full-scale roof testing for combined loading applications. Their effects should be captured more accurately through full-size wind tunnel testing.

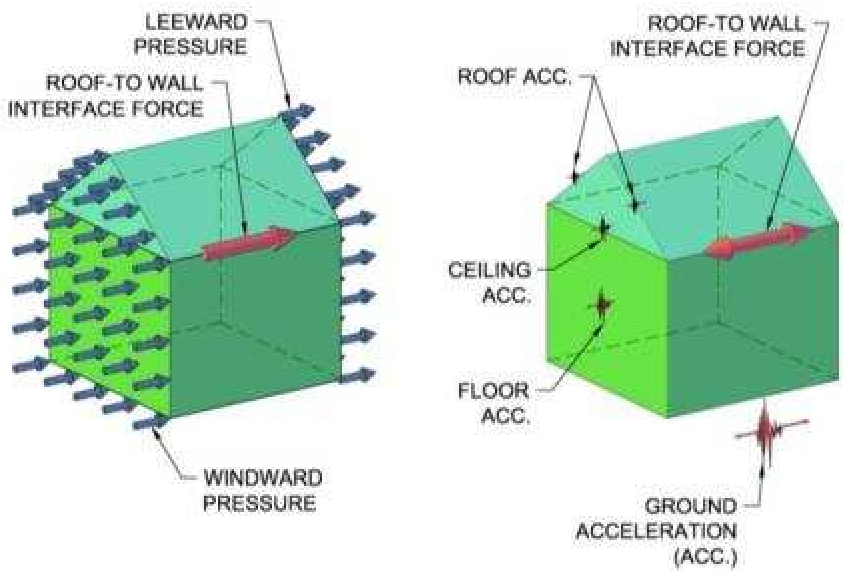


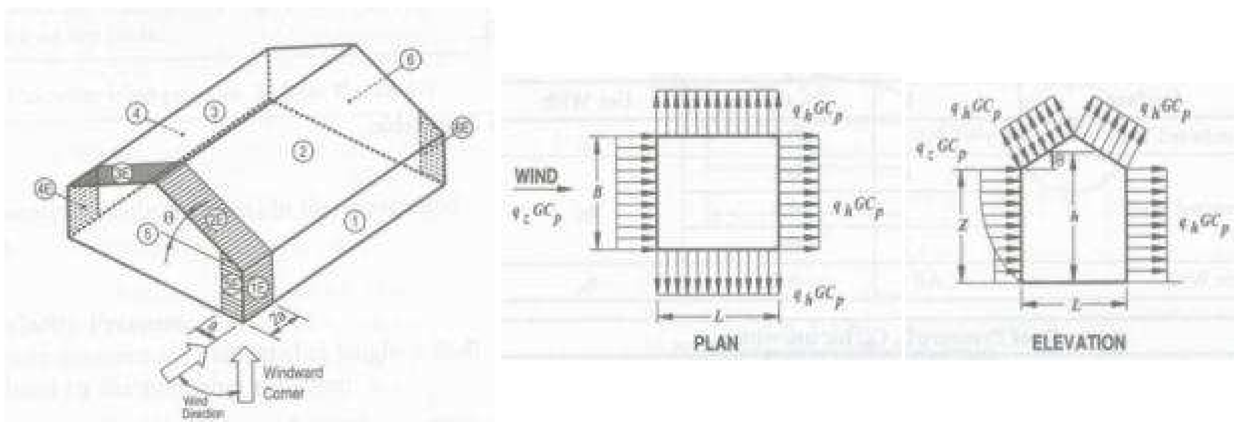
Figure 2 – Wind and seismic loading in the parallel to wind direction

(Uplift and orthogonal components are not shown for clarity)

Lateral forces from wind and seismic events are generated through two different mechanisms and imposed on the structure in two different manners. Therefore, the loading type has implications on the selection of appropriate testing procedures. Wind pressures act on the building surfaces whereas seismic forces act at the location of the masses of the building elements. The discussion below identifies the unique features of each loading type with respect to the forces acting at the roof-to-wall connections.

Wind Considerations

The ASCE 7 lateral wind load profile in the direction parallel to the ridge is shown in Figure 3 with wind pressures acting on both the windward wall surface (positive pressure) and leeward wall surface (negative pressure – suction) of the building. For a typical residential floor aspect ratio, the windward pressure is about twice the leeward pressure. These pressures have the same vector direction and their actions are superimposed to develop the total lateral force acting on the building. The wind pressures on the wall surface are transferred through the vertical framing members to roof, ceiling and floor diaphragms and from there to shear walls. This discussion is focused on the forces transferred from the gable end walls into the roof assembly and into the walls below the roof assembly. The forces from gable end wall wind pressures are resisted by the roof sheathing diaphragm and the ceiling diaphragm. Figure 4 shows the tributary wind areas associated with each diaphragm. The applicable forces from the top story wall are transferred into the ceiling diaphragm. Typical bracing details at the gable end wall are intended to redistribute the load into the diaphragms and may to some degree affect the distribution of forces between the roof sheathing diaphragm and the ceiling diaphragm. This bracing may also impart some amount of rotational restraint to the roof members near the gable ends. Conversely, this restraint contribution would be minimal at the interior roof sections.



(a) Directional Procedure (b) Envelope Procedure

Figure 3 – ASCE 7 wind loading profile

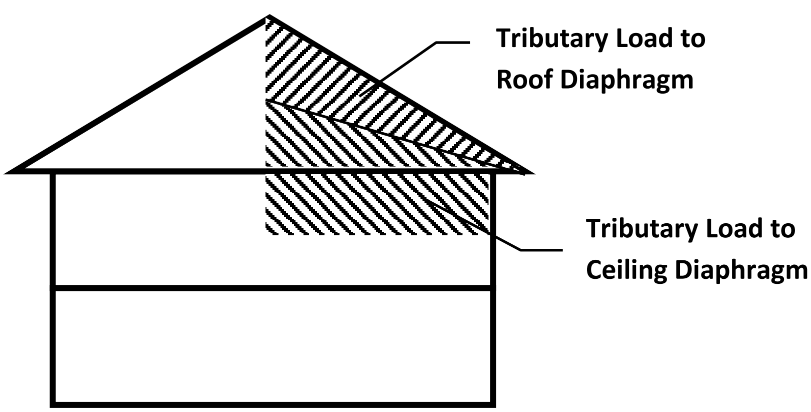


Figure 4 – Tributary area of wind loads

Eccentricity at the connection depends on the ratio of load delivered to the truss heel through the ceiling to the load delivered through the roof diaphragms, with a higher ratio indicating that there is less load transferred through the roof sheathing and therefore less overturning load at the truss heel. Each diaphragm resists a part of the total load and the ratio of the loads between the two diaphragms varies depending upon the roof configuration and location of the gable end wind load resultant. Example 1 is provided below to calculate the ratio of load to the two diaphragms for a typical house configuration. For a house with a 5/12 roof pitch and a roof span of 32 feet, the resultant loading ratio of ceiling diaphragm load to roof diaphragm load is 3.2 to 1, i.e., the force transferred through the ceiling diaphragm is 3.2 time greater than the force transferred through the roof diaphragm. For a roof pitch of 7/12 for the same house configuration, the loading ratio is 2.5 to 1.

Example 1 – Calculation of the ratio of wind load to roof diaphragms

Building Configuration:

Building plan: 32 feet by 50 feet
Two stories
Story height: 9 feet
Roof pitch: 5:12
Mean roof height: 25 feet
Gable end roof
Wind parallel to ridge
Overhang: 2 feet

Basic Wind Speed: 140 mph (ASCE 7-10)

Exposure: B

Wind pressure: 30 psf (rounded for simplicity of calculations)

Gable end wall forces:

Total gable end area:

(Dist. from top of wall to ridge) x (Roof width) x 0.5
[((32’/2)+2’)(5/12)] x (32’+4’) x (0.5) = 135 ft^2

Force into each diaphragm from gable end:

(Simply-supported, vertical framing members spanning between two diaphragms)
(135 ft^2) x (30 psf) x (0.5) = 2,025 lb

2^{nd} Story wall forces:

Tributary wall area of the top story:

(Building width) x (Wall height) x 0.5
(32’) x (9’) x (0.5) = 144 ft^2

Force into ceiling diaphragm from top story wall:

(144 ft^2) x (30 psf) = 4,320 lb

Ratio of roof diaphragm load to ceiling diaphragm load:

1 : 3.2

The same example for a roof pitch of 7/12 results in:

1 : 2.5

Seismic Considerations

Unlike wind pressures that are imposed on the outside shell of the structure, seismic load is distributed throughout the building based on the mass of the elements. The eccentricity at the truss heel connection is still governed, however, by the ratio of loading between the two diaphragms. The total force resisted by the roof-to-wall connections is associated with (1) the mass of the roof diaphragm, (2) the mass of the ceiling diaphragm and (3) the mass of half the height of the top story walls below the roof gable end. The walls supporting the roof eave do not directly contribute to the overturning forces at the roof heel.

The distribution of the weight between the two diaphragms is close to symmetric with the weight of the sheathing materials (OSB and gypsum) and the primary framing members being located around the perimeter of the roof triangle. The weight of the shingles is attributed to the top chords, and the weight of insulation, eaves, overhangs, any mechanical equipment is attributed to the bottom of the roof assembly. Example 2 below shows that the top story walls contribute only 10% to the total lateral force resisted by the ceiling diaphragm and that contribution is ignored for the purposes of this study. Therefore, the resultant loading ratio of ceiling diaphragm load to roof diaphragm load is about 1 to 1, i.e., half of the total force is transferred through the ceiling diaphragm and half through the roof sheathing diaphragm.

Example 2 – Calculation of the eccentricity of seismic load

Building Configuration:

Building plan: 32 feet by 50 feet
Two stories
Story height: 9 feet
Roof pitch: 5:12
Mean roof height: 25 feet
Gable end roof
Acceleration parallel to ridge
Overhang: 2 feet

Roof dead load: 15 psf
Wall dead load: 9 psf

Roof weight:	(15 psf) x (32'+2') x (50') = 25,500 lb	(half overhang is used at 15 psf dead load)
Wall weight:	(9 psf) x (32') x (9'/2) x (2 walls) = 2,592 lb	

Ratio of roof diaphragm load to ceiling diaphragm load:

1 : 1

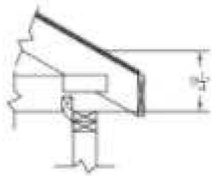
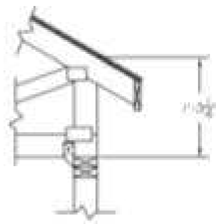
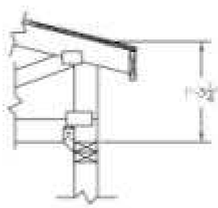
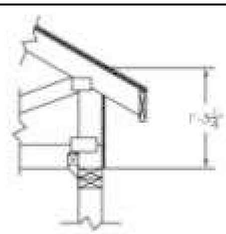
METHODS AND MATERIALS

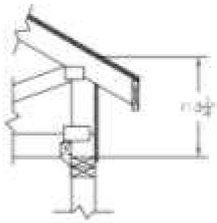
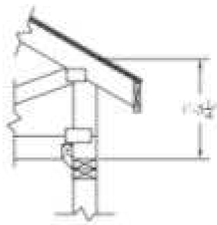
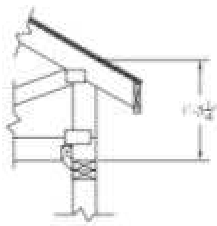
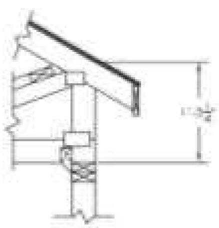
General

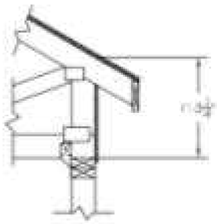
Testing was conducted at the NAHB Research Center Laboratory Facility located in Upper Marlboro, MD. All specimens were constructed in the laboratory and all construction materials were purchased from local suppliers.

Table 2 provides a test matrix summarizing specimen configurations including connections, truss heel height, and blocking. A purpose statement with explanation is provided for each configuration. A total of nine (9) full size roof systems were tested with various levels and types of heel detailing to measure the lateral performance of the roof-to-wall interface.

Table 2 – Test Matrix

Configuration	Diagram	Truss / Heel Height	Roof-to-Wall Attachment	Blocking/Bracing at Heel	Purpose	Notes
A –Low-heel truss		Low-heel truss at 24" oc H=9¼"	H2.5T hurricane clip at each truss	None	- Benchmark performance of typical low-heel trusses - Benchmark performance of unblocked roof and ceiling diaphragms	This configuration will allow establishing performance limits on the conventional low-heel roof system with hurricane clip connections
B – High-heel truss without blocking		High-heel truss at 24" oc H=15¼"	H2.5T hurricane clip at each truss	None	Investigate the impact of a high heel truss through comparison to Configuration A	This configuration will allow establishing limits on high-heel conditions constructed w/o blocking/bracing
C – High-heel truss without blocking with low (3/12) roof pitch		High-heel truss at 24" oc H=15¼"	H2.5T hurricane clip at each truss	None	Through comparison to configuration B, the impact of low roof slope (presumed decreased rotational restraint from the roof sheathing) will be investigated	This configuration will demonstrate whether roof slope has a measurable impact on contribution of roof sheathing to overturning resistance of the roof heel
D – High-heel truss braced with OSB sheathing		High-heel truss at 24" oc H=15¼"	H2.5T hurricane clip at each truss	Truss heel is braced with OSB sheathing (OSB is not extended over wall top plate)	Evaluate the effectiveness of continuous OSB sheathing nailed to the truss heel in restraining truss overturning	This configuration represents an alternative to the blocking details in the 2012 IRC

Configuration	Diagram	Truss / Heel Height	Roof-to-Wall Attachment	Blocking/Bracing at Heel	Purpose	Notes
E – High-heel truss braced with OSB sheathing extended over wall plate		High-heel truss at 24” oc H=15¼”	H2.5T hurricane clip at each truss	Truss heel is braced with OSB sheathing (OSB is attached to the wall top plate)	Same as above, but the OSB sheathing extended over the top plate of the double top plate and attached with an additional horizontal row of nails into top plate	This configuration represents an alternative to the blocking details in the 2012 IRC
F – High-heel truss with blocking at intermittent locations		High-heel truss at 24” oc H=15¼”	H2.5T hurricane clip at each truss	Partial height blocking (25% of wall length)	Evaluate the condition where blocking is installed in an intermittent configuration (this configuration represent a scenario where blocking is installed at braced wall panels only)	These configurations are intended to evaluate the use of intermittent versus continuous blocking
G – High-heel truss with blocking at every other bay		High-heel truss at 24” oc H=15¼”	H2.5T hurricane clip at each truss	Partial height blocking (50% of wall length)	Evaluate the condition where blocking is installed at every other bay	
H –High-heel truss with braced webs		Low-heel truss at 24” oc H=15¼”	H2.5T hurricane clip at each truss	Diagonal bracing of truss webs	Evaluate the effectiveness of web bracing in restraining truss overturning at the heel	This configuration is intended to evaluate the contribution of web bracing that may already be present in the roof assembly in resisting other forces

Configuration	Diagram	Truss / Heel Height	Roof-to-Wall Attachment	Blocking/Bracing at Heel	Purpose	Notes
I – High-heel truss braced with OSB sheathing and a reinforced ceiling diaphragm		High-heel truss at 24" oc H=15¼"	H2.5T hurricane clip at each truss	Sheathing extended down to capture top plate	Evaluate performance of Configuration E blocking method with reinforced gypsum diaphragm	The ceiling diaphragm is reinforced in an attempt to force failure at the truss heel connection instead of the gypsum diaphragm. This test is intended to validate the performance of OSB bracing detail at higher diaphragm capacities (i.e., higher overturning force at the heel).

Specimen Construction

Table 3 provides a summary of materials and methods used in construction of the specimens and Table 4 provides details of the various blocking/bracing methods.

Each specimen was constructed with five (5) 24-foot span wood trusses spaced at 24 inches on center, with the overall size of the full roof system at 24 feet wide by 8 feet deep with additional 16-inch long overhangs on each side (Figure 5). Trusses were supported at the heel by 4-foot high light-frame knee walls anchored to the lab's strong floor. The strength and stiffness of the knee walls was sufficiently higher than that of the roof system to prevent any significant deformations in the supporting structure. The roof trusses were attached to the double top plate of the knee wall in accordance with fastening schedules specified in Tables 3 and 4. The double top plate was attached to the knee wall framing with bolts and was replaced after each test.

For all tests, the truss bottom chords were connected to the top plates of the supporting knee walls using H2.5T hurricane clips. The clips on both ends of a truss were installed on the same face of that truss; the installation face was alternated between adjacent trusses (Figure 5) to eliminate any directional bias in the resistance behavior of the clips. A continuous 1x6 nominal fascia board was installed on both sides of the specimens.

The 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 (one inch 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 row of fasteners was located approximately 8 inches from each knee wall (i.e., floating edges) in accordance with the Gypsum Association's *Application and Finishing of Gypsum Panel Products (GA-216-2010).* All interior gypsum panel joints were taped and mudded, no finishing was done at the interface of the ceiling and the knee walls. The ceiling diaphragm of Specimen I was reinforced at the front and back trusses with a double top plate boundary member and 2x nailing member (Figure 6(f)). The fastener spacing of the ceiling diaphragm in Specimen I was also reduced from 12 to 8 inches on center to increase the diaphragm's capacity. (Specimens A through H were constructed without these additional boundary/chord members.)

Table 3 – Specimen Materials and Construction

Roof Span:	24 feet (plus 1 foot 4 inch overhang on each end)
Roof Length:	8 feet
Roof Pitch:	7/12 or 3/12 Per test matrix (Table 2)
Roof Framing Members	Metal plate connected wood trusses fabricated with No. 2 Southern Yellow Pine lumber; Heel heights either 9¼ inches or 15¼ inches per test matrix (Table 2)
Truss Spacing:	24-inches on center
Truss-to-Wall Connections:	Simpson Strong-Tie H2.5T hurricane truss clips connecting each truss and both top plate members with a total of ten (10) 8d common (2½" x 0.131") nails (5 nails per each truss)
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 taped and mudded
Ceiling Fasteners:	1-5/8 inch Type W drywall screws: - *Configurations A through H* - 12 inches on center w/ first rows of fasteners 8 inches in from side walls (i.e., floating edges) - *Configuration I* - 8 inches on center w/ first rows of fasteners 8 inches in from side walls (i.e., floating edges)
Ceiling Boundary Chord:	*Configuration I only* – 2x4 double chord member face-nailed together w/ 10d (3" x 0.128") at 24 inches on center and eight (8) 16d (3½" x 0.135") in spliced sections. Outer trusses toe-nailed to double chord member w/ 8d box (2½" x 0.113") at 6 inches on center.
Knee Wall Framing (including top plates):	2x4 nominal SPF No. 2 grade lumber
Knee Wall Sheathing:	7/16-inch-thick OSB sheathing attached with 8d common (2½" x 0.131") at 3 inches on center on panel perimeter and 12 inches on center in the panel field

Table 4 – Truss Blocking/Bracing Construction Details

Configuration	Blocking/Blocking	Connection
High Heel braced w/ OSB	7/16 inch OSB, 10½ inches wide by 8 feet long	Face-nailed to each truss heel with three (3) 8d common (2½" x 0.131")
High Heel braced w/ OSB attached to Top Plate	7/16 inch OSB, 11½ inches wide by 8 feet long	Face-nailed to each truss heel w/ three (3) 8d common (2½" x 0.131"); Face-nailed to top member of double top plate w/ 8d common (2½" x 0.131") at 6 inches on center
High Heel w/ 25% Blocking	1-1/8–inch-thick by 14inch-high iLevel Rim Board contact fit between trusses	End-nailed to trusses w/ two (2) 16d box (3½" x 0.135"); Toe-nailed to top plate w/ five (5) 8d box (2-3/8" x 0.113") at 6 inches on center
High Heel w/ 50% Blocking		
High Heel w/ Diagonal Web Bracing	2x4 SPF No.2 Grade lumber	Face-nailed to truss web w/ two (2) 8d common (2½" x 0.131")

Figure 6 provides details of the various blocking methods evaluated in this testing program. All blocking methods were in addition to the typical roof specimen described above.

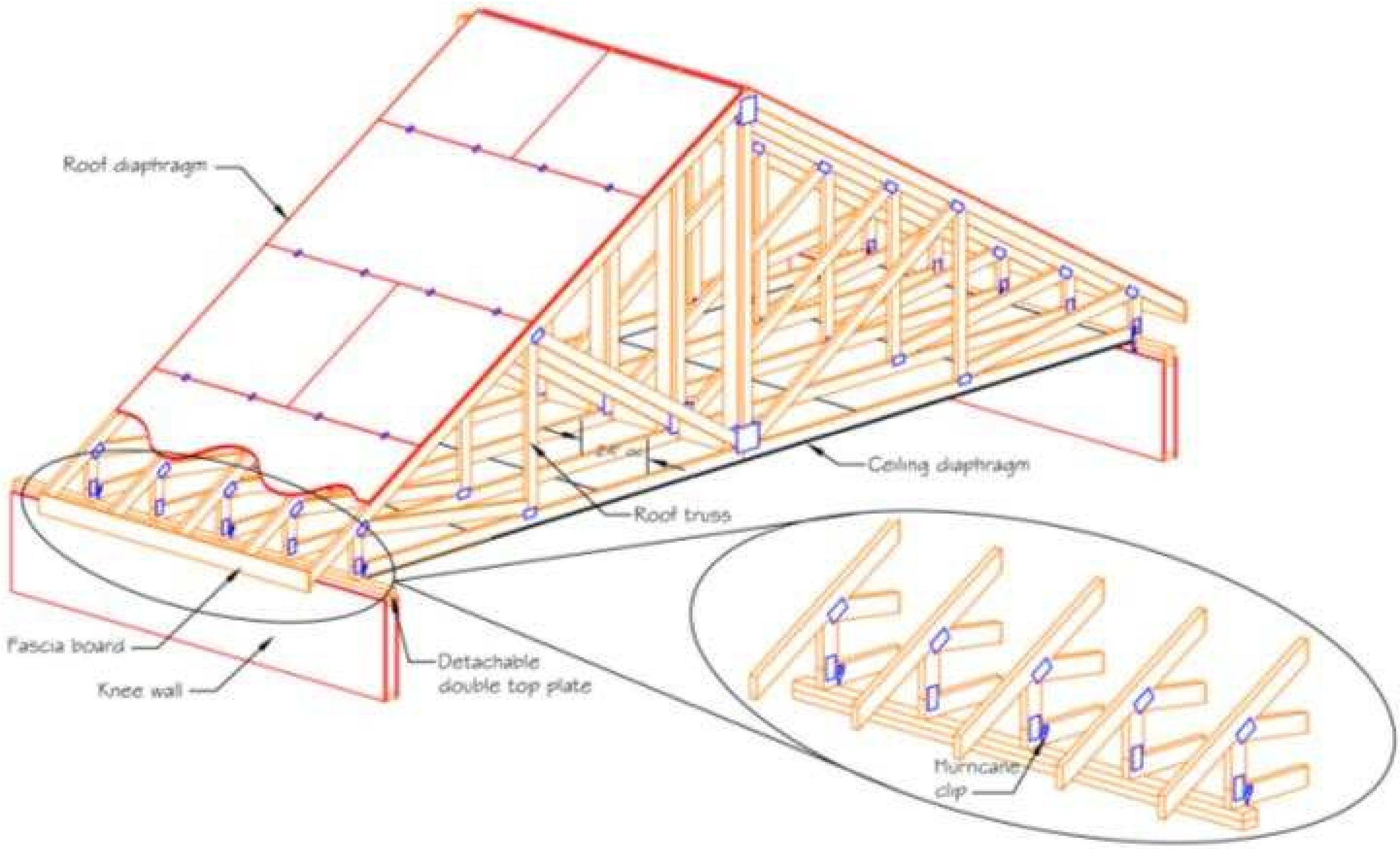
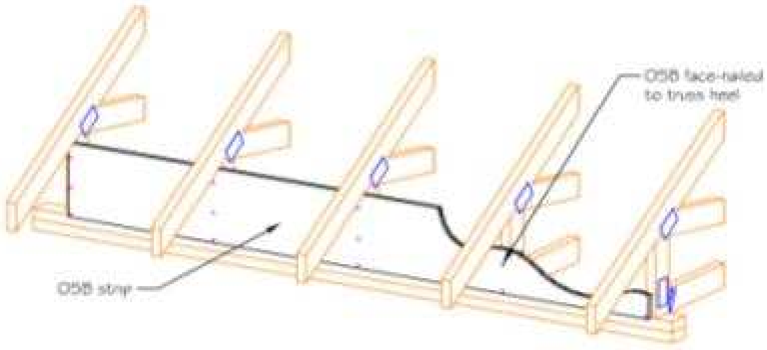
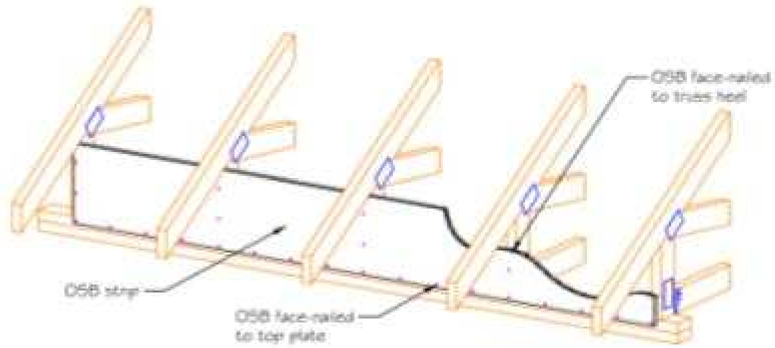


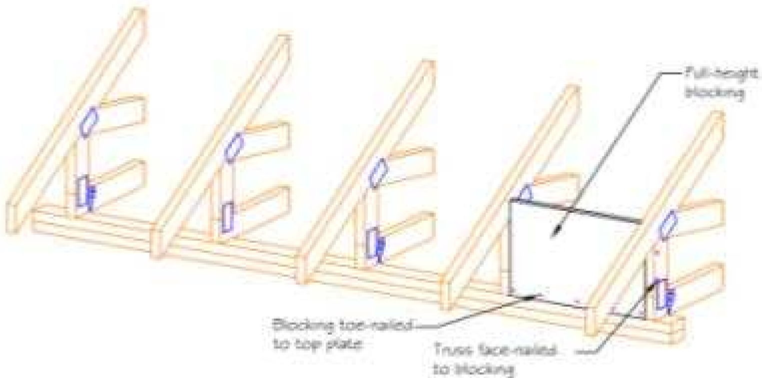
Figure 5 – Specimen construction



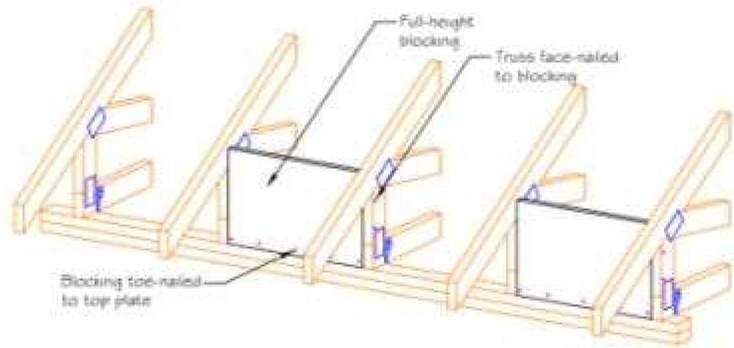
(a) High Heel w/ OSB



(b) High Heel w/ OSB to Plate

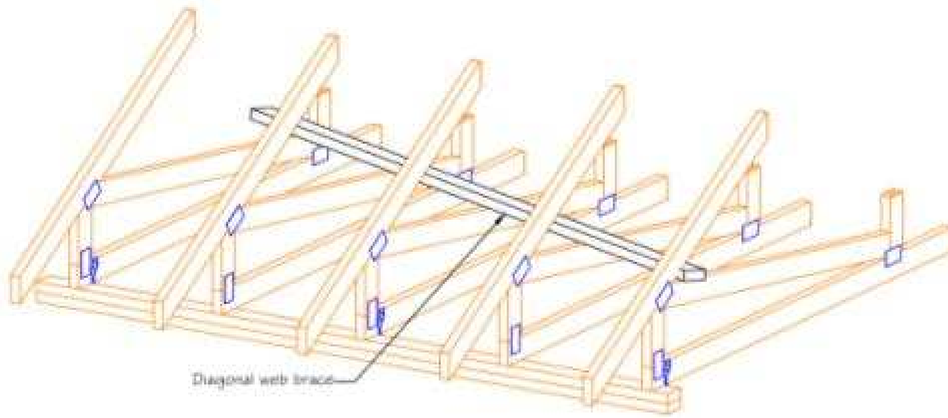


(c) High Heel w/ 25% Blocking

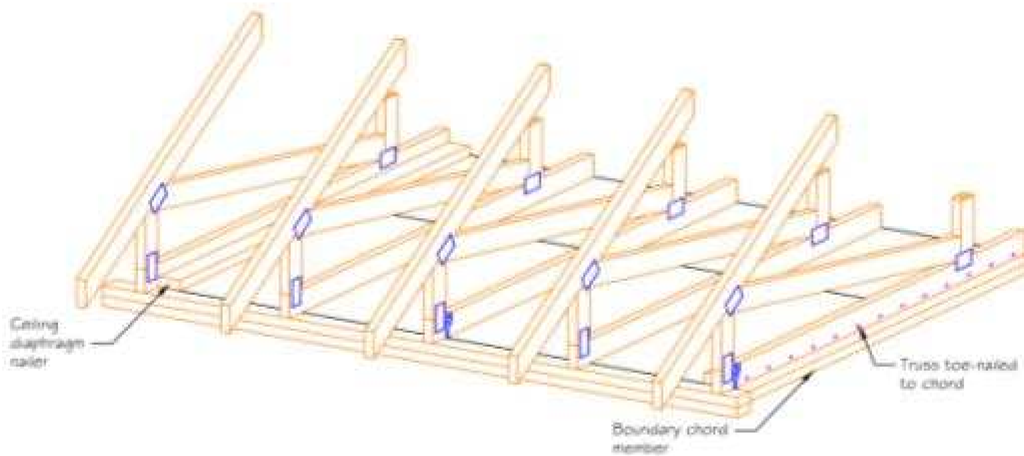


(d) High Heel w/ 50% Blocking

Figure 6 – Truss blocking details



(e) High Heel w/ Diagonal Web Bracing



(f) High Heel w/ OSB to Plate and Reinforced Ceiling Diaphragm
(OSB not shown for clarity)

Figure 6 (cont) – Truss blocking details

Test Setup and Protocol

Figure 7 shows the test set-up including the specimen, loading brace, and instrumentation. Figure 8 provides a photograph of the test set-up, reaction frame, and data acquisition system.

Load was applied to the specimen through permanent truss bracing (2x6 nominal Southern Pine, No. 2 Grade lumber) attached at mid-height of the center vertical web member of each truss. 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. Load was applied at a mid-height permanent bracing location that yielded a 1:1 roof diaphragm to ceiling diaphragm loading ratio (i.e., the loading ratio caused by a seismic loading scenario). This loading condition results in the highest eccentricity at the heel such that observations on the effectiveness of the tested heel blocking/bracing options are appropriate for a broad range of applications.

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.

The loading brace members were loaded in tension using a computer controlled hydraulic cylinder mounted to a steel reaction frame. The reaction frame was attached to the laboratory structural floor. Load was applied monotonically in tension at a constant displacement rate of 0.06 inches per minute to allow for sufficient visual observations throughout the test and was measured using an electronic load cell installed between the cylinder and the loading bracket. Displacement was continued until failure, defined as a 20% drop in load from the peak.

Displacements of the roof system relative to either the supporting knee walls or the laboratory structural floor were measured using electronic Linear Motion Position Transducers (LMPT's) at several locations, including:

- The ceiling diaphragm at mid-span of the roof/truss assembly;
- The top and bottom of the heel on the first/front truss at both ends;
- The top and bottom of the heel on the fourth truss at both ends (Specimen F & G only), and,
- The bottom of the heel on the fifth/rearmost truss.

Displacement of the top of the supporting knee walls was also measured relative to the structural floor using LMPT's. Finally, displacement at the peak of the roof/truss assembly was measured relative to the steel reaction frame using a string potentiometer. Uplift at the rear of the specimen was not measured; initial tests showed that uplift was minimal due to the vertical restraint provided by the hurricane clip connections.

All load and displacement measurements were recorded using an electronic data acquisition system.

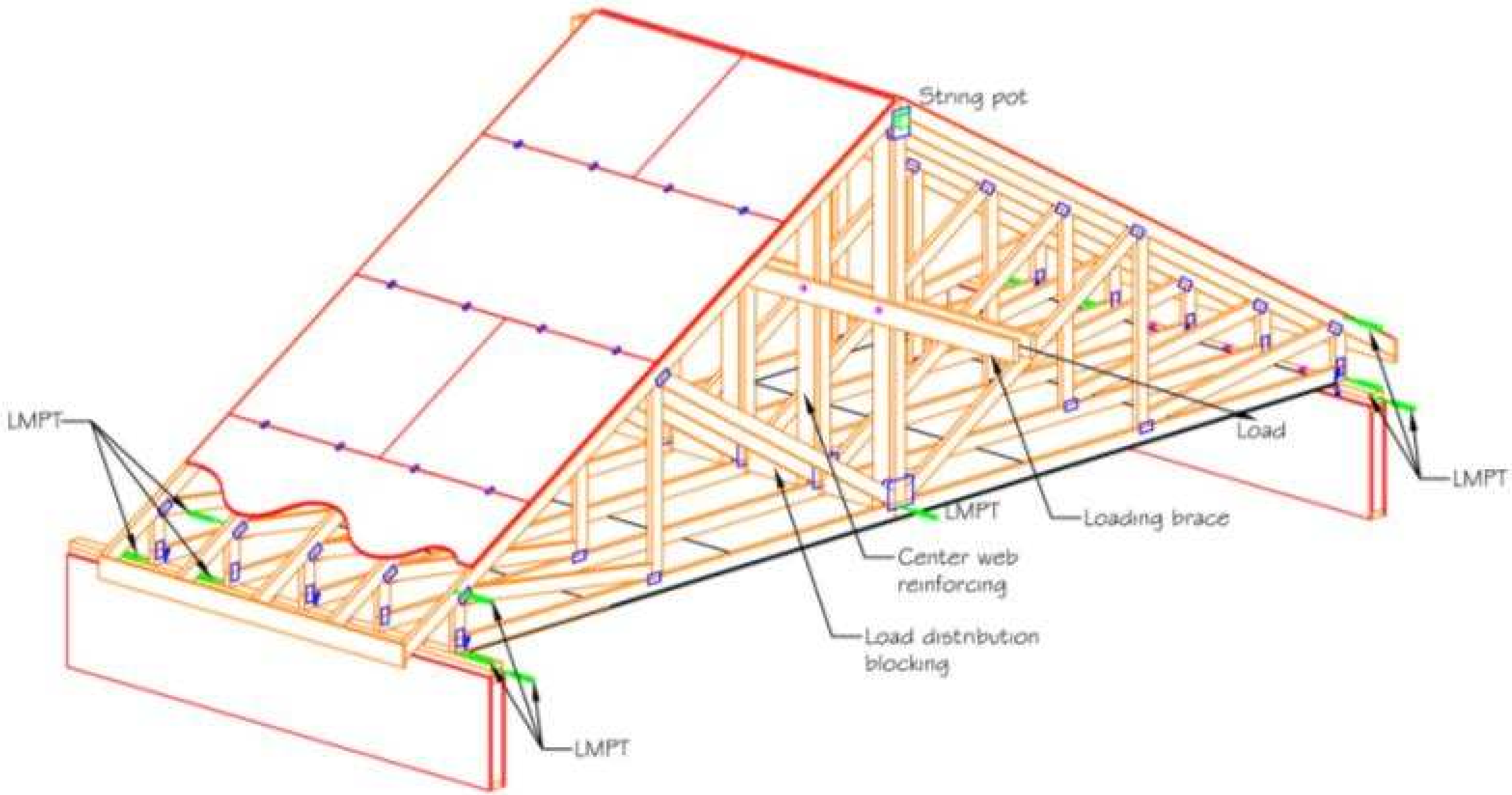


Figure 7 – Test set-up and instrumentation

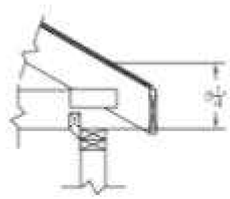
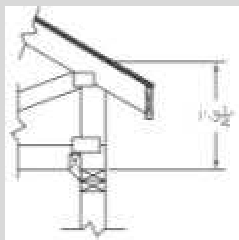
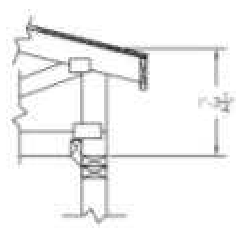
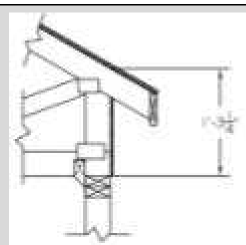
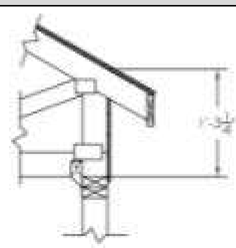
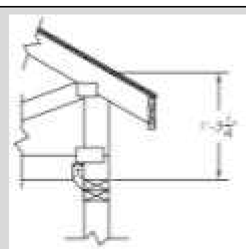
Figure 8 – Photo of test setup

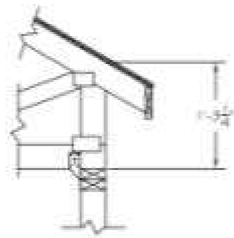
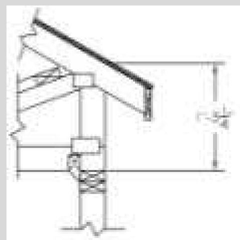
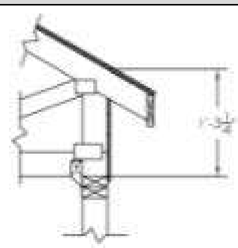
RESULTS

The results of the testing are summarized in Table 5 including the peak load reached by the roof assembly and the unit peak capacity of the truss-to-wall connections. Table 5 also includes initial stiffness values for each specimen determined from the displacement at the top of the truss heel (TOH) measured relative to the top plate. The initial stiffness was calculated at a 760 lb load level, selected as an approximate representation of the linear range for performance comparison between systems tested in this study. Figures 9 and 10 provide visual comparisons of peak capacity and stiffness, respectively, for the various specimens tested. (Note: Figure 10 shows specimen load versus displacement curves where displacement was measured at the peak of the specimen, not at the top of the truss heel). See Appendix A for summary figures of load versus displacement curves measured at the TOH location. Appendix B provides several load versus displacement curves for each individual specimen, measured at various locations on the specimen including the midpoint of both the top and bottom chords of Truss 1, the left and right TOH of Truss 1 and where applicable, the left and right TOH of Truss 4.

A discussion of each of the individual tests is provided in this section, including discussion of peak capacities and initial stiffnesses relative to baselines (where applicable) and observed governing failure modes. Visual observations regarding rotation of the trusses are noted as part of the failure mode discussion. Additional analysis of the rotation/displacement of the truss heels, as well as comparisons of peak capacities to typical design loads, is summarized and presented at the end of this section.

Table 5 – Test Results

Configuration	Diagram	Peak Load (lb)	Peak Load per Truss Connection (lb)	Unit Peak Capacity (lb/ft)[1]	Initial Stiffness (lb/in)[2]
A – Low-heel truss		5,140	514	255	8,828
B – High-heel truss without blocking		3,525	352	175	4,432
C – High-heel truss without blocking with low (3/12) roof pitch		3,780	378	190	3,950
D – High-heel truss braced with OSB sheathing		4,344	434	215	10,395
E – High-heel truss braced with OSB sheathing extended over wall plate		4,755	475	240	32,224
F – High-heel truss with blocking at intermittent locations		3,988	399	200	23,548

Configuration	Diagram	Peak Load (lb)	Peak Load per Truss Connection (lb)	Unit Peak Capacity (lb/ft)[1]	Initial Stiffness (lb/in)[2]
G – High-heel truss with blocking at every other bay		4,520	452	225	26,581
H – High-heel truss with braced webs		3,633	363	180	5,469
I – High-heel truss braced with OSB sheathing and a reinforced ceiling diaphragm		6,794	679	340	41,362

1. Unit peak capacity is calculated by dividing the peak load per connection by the typical 2’ truss spacing (i.e., the tributary area of a typical truss)
2. Initial stiffness measured at roof peak of the specimen.

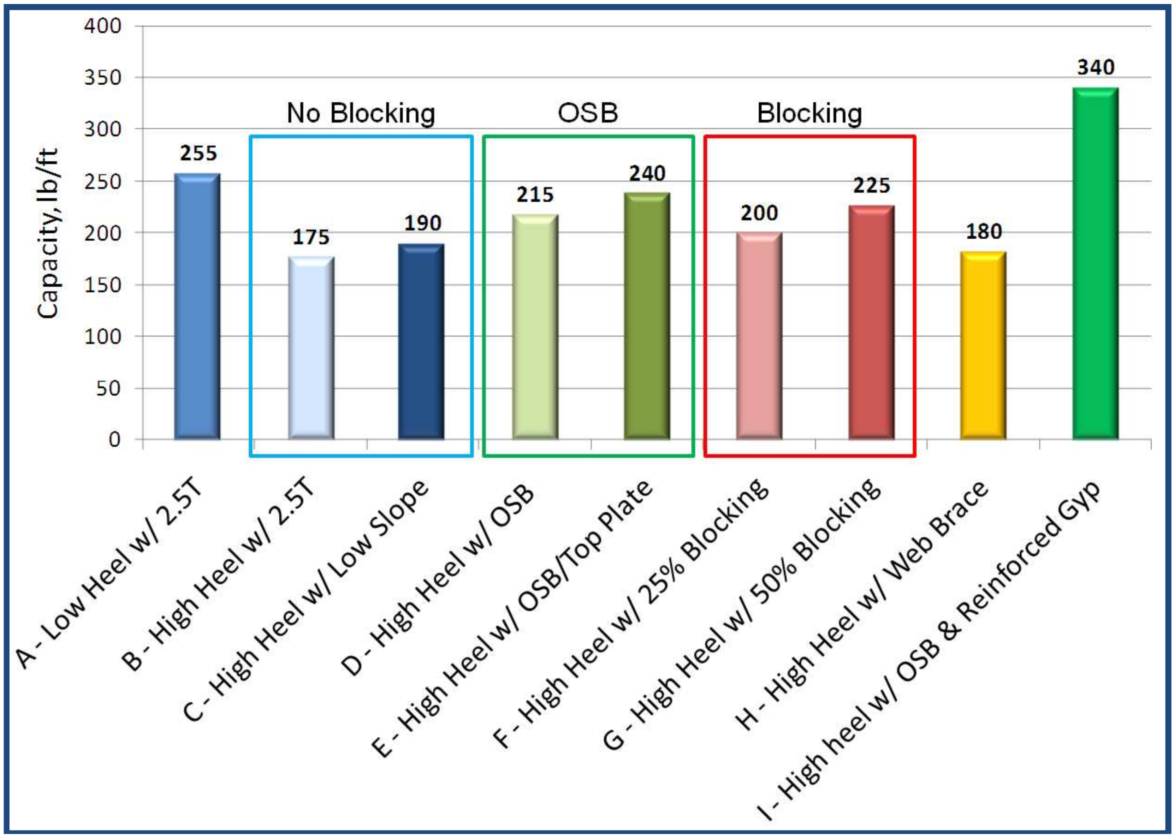


Figure 9 – Comparison of peak capacities per truss connection

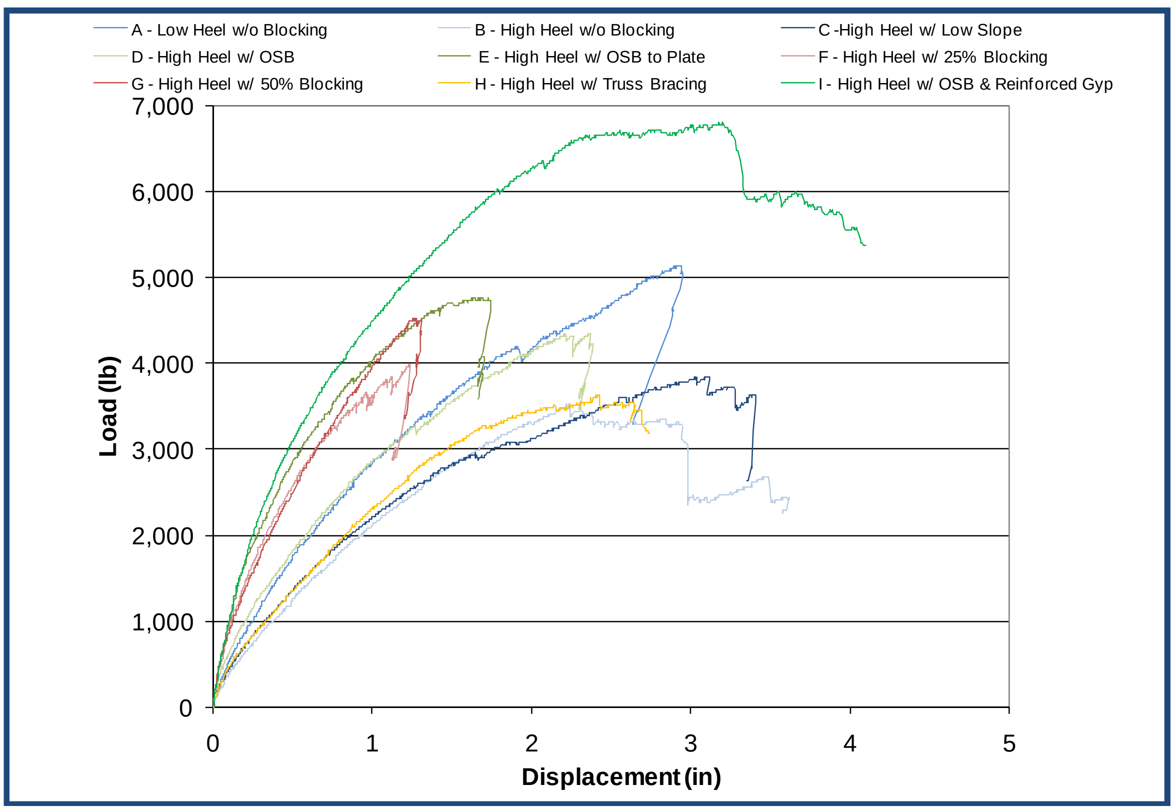


Figure 10 – Load vs. displacement curves (measured at peak of the roof)

Specimens A through C were intended to establish baseline capacities and performance characteristics for low- and high-heel roof systems without blocking or bracing the truss heel. The low-heel truss configuration (Specimen A) represents the highest allowable heel height by code that does not require blocking. Specimen A achieved a peak load of 5,140 lb (peak unit capacity of 255 lb/ft) and initial stiffness of 8,828 lb/in. Failure of Specimen A included initial fastener tear-through at the outer ends of the ceiling diaphragm near the knee walls followed by complete failure of the fasteners in the center gypsum panel (see Figures 11 & 12). Only minor rotation of the truss heels was observed during testing as well as minor rotation and buckling of the hurricane clips (Figure 13). Minor displacement of the truss top chord relative to the bottom chord at the heel joint was also observed; along with slight deformation of the metal connector plate at the heel joint (Figure 14).

Figure 11 – Gypsum fastener tear-through (Specimen A)

Figure 12 – Complete failure of gypsum fasteners (Specimen A)

Figure 13 – Rotation of hurricane clip at failure (Specimen A)

Figure 14 – Truss member displacement and metal connector plate deformation at heel joint (Specimen A)

Specimen B represents the second trigger height specified by code (see Table 1). Trusses with heel heights between the 9¼-inch height of Specimen A and the 15¼ -inch height of Specimen B are currently required to have solid blocking between each truss when framed over top of a braced wall panel. Specimen B omitted this blocking in order to compare the effect of the higher heel height on roof system capacity with the Specimen A results, as well as to establish a baseline performance benchmark against which the various blocking / bracing details evaluated in this study could be measured. Specimen B reached a peak load of 3,525 lb (peak unit capacity of 175 lb/ft) and initial stiffness of 4,432 lb/in. This is a 33% drop in capacity and a 50% drop in initial stiffness from the results of Specimen A, illustrating the effect of an increased heel height on the global response for a system without blocking. The primary failure mode was again tear-through of the gypsum panel fasteners at both ends of the specimen near the knee walls. All of the gypsum panels, however, remained intact and attached to the framing members throughout the test. Significant rotation of the trusses at the heel connections as well as minor buckling of the hurricane clips was also observed.

Specimen C was designed to evaluate the effect of a lower roof slope on the performance of the truss-to-wall connections. Specimen C was similar in construction to Specimen B and reached a peak capacity of 3,780 lb (peak unit capacity of 190 lb/ft) and initial stiffness of 3,950 lb/in. Specimen C exhibited similar damage and failure modes as Specimen B (i.e., gypsum fastener tear-through, significant rotation observed at truss heel). Additional damage was also observed in the form of buckled hurricane clips (Figure 15) and member separation at the heel joint metal plate connectors. Comparisons of both peak capacity and initial stiffness values between Specimen B (7:12 roof slope) and Specimen C (3:12 roof slope) yields a less than 10% difference in strength and stiffness performance between the two specimens, indicating that the degree of roof slope has a minimal effect on heel connection performance. Figure 16 provides a photo of the truss rotation observed in Specimen C at failure.

Figure 15 – Buckling of hurricane clip (Specimen C)

Figure 16 – Specimen C heel rotation at failure

Specimens D and E were designed to investigate the contribution of OSB sheathing installed on the exterior face of the truss heel. The OSB sheathing in Specimen D was not attached to the top plates of the supporting knee walls and only provided rotational restraint to the truss heel. The OSB strip in Specimen E was extended down and nailed to the upper member of the wall double top plate, and as such provided both a rotational restraint for the trusses as well as an additional load transfer mechanism from the trusses to the supporting wall. Specimen D reached a peak capacity of 4,340 lb (peak unit capacity of 220 lb/ft) and an initial stiffness of 10,395 lb/in. This peak capacity is a 26% increase over the capacity of Specimen B and only 16% less than the peak capacity of the low heel configuration. The increase in performance due to the OSB bracing strip is also evident when comparing initial stiffness values; the addition of the OSB bracing strip increased the initial stiffness by 18% over the low-heel baseline specimen and by a factor of 2.3 over the unblocked high-heel baseline specimen. The primary failure mode was again fastener failure in the gypsum panels. Some fastener tear-through occurred at the edges of the OSB bracing strip. The bracing of heel joints with OSB was also effective in controlling rotation of the truss. See Figure 17 for a comparison of the truss heel position prior to testing and after gypsum failure. Specimen E exhibited the same failure modes as Specimen D while achieving a peak capacity of 4,760 lb (or a peak unit capacity of 240 lb/ft) and initial stiffness of 32,224 lb/in. While this peak capacity from Specimen E is 7% less than the peak capacity of the low-heel configuration (Specimen A), the additional nailing of the OSB bracing strip to the supporting wall below in Specimen E increased the initial stiffness of the specimen threefold over both the OSB bracing strip without top plate nailing and the benchmark low-heel specimen (3.1 times and 3.6 times greater, respectively).

(a) Before test

(b) Post testing

Figure 17 – Comparison of rotation at Truss 5 (Specimen D)

Figure 18 – Truss rotation of Specimen E at failure (Truss 5)

Specimens F and G were designed to evaluate the performance of code compliant blocking details. Specimen F included a solid blocking panel installed in a single truss bay on each side (i.e., 25% of the specimen wall length). Specimen G was constructed with alternating blocked and unblocked bays, resulting in blocking of two (2) of the bays per side (i.e., 50% of the specimen wall length). The 25% blocking specimen (Specimen F) reached a peak load of 3,988 lb (unit peak capacity of 200 lb/ft) and an initial stiffness of 23,548 lb/in and exhibited gypsum fastener failure as its primary failure mode as was observed and described for previous specimens. The same moderate rotation was observed at both the front two trusses (where blocking was installed) and the fourth (where no blocking was installed). This visual observation was confirmed through displacement measurements at both locations indicating that the single blocked bay provides rotational restraint to the entire specimen (i.e., the effect of the blocking is not localized). Figure 19 shows a comparison of the rotation of the first and last trusses.

(a) Truss 1 (b) Truss 5

Figure 19 – Comparison of truss heel rotation (Specimen F)

The 50% blocked specimen (Specimen G) reached a peak load of 4,520 lb (unit peak capacity of 225 lb/ft) and an initial stiffness of 26,581 lb/in and exhibited similar failure modes as Specimen F, including the primary failure mode of gypsum fastener tear-through. Some rotation of the trusses relative to the blocking panels was also observed, as shown in Figure 20. When compared to the tested benchmark capacities of Specimens A and B, the 50% blocked specimen exhibited a 28% increase in capacity over the unblocked high-heel specimen and showed a 12% decrease in capacity compared to the unblocked low-heel specimen. Interestingly, the addition of the second blocking panel only resulted in a 12% increase in stiffness over the 25%-blocked specimen (Specimen F). Both blocked specimens, however, exceeded the low-heel baseline configuration by more than a factor of 2.5.

Comparison between peak capacities and initial stiffness values of Specimens E through G shows that the use of an OSB bracing strip attached to the face of the truss heel and tied to the supporting wall below yields slightly greater performance than both the 25% and 50% blocking options.

Figure 20 – Rotation of truss heel at blocking location (Specimen G)

The purpose of Specimen H was to evaluate the effect of diagonal truss web bracing on the performance of the high-heel connection, particularly with regard to rotation. The diagonal bracing specimen reached a peak load of 3,633 lb (unit peak capacity of 180 lb/ft). There were no observed differences in response and failure mode compared to Configurations B and C (high heel without blocking or bracing) indicating that web bracing does not provide a mechanism for resisting the rotation of high-heel trusses.

Specimen I was designed to further evaluate the OSB blocking method by testing it in conjunction with a reinforced ceiling diaphragm. The intent was to validate the effectiveness of the OSB bracing option in applications with a stronger gypsum diaphragm, while also attempting to force failure in the truss heel joint and truss-to-wall connections. Testing of Specimen I yielded a peak load of 6,794 lb (unit peak capacity of 340 lb/ft). Tear-out failure of the OSB-to-heel fasteners, cross grain bending failure of a bottom chord, and failure of the metal plates (causing displacement of the heel relative to the bottom chord) were all observed. Tear-through failure of the gypsum fasteners was also observed at all four corners of the diaphragm. Observation of the overall system response indicates that Configuration I was a balanced system such that further improvements to individual parts of the system likely would not lead to significant improvements of the system's performance without implementing improvements for all parts. The reinforced ceiling diaphragm only served to strengthen the entire roof system and did not have an adverse effect on the performance of the heel connections.

Table 6 compares the measured lateral capacities of the roof-to-wall connections to several typical design wind load scenarios. The wind loads were determined using Table 2.5B of the *Wood Frame Construction Manual (WFCM) for One- and Two-Family Dwellings – 2001 Edition* (AFPA 2007) for a 36-foot-wide by 40-foot-long house built in Exposure Category B, with a mean roof height of 30 feet, a 7:12 roof pitch and the trusses spanning in the short direction. Table 6 also provides an alternative summarization of specimen rotational stiffness performance by normalizing the TOH (top of heel) displacements used to calculate initial stiffness by the specimen heel height.

Table 6 – Ratios of Lateral Roof Connection Capacity Relative to Typical Design Wind Loads[1]

Configuration	Factor of Safety			TOH Displacement as % of heel height[3]
	90 mph, Exp. B (39 lb/ft)	110 mph, Exp. B (57 lb/ft)	130 mph, Exp. B (89 lb/ft)[2]	
A –Low-heel truss	6.6	4.5	2.9	1.0%
B – High-heel truss without blocking	4.5	3.1	2.0	1.2%
C – High-heel truss without blocking with low (3/12) roof pitch	4.8	3.3	2.1	1.3%
D – High-heel truss braced with OSB sheathing	5.6	3.8	2.4	0.5%
E – High-heel truss braced with OSB sheathing extended over wall plate	6.1	4.2	2.7	0.2%
F – High-heel truss with blocking at intermittent locations	5.1	3.5	2.2	0.3%
G – High-heel truss with blocking at every other bay	5.8	4.0	2.5	0.3%
H –High-heel truss with braced webs	4.7	3.2	2.0	1.0%
I – High-heel truss braced with OSB sheathing and a reinforced ceiling diaphragm	8.7	6.0	3.8	0.1%

1. Typical design wind loads calculated for a 36-foot-wide by 40-foot-long house built in Exposure Category B, with a mean roof height of 30 feet, a 7:12 roof pitch and the trusses spanning in the short direction.
2. Design wind loading at 130 mph and exposure B is equivalent to the 110 mph and Exposure C design criteria that is the upper limit used in the 2012 IRC structural provisions.
3. TOH displacement measured at same load level as initial stiffness calculations (i.e., 760 lb)

Analysis presented in Table 6 shows that all tested specimens, including the unblocked benchmark specimens exhibited significant strength capacity over design wind loads in both 90 mph and 110 mph wind zones, with factors of safety ranging from 3.1 for the unblocked high heel specimen up to 6.0 for the OSB braced specimen with a reinforced ceiling diaphragm. The results are more moderately conservative when compared to the 130 mph design wind speed, but still meet or exceed a factor of safety of 2.0 in all cases. The analysis in Table 6 shows again the increased stiffness performance of the OSB sheathed and partially blocked specimens over the benchmark low-heel specimens. It is worth noting that the disparity in stiffness performance between the various unblocked specimens decreases when the results are normalized for heel height, indicating that even an unblocked high-heel condition yields stiffness performance characteristics that are comparable to the currently code accepted low-heel condition.

SUMMARY AND CONCLUSIONS

This testing program was designed to benchmark the performance of traditional roof systems and incrementally-improved roof-to-wall systems with the goal of developing connection solutions that are optimized for performance and constructability. The results of this study are expected to provide guidance towards determining appropriate trigger levels for continuous blocking between high-heel trusses as well as viable alternative blocking solutions to those currently required by code. The following is a summary of the results of this testing program:

1) The benchmark code-allowed low heel (9¼ inch) roof system with no blocking and hurricane truss clip connections reached a peak unit capacity of 255 lb/ft.

2) The benchmark high heel (15¼ inch) roof system without blocking achieved a peak unit capacity of 175 lb/ft. This is a 33% decrease in capacity compared to the low heel configuration. The initial stiffness of the high heel specimen was approximately half that of the low-heel specimen, indicating that heel height significantly affects truss rotation where no blocking is not installed.

3) Comparison of performance results for high-heel trusses with two different roof slopes (7/12 vs. 3/12) indicates no measurable effect of roof slope on the truss rotation at the heel.

4) The high-heel roof specimen with OSB sheathing used for heel bracing (Specimen D) exhibited a 26% increase in capacity over the benchmark high-heel test (220 lb/ft versus 175 lb/ft) and only an 18% decrease in capacity compared to the benchmark low-heel test (220 lb/ft versus 245 lb/ft). The addition of the OSB sheathing also increased the specimen's initial stiffness by 18% over the low-heel specimen (10,395 lb/in versus 8,828 lb/in). This increase in stiffness, along with the reserve strength capacity over typical design wind loads, indicates that using OSB sheathing as bracing in the high-heel condition is comparable to the currently code allowed unblocked, low-heel truss condition.

5) Nailing the OSB sheathing to the supporting top plate (Specimen E) increased the capacity to 240 lb/ft. This is only 7% less than the low heel configuration (Specimen B) and 7% higher than the intermittent blocking configuration (Specimen G). The attachment to the wall top plate also significantly increased the rotational stiffness of the heel joint exceeding that for the low-heel specimen (32,224 lb/in versus 8,828 lb/in).

6) High heel systems with intermittent blocking amounts of 25% (Specimen F) and 50% (Specimen G) achieved peak unit capacities of 200 lb/ft and 225 lb/ft, respectively. Comparison of TOH displacements at both blocked and unblocked locations within Specimen F indicates that a single blocked bay provides rotational restraint to the entire specimen length (i.e., the rotational restraint is not localized). Comparison of initial stiffness between Specimen F and Specimen G indicates that the addition of a second blocking panel provides only 12% greater rotational restraint to the specimen.

7) Comparison of Specimen E (OSB sheathing also nailed to the top plate) performance to that of the 50% intermittently blocked specimen (Specimen G) shows that the Specimen E exceeded Specimen G in both peak load capacity (240 lb/ft versus 225 lb/ft) and initial stiffness (32,224 lb/in versus 26,581 lb/in).

8) The addition of diagonal truss web bracing to a high heel truss without any additional blocking provides no measurable improvement over the benchmark high heel configuration in either peak unit capacity or rotational restraint.

Several conclusions can be drawn based on the results of this testing program. It can be seen through comparison between the performances of Specimen A and Specimen D that the OSB sheathed high-heel truss detail yields comparable (and in terms of stiffness, superior) performance to that of the un-blocked, low-heel truss configuration that is currently allowed by code (see Table 1 and Figure 1). This performance, along with the reserve strength capacity over typical design wind loads, indicates that using OSB sheathing as bracing (without any additional blocking in the heel) can be considered as an adequate bracing option in high heel conditions where the intent is to provide structural performance comparable to that of an un-blocked, low-heel truss condition.

Further comparison between Specimen E and Specimen F shows that extending the OSB sheathing down and including additional nailing to the top plate of the wall below provides superior strength and stiffness performance to that of the solid, intermittent blocking that is currently required in high-wind regions, and should be considered a viable truss-heel bracing solution to said intermittent blocking.

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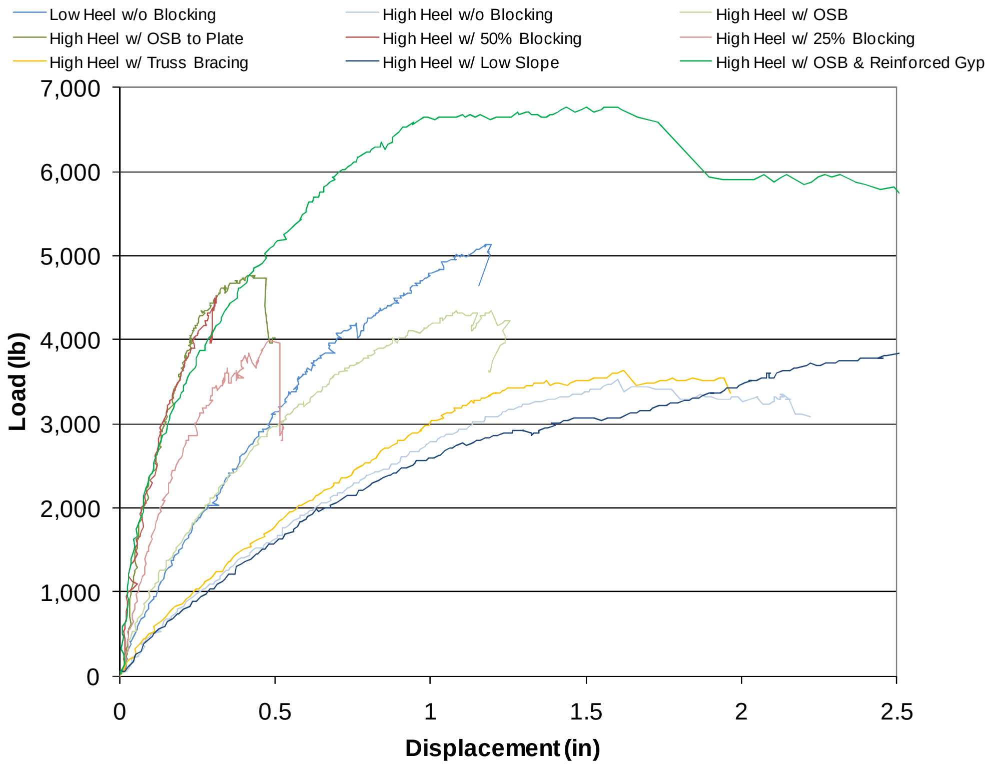
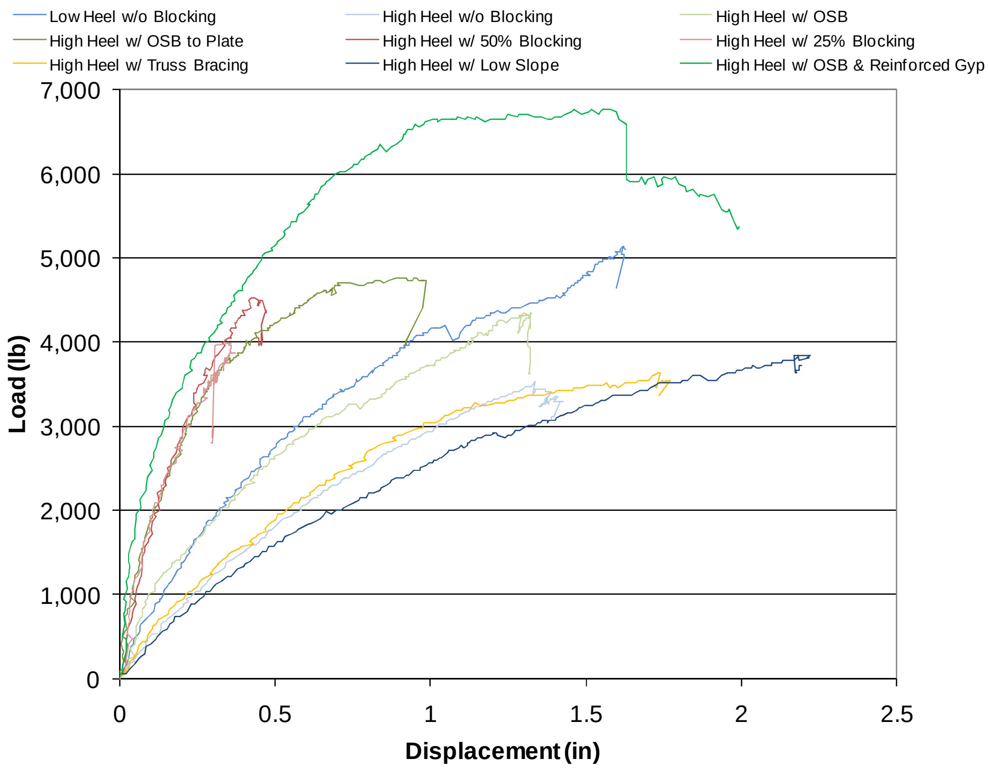
APPENDIX A

Summary Load vs. Displacement Curves



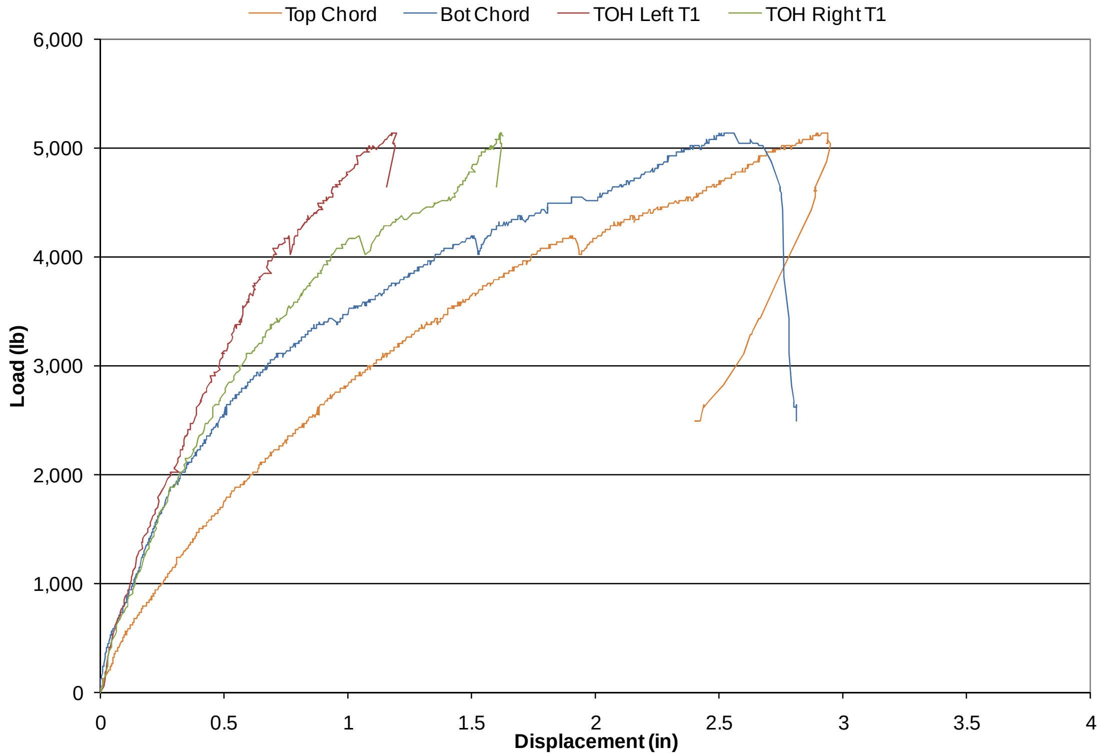
Load vs. TOH Displacement (Measured at left end of Truss 1)



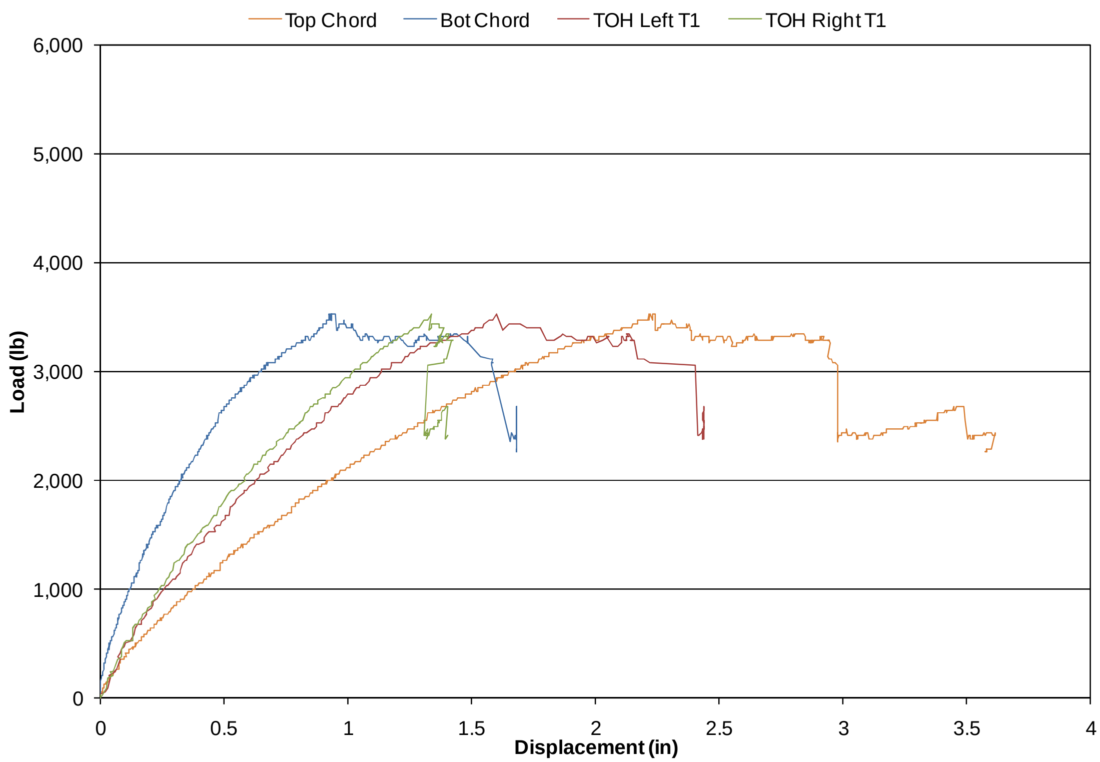
Load vs. TOH Displacement (Measured at right end of Truss 1)

APPENDIX B

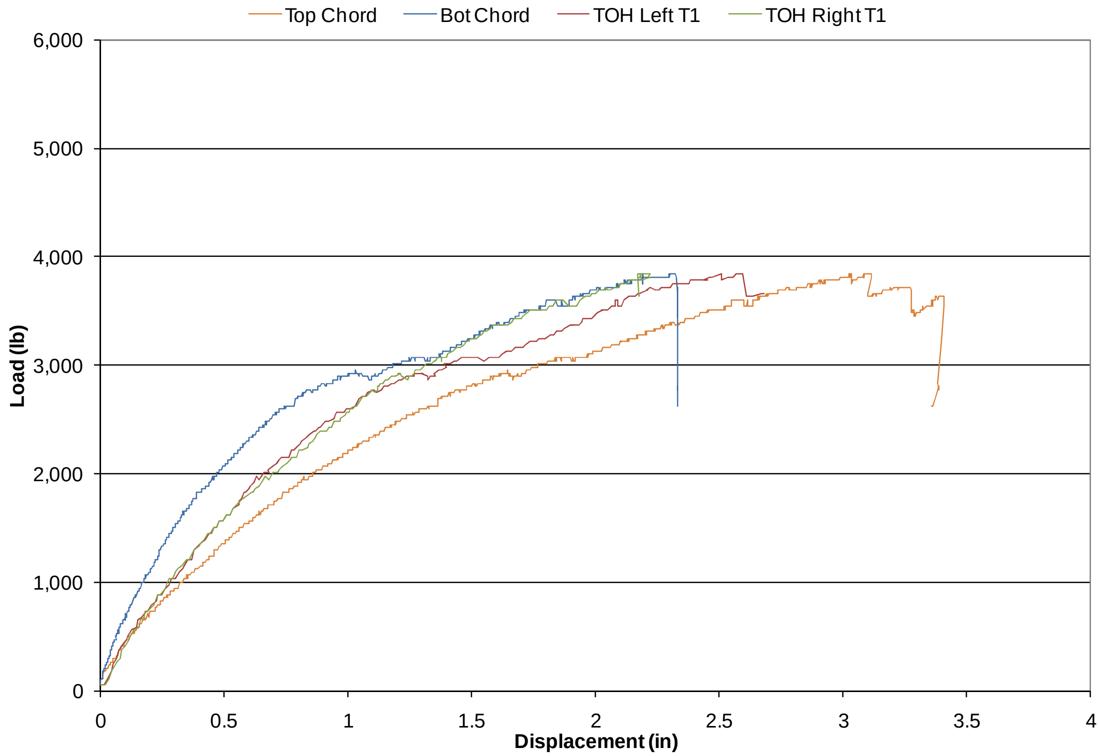
Specimen Load vs. Deflection Curves



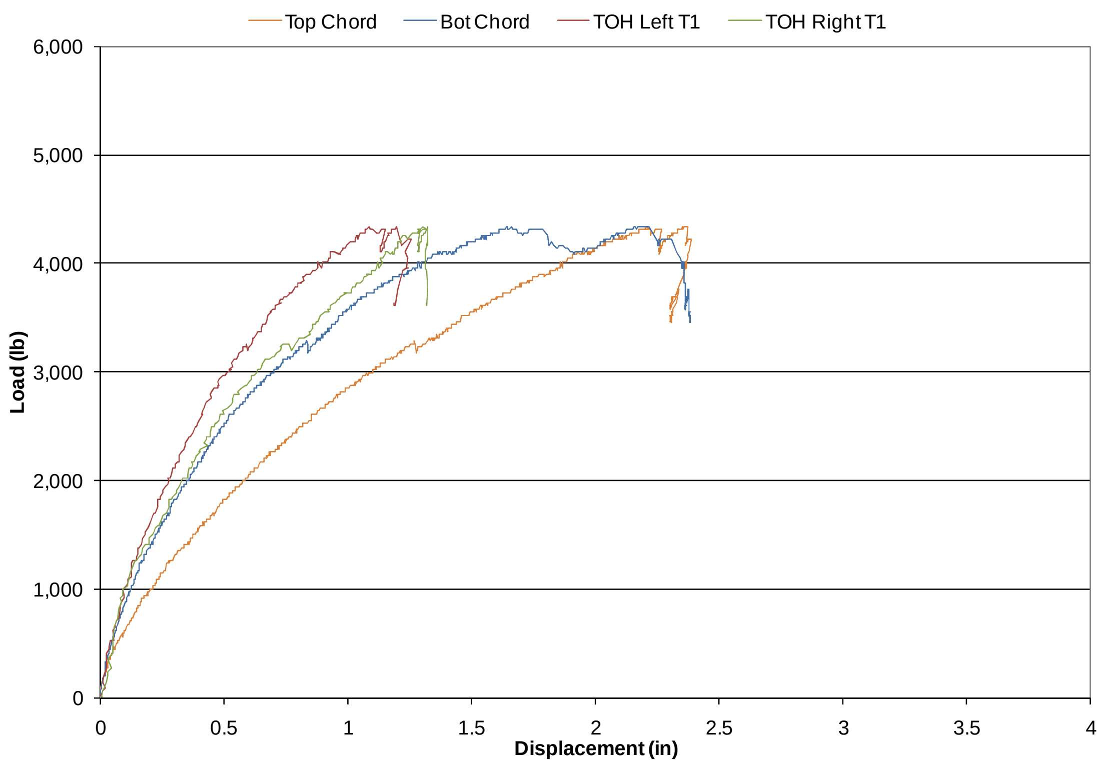
Specimen A



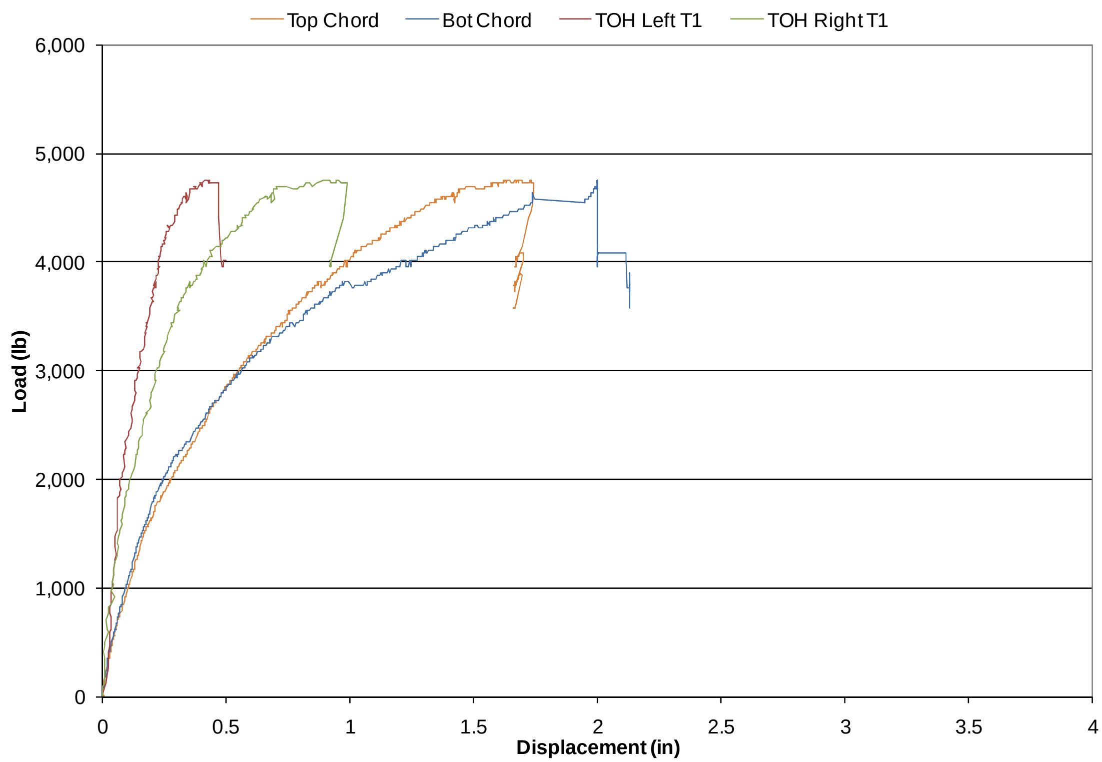
Specimen B



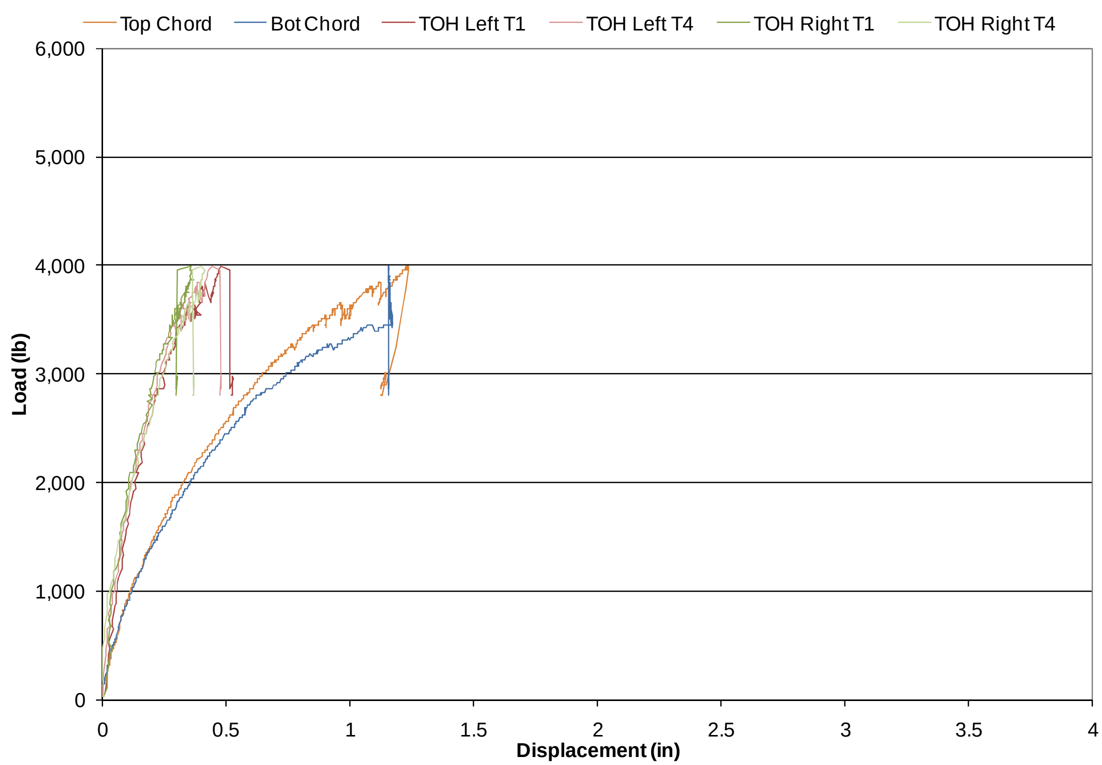
Specimen C



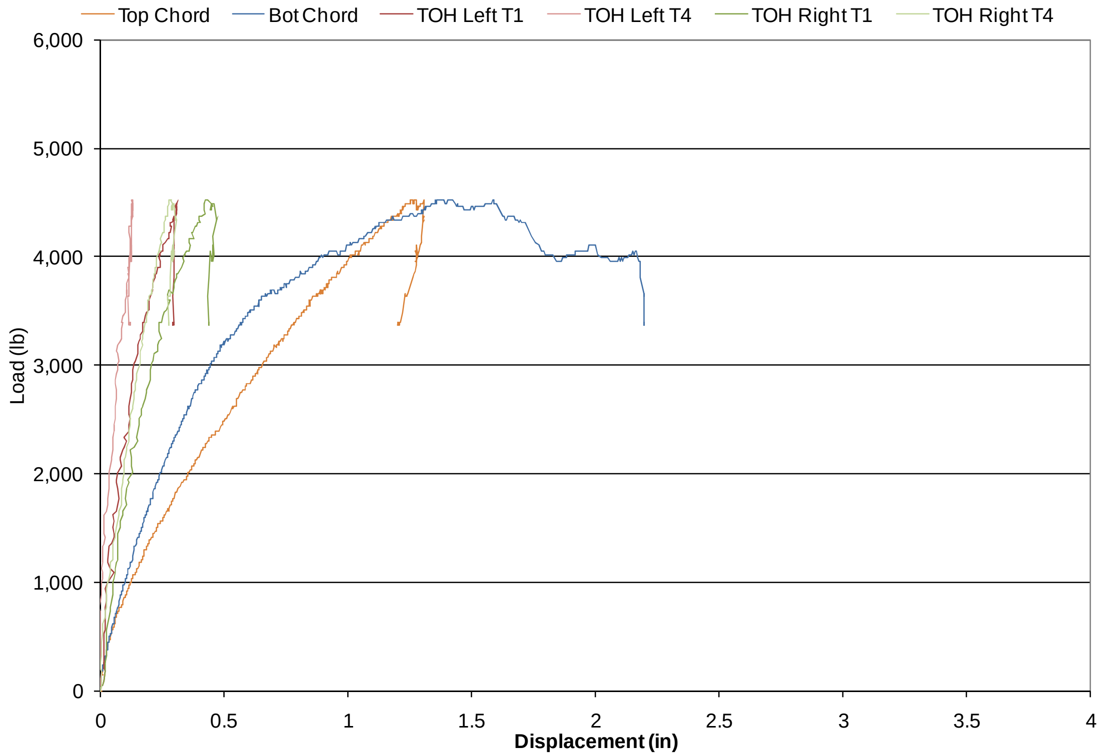
Specimen D



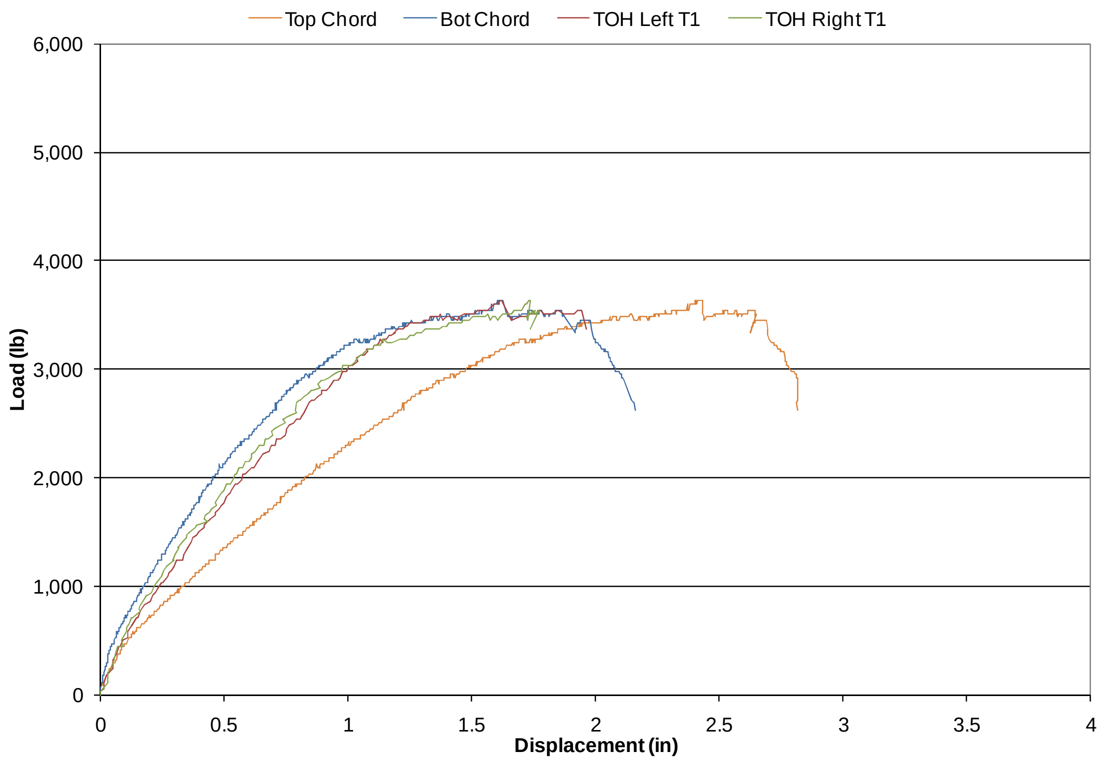
Specimen E



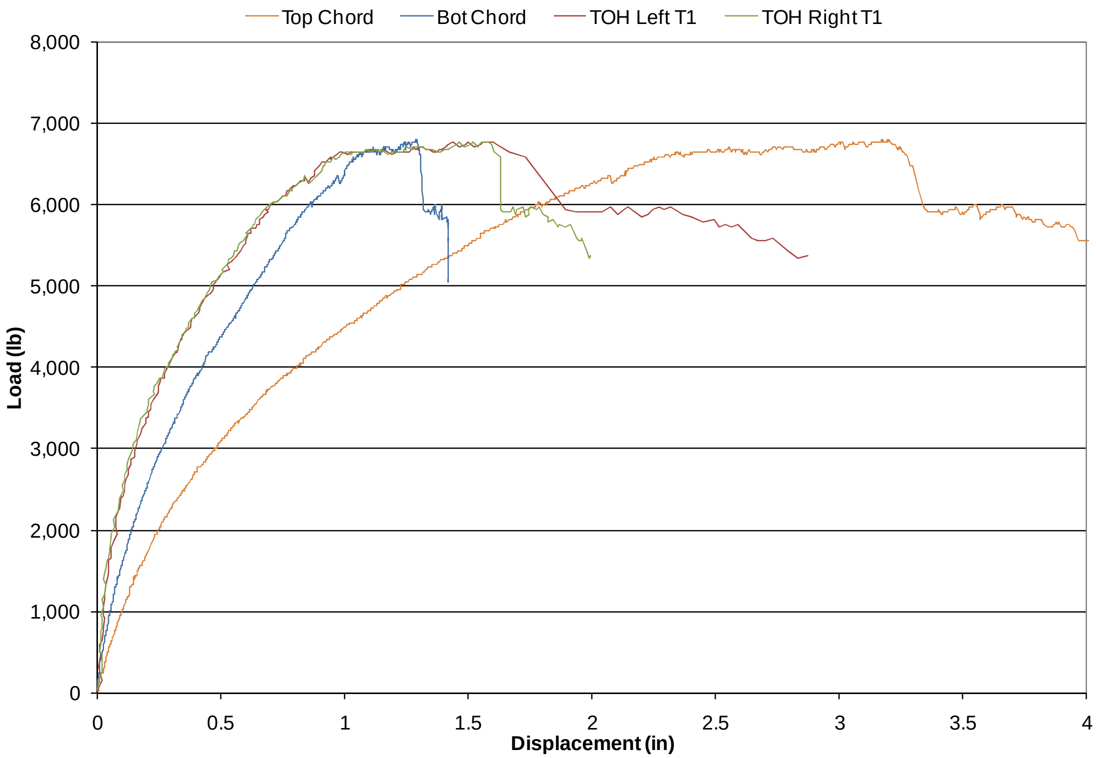
Specimen F



Specimen G



Specimen H


Top Chord
Bot Chord
TOH Left T1
TOH Right T1
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
Load (lb)
0
0.5
1
1.5
2
2.5
3
3.5
4
Displacement (in)

Specimen I