Probabilistic Descriptions of in-situ Roof to top plate connections in light frame wood structures

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ABSTRACT:

Failure of roof to top plate connections is a major cause of damage in light frame wooden structures subjected to high wind loads. This damage history highlights the need for understanding their resistance and behavior under such extreme loads. Earlier research replicated roof to top plate connections in the laboratory to study their ultimate capacity, but struggled to realize the actual capacity of the connections in their "as built" in-situ condition. Furthermore, little effort has been made in the past to identify the probability distributions that describe the response and the ultimate capacity of these connections. However, this type of information is essential for conducting reliability based studies of existing residential dwellings. In this study, the uplift capacities of roof-to-wall in-situ toe nail connections, in their as-built condition are determined and their associated probability distributions are presented.

Four residences of similar construction (wood-frame walls and stick framed roofs) built 30-40 years ago and slated for demolition, were tested and the uplift performance of the roof-to-wall connections were assessed in accordance with ASTM D1761-2000. Over 80 in-situ connections were tested, allowing for the uplift capacity of roof to top plate connections in these residential structures to be positively established. Results show that connections using 2-16d toenails have a mean ultimate capacity of 1.52 kN (342 lbs) and a coefficient of variation (COV) of 0.35. This COV value is found to be approximately twice that found from previous laboratory based studies lending motivation for conducting in-situ tests. Furthermore, cyclic testing of these connections also provided load-displacement relationships from which stiffness values were approximated. Results show that capacity and stiffness can plausibly be modeled as jointly lognormal.

Keywords:

Roof-to-wall connection, uplift capacity, toenail connection, in-situ connection, probability distribution.

INTRODUCTION

In spite of the new trends in construction methods like SIP (Structural Insulated Panels), Insulated wood forms and steel forms, light frame wood construction appears to still be the most popular and well established construction practice among residential structures in the United States. However, light frame wood structures are one of the most susceptible structure types to wind damage. Hence there is a need for better understanding the performance of these structures under high wind loads.

The suction and pressure exerted on a roof system due to wind must be transferred safely to the foundation through a vertical load path. Often this is accomplished by a proper tie down of roof sheathings to the roof framing members and the rafters to the wall top plates. Strong bottom plate to foundation connections complete the vertical load path ensuring complete transfer of uplift forces to the foundation. This vertical load path can be better visualized as a series of links with the weakest link being the critical link in deciding the reliability or performance of the entire system. Roof to wall connections are a key link in this load path and various research studies have been performed to evaluate the uplift capacities of these crucial connections.

In a study by Cheng [2002], the adequacy of the uplift capacity of the rafter to top plate toe nail connections located in high wind regions was examined. One recommendation from this study is that building codes should be changed to eliminate the use of toenail connections in these regions. Reed et al. [1997] in their comprehensive study evaluated the capacity of toenail connections and compared them against hurricane straps and adhesives. They also examined the uplift capacity of a repetitive system of fasteners with varying connection methods. Other studies looked further at the behavior of these connections under various loads and various retrofit conditions [Judge et al. 2002; Riley and Sadek 2003]. Though these studies have contributed significantly to understanding the behavior of the rafter to top plate connections, they have failed to completely capture the insitu uplift capacity and response of the connections. Laboratory mock-up tests fail to capture the variability that is inherent in as-built construction. Furthermore, only a few attempts have been made in the past to study the stiffness, in addition to the capacity, of such connections and to identify appropriate probabilistic models of these characteristics.

The present study is an encouraging contribution to the risk and reliability assessment of light frame wood structures. The objective of this study is to evaluate the insitu as-built uplift capacity and relative stiffness of roof-to-wall toenail connections and to identify the probability distributions that describe them. The results from this study may be used in assessing the component level reliability of these connections. Furthermore, the results from this study may be used to develop analytical model of the connection behaviors. Analytical models will permit more detailed analyses and assessments of existing structures. On the whole, the present study offers significant statistical data on uplift capacity and stiffness of toe nail connections which in turn supports the development of a safe and reliable residential infrastructure.

TEST SITE

The insitu tests for the present study were performed on structures in Douthit Hills, a family housing community located on the campus of Clemson University. This community contains 44 residential units spread over 55 acres of land. A move for demolition of these housing units provided an opportunity to destructively evaluate the structural performance and adequacy of the buildings. Furthermore, repetitive nature of the housing unit styles provided the rare opportunity to acquire a statistically significant amount of data.

Four duplex houses, identical in layout and construction were selected for testing. A photo of one such unit is given in Figure 1. Their overall plan view dimensions are 8.23m (27ft) wide by 20.73m (68ft) long. The building layout, also showing the roof framing plan, is presented in Figure 2. These houses are believed to be representative of typical wood frame residential structures constructed 40-50 years ago. Similar types of construction can be found on military bases and other types of government projects.

FIGURE 1 DOUTHIT HILLS DUPLEX RESIDENTIAL STRUCTURE

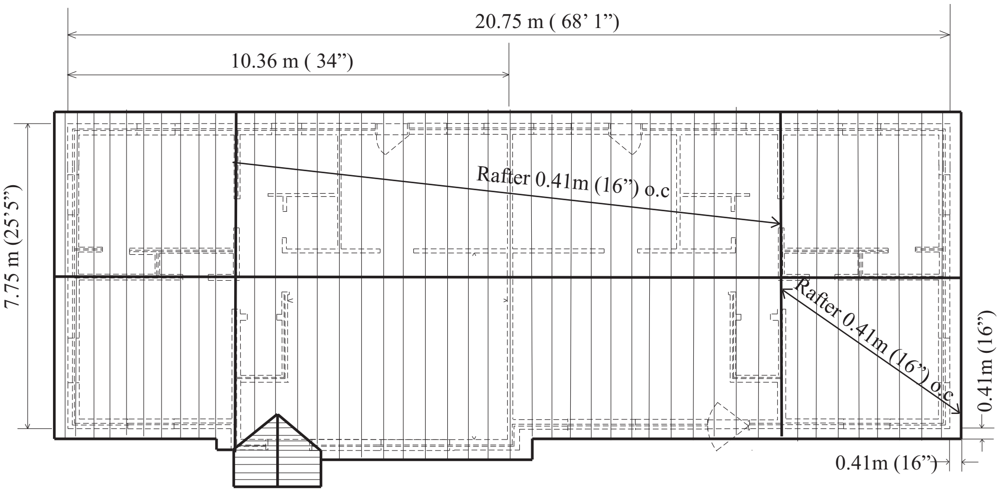


FIGURE 2 TYPICAL ROOF FRAMING PLAN OF THE STRUCTURES

The entire structure rests on spread continuous concrete footings approximately 20 in wide x 10" thick. The roof is stick built using dimensional lumber and is made up of 38 x 140 mm (nominal 2x6 inch) or 38 x 89 mm (nominal 2x4 inch) horizontal ceiling joists and 38 x 140 mm (nominal 2x 6) rafters. The rafters, having a 6:12 pitch, meet at a ridge board at the peak and are tied to the ceiling joists at the bottom of the roof. Collar ties are used on every fourth rafter line. The lower end of the rafter is attached to the side of the ceiling joist by three 3.3 mm (0.131 inch) diameter, 63.5 mm (2.5 in) long smooth shank 8-d common nails. The connection of the ceiling joist to the top plate, as shown in Figure 3, is made by two or three 4.1mm (0.161 in) diameter, 89mm (3 ½ in) long smooth shank 16-d common nails. Typically, one nail is toenailed on each side of the joist. Plank sheathing, 19 mm thick by 140 mm wide (nominal 1 x 6 inch), is covered with asphalt shingles and the building exterior is covered with a brick veneer and vinyl siding. Visual inspection of the framing members shows the wood to be southern yellow pine (SYP).

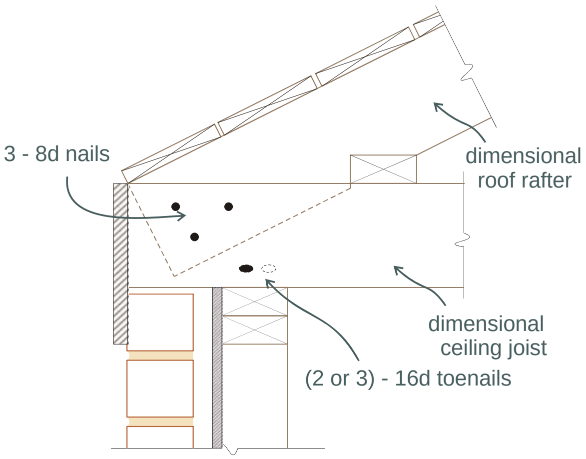


FIGURE 3 ROOF TO WALL CONNECTION DETAIL

TEST SETUP

Various test setups have been utilized in previous studies. Some tests have used hydraulic jacks and reaction frame to apply the requisite uplift load (Reed et al. 1997) while others used spreader beams, load trees and cranes to achieve the uplift (Judge et al. 2002). Some setups loaded single connections (Cheng 2002, Reed et al. 1997). Others wanted to capture system effects by loading up multiple connections simultaneously (Judge et al. 2002, Riley and Sadek 2003).

The test set up chosen for this research was designed to accommodate the simultaneous loading of multiple connections while maintaining control over the load rate and pattern. Two automated screw jacks mounted on a reaction frame carry a spreader beam which applies equal deflection on a system of four connections as shown in Figure 4. Four 22 kN (5 kip) load cells are attached to the top flange of the spreader beam and provide the loading to the bottom surface of the ceiling joist. LVDTs are mounted to the top plate of the wall and are used to measure the relative distance between the plate and the ceiling joist.

The modular reaction frame is designed to withstand the dead load of the jacks, motors, spreader beam, support beam and the reaction from the load cells and safely pass it on to the floor underneath. It is also designed for stability against accidental eccentric loads and is versatile to the changes in the height of the building. The cost effectiveness and the portability of the reaction frame add to its advantage.

FIGURE 4 EXPERIMENTAL SET UP FOR UPLIFT TESTS

Data Acquisition

National Instruments Data Acquisition devices were used for the collection of data. Four load cells and four LVDT's were used to record the load and displacement information respectively. The Omega Dyne load cells are compression/tension capable and have a capacity of 22.2 kN (5 kips). The LVDT's have a stroke length of ± 50 mm (2 in) and a spring return armature for easy installation. The screw jacks, driven by micro stepping motors, each have a capacity of 22.2 kN (5 kips) giving a total load capability of 45 kN (10 kips).

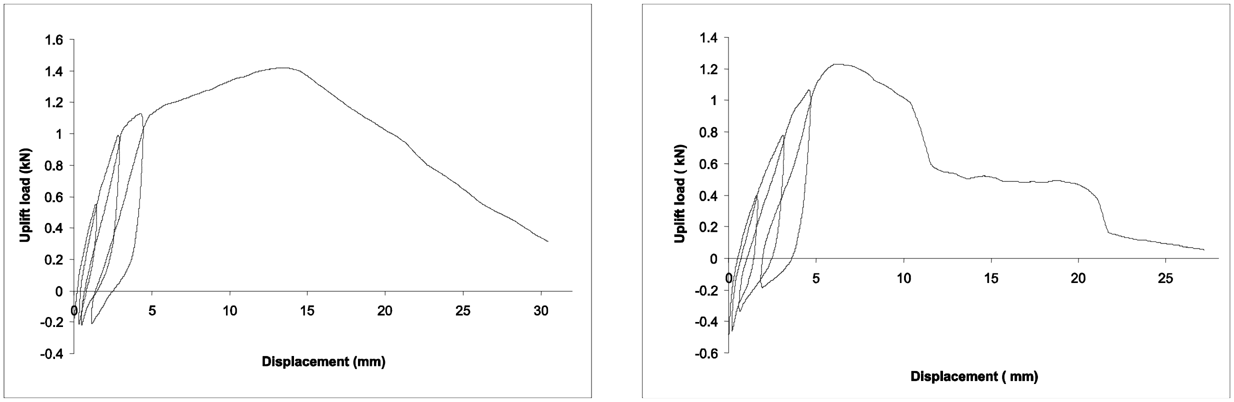
TEST PROCEDURE

For the present study, the connections between the top plates and ceiling joists were tested for uplift as opposed to testing the rafter to ceiling joist connections. The reason for this selection is that rafter to ceiling joist connections are more robust because they rely on nails acting in shear rather than withdrawal. Thus, the anticipation is that the joist to top plate connection serves as the weak link in the roof to wall connection.

The testing protocol DS-1761 was used as guidance for this study on evaluating the insitu capacity of toe nailed connections. A system of four connections was isolated from the other structural members by cutting the roof sheathing and rafters on either side of the segment. The decision to simultaneously test four connections is a compromise between the desire to capture some system effects and limitations in the size of the reaction frame and capacity of screw jacks. The system is then subjected to a cyclic uplift load using the spreader beam. Cyclic deflections corresponding to 1.6, 3.2 and 4.8 mm (0.0625, 0.125, 0.1875 in) were applied at a loading rate as recommended by DS 1761 for fastener withdrawal (2.54 mm/min (0.10in/min) ± 25 %.). Once the connection reached 4.8 mm (0.1875 in), the load was cycled back down then reapplied till failure. The dead load on each of the connections was recorded after the connection failed. The capacity of the connection is the maximum load sustained by the joint minus the dead load and the initial stiffness is taken as the secant modulus calculated at 1.59 mm (0.0625 in) displacement.

RESULTS AND DISCUSSION

A total of 100 (25 systems of four connections) connections were tested for this study. Out of the 100 connections tested, 81 had 2- 16d nails and 19 had 3-16d nails. Three types of failure modes were observed in the toe nail connection- 1) nail withdrawal 2) wood Split and 3) combination where one nail pulls out while the other nail splits/pulls through the wood. Figure 5a shows a typical load-displacement curve for a two nail connection that experiences withdrawal. This connection response is characterized by hysteretic behavior at relatively low levels of displacement caused by yielding of the nails and then experiences a gradual off loading as nail penetration decreases. Figure 5b show a connection which experienced a combination failure. The most significant difference between the two behaviors is that a combination failure is typified by a stepping in the post-ultimate region. This sudden drop in load is due to the brittle nature of splitting wood. Figure 6 provides photos of these two failure modes.



(a) (b)

FIGURE 5 TYPICAL RESPONSE OF CONNECTION WHICH FAILED DUE TO (A) NAIL WITHDRAWAL (B) COMBINATION.

Nail withdrawal was the mode of failure for 81 percent of the connections. A combined mode failure accounted for 16 percent while a complete splitting failure accounted for the remaining three percent. Clearly the dominant mode of failure is nail withdrawal and is considered to be the controlling behavior of these types of connections.

Uplift Capacity

The uplift capacity of the connection is the maximum load sustained by the connection minus the dead load and is defined as the ultimate strength of the connection. The two-nail connections in this study have an average uplift capacity of 1.52 kN (342 lbs) with a coefficient of variation (COV) of 0.35. The three-nail connections have, on average, a 30 percent larger uplift capacity than its two-nail counterpart (1.98 kN (445 lbs)). This is an interesting result in that one would have suspected approximately a 50 percent increase in capacity since there was a 50 percent increase in nails. One possibility for this discrepancy is explained by the nail configurations in the respective connections. In a two-nail connection the nails are driven at opposing angles and both must yield for the nails to withdrawal. However, in the three-nail connection, two nails angle in from one side while the third nail is driven at an opposing angle. This creates an imbalance in the resistance which causes the single nail to yield prior to the double nails. A small lateral shift occurs in the connection as one nail yields and the other two primarily avoid yielding and experience direct withdrawal. Figure 7 illustrates this phenomenon.

(a) (b)

Figure 6 Failures by (a) nail withdrawal and (b) wood split

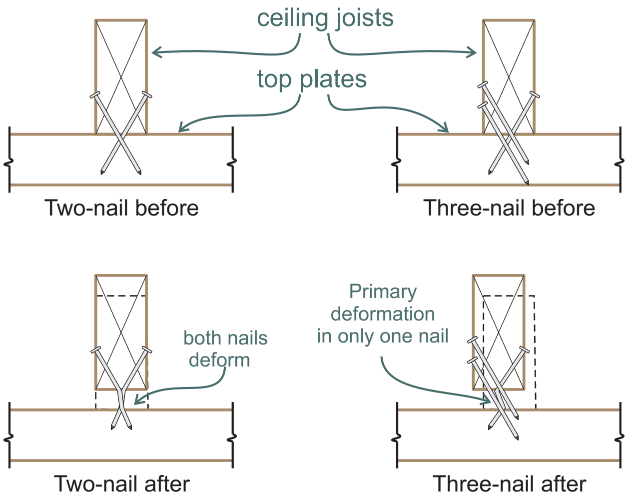


Figure 7 Suspected failure mechanism of three-nail connection.

Table 1 presents the uplift capacity statistics for the two types of toenail connections considered in this study and compares them with the findings from previous research studies.

The results from several other studies [Riley and Sadek 2003 and Judge et al. 2002] are not presented for comparison as their sample sizes are not known. The insitu uplift capacity was found to be less than the laboratory uplift capacity of toenail connections but the most notable change due to insitu testing is the appreciable increase in the COV of the capacity. The larger mean values and COVs for the laboratory tests are likely due to the controlled manner in which the test specimens were constructed. Insitu as-built testing has the ability to capture the variability connection behavior due to actual construction practices. As seen in Table 1, the COV estimates resulting from laboratory tests can underestimate actual COVs by as much as 50 percent. Failure to capture this uncertainty may significantly affect the reliability assessment of these connections under wind loads.

TABLE 1 COMPARATIVE TABLE OF UPLIFT CAPACITIES

Type of connection	No. of specimen	Average ultimate capacity kN (lbs)	COV	Study
Toenail				Present Study
2- 16d	81	1.52 (342)	0.35	
3 -16d	19	1.98 (445)	0.37	
Toenail- 2-16d box nail				Cheng, J., 2002
SPF	16	1.56 (350)	0.164	
DF	14	2.59 (584)	0.212	
SP	14	2.69 (605)	0.155	
Toenail- 3- 8d nail				Reed ,T.D. et al., 1997
Single	16	1.92 (430)	0.23	
Repetitive	2	2.99 (670)	na[1]	

[1] not available due to small sample size

Stiffness

The initial stiffnesses of the connections are calculated as the secant stiffness taken at a deflection of 1.59 mm (0.0625 in). This deflection is selected because in this range the stiffness is seen to be approximately linear. Furthermore, this deflection provides for moderate stability in the stiffness calculation as compared with estimates using smaller deflections. The average stiffness computed for the two-nail connections is 0.45 kN/mm (2590 kips/in). The associated COV is calculated to be 0.26. The three-nail connection has an average stiffness of 0.55 kN/mm (3159 kips/in) and a COV of 0.33.. While the COV of the three nail connection is higher than its two nail counterpart, the mean value for the three-nail connection is 22 percent greater. This follows the same trend as seen in the ultimate capacity comparison. One reason for higher COV for the three nail connection in both the cases can be attributed to the smaller number of samples.

Probability Models

The data collected pertaining to uplift capacity and stiffness was fit to relevant probability distributions. Numerous distribution types were considered and compared to the collected data through a goodness-of-fit (GOF) test. The Kolmogrov - Smirnov GOF with a 5% significance level is adopted for this study. This says that if the computed p-value is greater than 0.05 then the assumed distribution is considered to be plausible [Ang & Tang 2007]. The larger the p-value, the stronger this statement becomes.

For the three-nail connection, the strongest fit for both the uplift capacity and the stiffness is a two-parameter lognormal distribution - LN(λ, ζ). The distribution fitting for the two-nail connection showed that the uplift capacity is most strongly lognormal but the stiffness is best modeled as normally distributed with a p-value of 0.988. A lognormal distribution, however, is also considered to be a plausible model for the stiffness of this connection. Therefore, the lognormal distribution is recommended for all cases citing convenience of expressing connection behavior as jointly lognormal distributions. The parameters for each response are estimated using the maximum likelihood method and are reported in Table 3. The correlations between uplift capacity and stiffness are significantly correlated giving correlation coefficients of 0.69 and 0.77 for the two and three nail connections respectively.

TABLE 2 GOODNESS OF FIT VALUES FOR UPLIFT CAPACITY AND STIFFNESS

Type of test	**Uplift Capacity**		**Stiffness**	
	2 – 16d	**3 – 16d**	**2 – 16d**	**3 – 16d**
p-value	0.659	0.591	0.415	0.958
Distribution	lognormal	Lognormal	lognormal	lognormal

TABLE 3 PROPOSED PROBABILITY DISTRIBUTIONS FOR CONNECTION BEHAVIOR PARAMETERS

Connection Type	**Uplift capacity - kN (lbs)**			**Correlation**	**Stiffness – kN/m (lbs/in)**		
	Dist	λ	ζ		**Dist**	λ	ζ
2 – 16d	lognormal	0.362 (5.778)	0.34	0.69	lognormal	6.076 (7.826)	0.26
3 – 16d	lognormal	0.621 (6.036)	0.35	0.77	lognormal	6.258 (8.006)	0.32

The in-situ moisture content of the ceiling joists were measured and recorded using a two-prong moisture meter. Adjustments for lumber type were made and resulted in a range of moisture readings from 7 – 9.5 percent with an average of 8.5 percent. The correlation between moisture content and ultimate capacity was examined and found to be 0.14. This indicates that over the range considered that these two parameters are only slightly correlated and may likely be ignored in reliability studies.

CONCLUSION

Numbers of research studies have been conducted in the past to investigate the uplift capacity of the toenail connection. But most studies were done in the laboratory failing to account for the variability and uncertainty involved in as-built construction. The uncertainty in the field testing is due to variation in construction practices, influence from non structural members and the variation in wood quality. The present study aims to exploit structures slated for demolition to close the gap by carrying out uplift capacity tests of toenail connections insitu. Four light frame residential house were tested to evaluate the as-built roof to wall connection uplift capacities and their relative stiffnesses. The results were compared with previous studies and statistical inferences were made. The average ultimate capacity of two-nail connections was assessed to be less than comparable laboratory based results. The COV of the insitu capacities was higher than those calculated from other studies. This increase in COV is significant and important in understanding the performance and efficiency of these structural connections.

The probability distributions that describe the uplift capacity and relative secant stiffnesses are proposed. A jointly lognormal distribution is proposed to relate these two parameters. The correlation between uplift capacity and stiffness was investigated and as expected showed a moderate to high degree of correlation.

The proposed probabilistic models of key roof to wall connection characteristics will significantly contribute to the component reliability analysis and risk assessment of light frame wood structures. Furthermore, the data from this study can be used for analytical modeling of the joints. Hence it can be concluded that the present study would provide for improved analysis and connection design, leading to stronger and more efficient structures. One should note however that the results of this study are influenced by the age of the connections. Hence, comparisons with the performance of new structures should be made with caution unless an age factor is calculated.

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