4.2 TASK 2 – FULL-SCALE ROOF-TO-WALL CONNECTION SYSTEM TESTS

4.2.1 Objective

The objective of Task 2 was to measure and compare the lateral (parallel-to-wall) performance of full-scale roof-to-wall connection systems constructed with conventional common nails, pneumatic nails, and metal connector hardware. The nailing schedules included the current building code requirements for conventional residential construction [31][32] with interpretations representative of the field framing practices. Common and pneumatic nails were investigated. Results were used to evaluate capacity-based design procedures for analysis of nailed connections. Based on the test results the scope of the minimum prescriptive provisions for roof-to-wall attachment was determined for a selected building configuration and loading condition.

4.2.2 Experimental Approach

Six full-scale roof-to-wall connection system tests were conducted. Table 14 describes the test specimen configurations and Table 15 summarizes the materials, construction, and fastening schedules. Figure 9 shows the test setup.

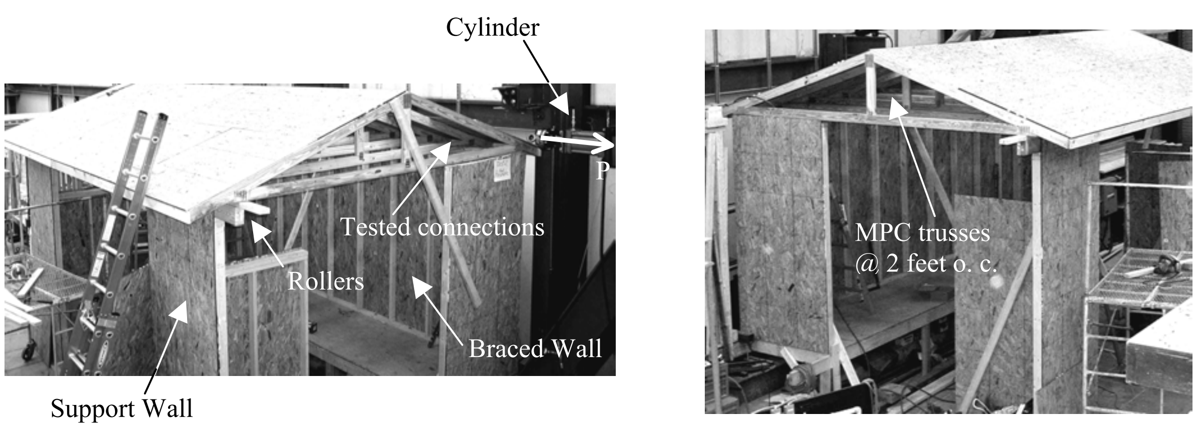
TABLE 14
TEST CONFIGURATIONS FOR ROOF-TO-WALL CONNECTION SYSTEM TESTS

Configuration	Test Number	System Connection[1]
1	1	22-16d pneumatic nails Toe-nailed (2 per truss)
	2	
2	3	33-8d common nails Toe-nailed (3 per truss)
	4	
3	5	22-12d pneumatic nails, toe-nailed (2 per truss) 9-H2.5 Hurricane Clips (at interior trusses)
4	6	4-12d pneumatic nails, toe-nailed (2 per end truss) 9-H2.5 Hurricane Clips (at interior trusses)

[1]For actual nail sizes, refer to Section 4.1.

TABLE 15
MATERIALS, CONSTRUCTION, AND FASTENING SCHEDULES FOR ROOF SYSTEMS

COMPONENT	MATERIALS, CONSTRUCTION, AND FASTENING SCHEDULE
Roof Truss	12-foot-span metal plate connected wood truss, 4/12 pitch, constructed with 2 inch x 4 inch nominal size Southern Yellow Pine lumber (SYP), installed 2 feet on center, attachment to top plate – see Table 14
Roof Sheathing	7/16-inch-thick 4 foot by 8 foot OSB panels, 8d pneumatic nails (D=0.131inch) spaced 6 inches on-center at panel edges and 12 inches on center in field, panels installed with the long dimension perpendicular to the trusses
Tests 1, 3, & 5 Roof Sheathing/Edge Row	Same, except: nails are replaced with 1-5/8-inch-long all-purpose screws only on opposite side of tested side
Fascia Board	1 inch x 6 inch nominal size, # 2 Common Pine, attached to each truss with two 8d pneumatic nails (D=0.131inch)
Truss Support	SPF double top plate as a part of braced wall assembly on one side and steel roller plates on top of support wall on the opposite side
Loading Strap	8-inch-wide 17-feet-long 14-gage steel strap attached to roof sheathing panels with a total of 32 screws spaced evenly along the length of the strap in three rows



Front View **Back View**

Figure 9
Roof-to-Wall Connection System Test Setup

Each test specimen included a 12-foot-wide by 20-foot-long roof assembly framed with eleven prefabricated 12-foot-span metal plate connected (MPC) wood trusses spaced at 2 feet on center and sheathed with 7/16-inch-thick OSB panels. One side of the roof assembly was supported by a 20-foot-long braced wall anchored to a support steel platform. Four-foot-long corners were built on each end of the wall. A hold-down device was installed at the uplifting wall corner for tests 5 and 6. The opposite side of the roof assembly rested on a reaction wall anchored to the concrete floor. The walls were framed with 2-inch by 4-inch nominal size SPF lumber and sheathed with 7/16-inch-thick OSB. The braced wall was designed to have capacity greater than the tested connections and it was reused throughout all six tests. The bottom member of the shear wall double top plate was nailed to the studs and sheathing, whereas the top member of the double top plate was fastened to the bottom member with screws to facilitate replacement of the top member after each test.

The trusses were attached to the top plate of the braced wall according to nailing schedules specified in Table 14. Steel roller plates were placed between the roof assembly and the support wall on the opposite side to allow horizontal movement of the roof with minimum friction. A 1-inch by 6-inch nominal size fascia board was nailed to the plumb-cut ends of the truss top chords on both sides of the roof assembly to provide a rotation restraint for individual trusses.

A total of three roof systems were built. Each roof system was tested twice. After the first test was completed, the roof assembly was lifted and rotated to run another test on the opposite side. The edge layer of OSB, that was attached with screws, was removed temporarily and the trusses were connected to the new top plate. The OSB panels were reattached using a standard roof sheathing nailing schedule.

Load was applied to the roof assembly through a 14-gage steel strap which was attached to the sheathing roof panels with screws (Figure 10). The screws were installed in the intervals between the trusses so that there were no additional fasteners connecting roof sheathing and top chords of the trusses. The use of the flexible steel strap minimized the effects of the boundary conditions imposed on the roof system by the test apparatus. The strap was attached to a hydraulic actuator using a clevis. The hydraulic actuator was mounted on a steel reaction frame using a pinned connection so that the moment forces were not transferred from the specimen into the cylinder and from the cylinder into the reaction frame. Tension load was applied to the strap at a constant displacement rate of 0.3 inch/min and the test was run until the load decreased by a minimum of 30 percent from the ultimate value. Load was measured with a 100,000 lb rated capacity electronic load cell positioned between the strap and the hydraulic actuator.

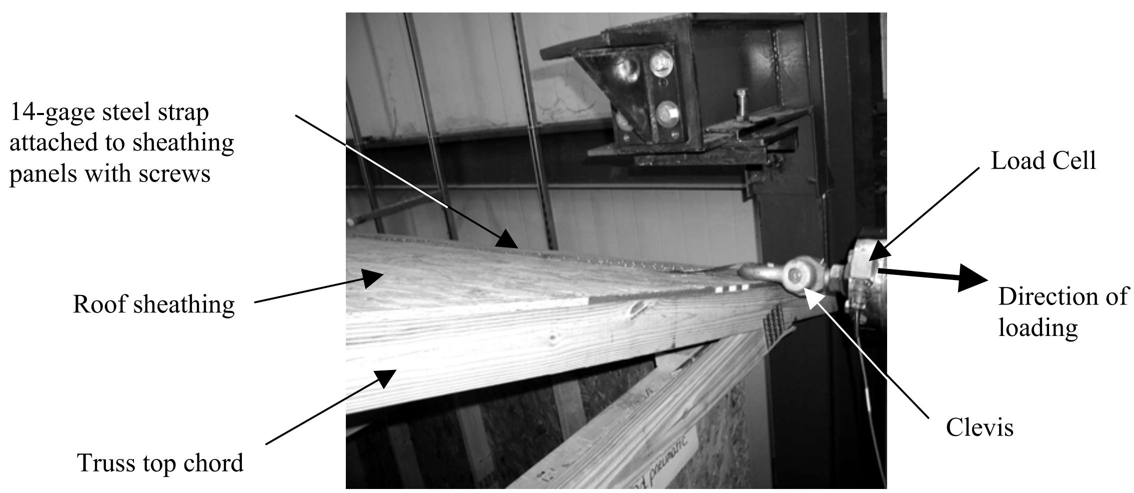


Figure 10
Loading Steel Strap Attachment

Two linear variable displacement transformers (LVDT) were positioned on the opposite end of the test specimen to measure the deformation of the roof diaphragm relative to the braced wall (Figure 11). One LVDT was setup to measure displacement of the roof sheathing, and another was setup to measure the displacement of the top plate of the shear wall. The difference between these two readings was the total deformation of the roof relative to the wall top plate including roof assembly translation, truss rotation, and sheathing panel slip. A computer-based data acquisition system was used to record the load and displacement measurements at a sampling rate of 1 Hz.

Figure 11
LVDT Setup (displaced position)

4.2.3 Results and Discussion

Table 16 summarizes the results of the full-scale roof-to-wall connection system tests. The average peak load for the systems assembled with two 16d pneumatic nails per joint (3,115 lb) was marginally higher than that for the systems assembled with three 8d common nails per joint (3,030 lb). The toe-nailed roof-to-wall connections (configurations 1 and 2) provided an average unit resistance of 280 lb per joint. However, due to high scatter of peak loads between two repetitions of test configuration 1 (2,387 lb vs. 3,843 lb), it can not be decisively concluded that two 16d pneumatic and three 8d common nails are equivalent with respect to the connection capacity. It is believed that this variability was the result of workmanship and framing practices used by the laboratory technician to assemble the test specimens. The laboratory technician was a framer with extensive construction experience and he used his knowledge and judgement in applying framing practices. Therefore, the performance of test specimens 1 and 2 is considered as representative of “as-built” conventional construction and is characteristic of the lower and upper bound of the performance of toe-nailed roof-to-wall connections. This serves as an evidence to sensitivity of the response of toe-nailed connections to workmanship and framing practices.

TABLE 16
SUMMARY OF TEST RESULTS FOR ROOF-TO-WALL CONNECTION SYSTEM TESTS

CONFIGURATION	TEST SPECIMEN NUMBER	SYSTEM CONNECTION	PEAK LOAD, LB	DISPL. @ PEAK LOAD, INCH	AVERAGE PEAK LOAD, LB	UNIT LOAD, LB/JOINT
1	1	22-16d pneumatic nails Toe-nailed (2 per truss)	2,387	0.58	3,115	283
	2		3,843	n/a[1]		
2	3	33-8d common nails Toe-nailed (3 per truss)	2,954	n/a[1]	3,030	276
	4		3,107	0.61		
3	5	22-12d pneumatic nails, toe-nailed (2 per truss) 9-H2.5 Hurricane Clips (at interior trusses)	5,995	1.09	5,995	545
4	6	4-12d pneumatic nails, toe-nailed (2 per end truss) 9-H2.5 Hurricane Clips (at interior trusses)	6,427	1.10	6,427	584

[1]LVDT malfunctioned during the test.

Although designed primarily to resist roof uplift forces, the hurricane clips increased the peak lateral resistance of the roof-to-wall connections by approximately a factor of two. The unit resistance of specimens that included hurricane clips (configurations 3 and 4) was between 545 lb/joint and 584 lb/joint compared to approximately 280 lb/joint for toe-nailed-only specimens (configurations 1 and 2). Therefore, the hurricane clips can be successfully used to enhance the lateral resistance of conventional roof-to-wall connections. The system with 22 toe-nails and 9 hurricane clips (configuration 3) exhibited lower peak load than the system with 4 toe-nails on end trusses only and 9 hurricane clips (configuration 4). This observation indicates that toe-nails are incompatible with engineered hardware and the addition of toe-nails does not improve the lateral resistance of connections assembled with hurricane clips. The displacement at peak load of 0.6 inches observed for toe-nailed-only connections versus 1.1 inches for connection with hurricane clips further supports the evidence than the two connection types have different stiffness characteristics and achieve capacities at different deformations. Therefore, resistance of toe-nails can not be superimposed with the resistance of hurricane clips.

Figures 12 through 15 exemplify the response and failure modes observed in test specimens 1 and 2. The trusses slid along the top plate of the braced wall with little out-off plate rotation (Figure 12). The failure mode of toe-nailed connections was direction dependent and included wood splitting and tearing out on the tension side of the connection (Figure 13) and nail bending on the compression side of the connection. In one joint, the truss plate withdrawal resistance was exceeded and the top chord of the truss separated from the bottom chord (Figure 14). However, the truss plate failure of only one joint in two system tests (22 joints in total) indicates that toe-nails are the predominant weakest link in this type of connection under lateral loading.

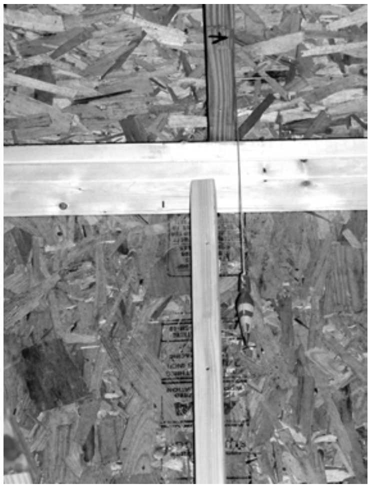


Figure 12
Horizontal Movement of Truss
(in initial position truss was aligned with stud)

Figure 13
Wood Tear Out and Plate Bending
on Tension Side of Connection

Figure 14
Truss Plate Separation

Figure 15
No Visual Damage on Compression
Side of Connection

Figures 16 and 17 show the failure modes for test specimens 3 and 4. In addition to the failure modes associated with specimens 1 and 2, the withdrawal of shorter 8d common nails from the wall top plate was observed. The nail withdrawal also caused uplift deformations of trusses from the wall top plate (Figure 16).

Figure 16
Truss Separation from Top Plate
due to Nail Withdrawal

Figure 17
Wood Tear Out and Plate Bending
on Tension Side of Connection

The location of the truss plates in the heel joint assembly directly above the supporting wall limits the available surface for installation of nails and other connectors. In this test program, the nails were installed into the bottom chord member in the region between the truss plate and exterior surface of the wall (Figure 16). The nail location near the beveled end of the truss bottom chord precipitated the premature wood splitting and tear-out failure. The installation of toe-nails through the metal truss plates, as sometimes done in the field, is likely to defer or suppress the premature splitting and improve the overall connection performance. Therefore, these tests can be considered as representative of the "lower bound" performance of conventional roof assemblies using MPC wood trusses.

Figures 18 through 22 exemplify the failure modes observed in test specimens 5 and 6. The hurricane clips changed the response and failure modes of the connections. Truss plate separation was more frequently observed (Figures 18 and 22) and trusses rotated out-of-plane (Figure 19). The degradation of hurricane clips was caused by excessive deformation of the body of the clip due to localized buckling of light-gage steel (Figure 20). One hurricane clip failed in tension along the cross section with two nail perforations (Figure 21).

Figure 18
Truss Plate Separation

Figure 19
Truss Slip and Rotation

Figure 20
Hurricane Clip Buckling

Figure 21
Hurricane Clip Tension Failure

Figure 22
Truss Plate Separation

Table 17 compares the experimental data with the analytical predictions of the yield theory at the NDS design and capacity limit states. The lateral design resistance of 130 lb for a single H2.5 hurricane clip is adopted from the manufacturer's specification [34]. Because the ultimate lateral resistance of hurricane clips is not reported by the manufacturer, the comparison between the tested and predicted values at capacity limit state was not performed. The resistance of connections with hurricane clips (configurations 3 and 4) is calculated for three scenarios based on contribution of hurricane clips only (HC), toe-nails only (TN), and both hurricane clips and toe-nails (HC+TN). Although the NDS [1] does not permit superimposing the resistances of different connectors, the HC+TN values are calculated to explore the correlation with the experimental data and are given in parentheses to indicate the research purpose of the estimates.

TABLE 17
COMPARISON OF ANALYTICAL AND EXPERIMENTAL RESULTS FOR ROOF-TO-WALL CONNECTION SYSTEM TESTS

CONFIGURATION	SYSTEM CONNECTION	AVERAGE PEAK LOAD, LB	CALCULATED LATERAL DESIGN VALUE[1], LB	PEAK LOAD/ CALCULATED (SAFETY MARGIN)	CALCULATED ULTIMATE VALUE[1], LB	PEAK LOAD/ PREDICTED RATIO
1	22-16d pneumatic nails Toe-nailed (2 per truss)	3,115	2,470	1.26	4,871	0.64
2	33-8d common nails Toe-nailed (3 per truss)	3030	3,051	0.99	5,850	0.52
3	22-12d pneumatic nails, toe-nailed (2 per truss) 9-H2.5 Hurricane Clips (at interior trusses)	5,995	1,170 – HC[2] 2,124 – TN[3] (3,294 – HC+TN)[4]	5.1 – HC[2] 2.8 – TN[3] (1.8 – HC+TN)[4]	n/a[5]	n/a[5]
4	4-12d pneumatic nails, toe-nailed (2 per end truss) 9-H2.5 Hurricane Clips (at interior trusses)	6,427	1,170 – HC[2] 386 – TN[3] (1,556 – HC+TN)[4]	5.5 – HC[2] -- (4.1 – HC+TN)[4]	n/a[5]	n/a[5]

[1] See Appendix A for calculations.
[2] Based on resistance of hurricane clips.
[3] Based on resistance of toe-nails.
[4] Based on superposition of toe-nails and hurricane clips. The values are given is parenthesis because the NDS does not permit superposition for mixed fastener connections [1].
[5] Ultimate lateral resistance of hurricane clip is not specified by the manufacturer [34].

The average safety margin of 1.1 for toe-nailed connections (configurations 1 and 2) manifests the deficiencies of the design methodologies for analysis of this type of connection. Similarly, the yield theory predictions of the ultimate toe-nail connection strength overestimate the test peak load by as much as a factor of 1.9 (configuration 2). The disparity between the analytical values and tested resistance of toe-nailed systems is partially attributed to the constructability of toe-nailed connections in general and framing practices used in this testing program. Yet, the differences in the lateral response between toe-nailed and face-nailed connections should be better understood to identify the limitations of the yield theory application to toe-nailed connections and to reevaluate the current design provisions for lateral analysis of toe-nailed connections. A testing program of individual roof-to-wall connections was conducted to quantify the lateral performance of toe-nailed connections. Results of the testing and analytical findings are summarized in the next section (Section 4.3).

The calculated lateral design values for test configuration 3 (Table 17) expose the inconsistencies in using the joint slip limit state for establishing characteristic connection properties. According to the current design provisions, the lateral design resistance of the toe-nailed connections is greater than that of hurricane clips for roof configuration 3 by as much as a factor of 1.8. Given this design value, the engineer is more likely to specify toe-nailed connections for the roof-to-wall lateral load path. The lack of information available to the engineer on the correlation of the design properties and the connection capacities creates a perception that the toe-nailed connections provide a better degree of safety relative to failure. However, results of these tests demonstrate a contrary trend with the hurricane clips providing as much as twice of the toe-nail lateral resistance.

The safety margin of 5.1-5.5 for the hurricane clip connections is excessive. The allowable design value for the hurricane clip adopted from the manufacturer's specifications are established based on a joint slip limit state. This direct implementation of design methods developed for single dowel connections to light-gage steel hardware connections, which exhibit different response and unique failure modes, results in ambiguous design values and an arbitrary design basis with respect to the performance levels of the hardware systems. Based on this limited testing, the allowable lateral resistance of hurricane clips in the direction parallel to wall can be increased from 130 lb to 260 lb per clip.

4.2.4 Design Applications

This section explores the design application of test results from Task 2. A simplified seismic analysis is performed to design a roof diaphragm-to-shear wall connection using the tested joint configurations. For a selected roof configuration, seismic design categories are assigned to the conventional toe-nailed and engineered connections.

Design input parameters:

Truss span	36 feet
Truss spacing	24 inches
Dead load	15 psf
Load combination	0.7E
Response modification factor (assumed)	R = 5
Overstrength factor (assumed)	$\Omega = 3$
Vertical load distribution factor for simplified design procedure	$\psi = 1.2$

Unit seismic weight per joint: (15)(36/2)(24/12) = 540 lb

Allowable resistance values, F, for individual roof-to-wall connections are determined from test results (Table 16). A safety factor of 2.0 relative to the joint peak load (capacity) and standard use conditions (i.e., adjustment factors equal unity) are used.

Configuration 1:	2-16d Pneumatic Nails	F = 283 / 2 = 141 lb/joint
Configuration 2:	3-8d Common Nails	F = 276 / 2 = 138 lb/joint
Configuration 4:	H2.5 Hurricane Clip	F = 584 / 2 = 292 lb/joint

Maximum 0.2 sec design spectral response acceleration, $S_{DS,}$ is calculated as follows:

Configurations 1 and 2: $S_{DS} = (140)(5)/[(0.7)(1.2)(540)(3)] = 0.51g$
Configuration 4: $S_{DS} = (292)(5)/[(0.7)(1.2)(540)(3)] = 1.1$ g

Based on these findings, the conventional toe-nailed connection schedule is generally sufficient to provide the shear load transfer for seismic design categories A, B, and C with $S_{DS}<0.5$ (refer to Table R301.2.2.1.1 [32] for classification of seismic design categories). In the areas of moderate to high seismicity (i.e., West Coast, New Madrid, and Charleston areas) with assigned seismic design categories D_1 ($S_{DS}<0.83g$) or D_2 ($S_{DS}<1.17$), shear transfer can be provided with hurricane clips. For seismic design category E ($S_{DS}>1.17g$), which includes the near-fault regions, additional measures such as blocking and increased fastening schedule should be implemented.

These recommendations are valid for the specified building configuration. The fastening requirements can be relaxed for lighter roofs and smaller roof spans, or become more stringent for heavier roofs and longer spans. Moreover, the default soil type for this classification of seismic design categories is based on site class D. The connection requirements can be further adjusted for other site class categories.

This example is intended to provide prescriptive design recommendations applicable to a variety of building configurations. Therefore, design assumptions (i.e., R-factor, Ω-parameter, safety factor) are selected to provide conservative fastening schedules for the majority of houses. If the roof-to-wall connections are analyzed as a part of a specific lateral force resisting system (LFRS), as may be done with engineered houses, R-factor and Ω-parameter are used with the resistance of shear walls and diaphragms to determine the maximum potential force demand that can be applied to the connections. Using capacity-based system and component design values, this approach allows for better balancing of the connection capacity relative to other components of the LFRS. In addition, light-frame wood houses generally exhibit a response characteristic of "soft-story" behavior with the weakest link in the first-story shear walls so that the demand on the roof-to-wall connections is typically limited to elastic response. Therefore, the design recommendations provided in this example can be further adjusted and are likely to become less stringent.

4.2.5 Conclusions

1. Conventional toe-nailed roof-to-wall connections assembled with 3-8d common or 2-16d pneumatic nails per truss provided about 280 lb/joint of capacity for shear loads parallel to the wall in full-scale system tests (Table 16).

2. The primary failure modes for toe-nailed connections included splitting and tear-out of wood, nail bending, and nail withdrawal (Figures 12-17). The wood splitting and tear-out were caused by reduced end distance between the nails and beveled end of the bottom truss chord. The primary failure modes for joints with hurricane clips included buckling of the body of the clip, separation of metal truss plate, and truss rotation (Figures 18-22).

3. An average safety margin of 1.1 for predicted performance of toe-nailed connections (Table 17) indicate deficiencies in the design methodologies. This effect is partially attributed to the

connection failure modes (i.e., wood splitting and tear-out) that preceded more ductile failure modes associated with the yield theory.

4. Use of light-gage steel hurricane clips doubled the shear transfer capacity of the system to about 560 lb/joint (Table 16) without use of blocking between the trusses.

5. The resistances of toe-nails and hurricane clips can not be superimposed due to different stiffness characteristics of two connection types (Table 16).

6. Because metal truss plates limit the area available for installation of toe-nails (Figure 16) and the beveled end of ceiling joist is susceptible to premature splitting (Figure 17), the toe-nailed truss-to-wall connection is not necessarily equivalent to conventional roof-to-wall connections that use roof systems assembled with rafters and joists rather than trusses. Therefore, further research is needed to develop prescriptive connection requirements for MPC trusses consistent with the use of three 8d common toe-nails with conventional roof systems.

7. Using capacity as the design basis, the lateral allowable resistance of hurricane clip H2.5 in the direction parallel to wall can be doubled relative to the values provided by the clip manufacturer.

8. In moderate- to high-hazard areas of the United States, use of simple roof ties without additional blocking or detailing can significantly improve the shear transfer through roof diaphragm systems into shear walls in conventional residential construction and engineered wood-frame construction.

4.3 TASK 3 – INDIVIDUAL ROOF-TO-WALL TOE-NAILED CONNECTION TESTS

4.3.1 Objective

The objectives of Task 3 were to measure the performance of individual toe-nailed roof-to-wall connections and to evaluate the engineering design methodologies for analysis of toe-nailed connections. Common and pneumatic nails were investigated. The differences in the lateral response between toe-nailed and face-nailed connections and the limitations of the yield theory application to toe-nailed connections were identified. Moreover, potential system effects were investigated through comparison of the results of full-scale (Task 2, Section 4.2) and individual connection tests.

4.3.2 Experimental Approach

A series of tests on individual roof-to-wall connections with the nailing schedules adopted from the full-scale testing (Section 4.2) was conducted. Two connections (Table 18) corresponding to specimen configurations 1 and 2 of the full-scale tests (Table 14) were investigated. Figure 23 shows the test setup.