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Load Path Uncertainty in a Wood Structure and the Effect on Structural Reliability

Wenqi Wang¹ and Christopher D. Eamon²

Abstract

The roof truss bearing points of a light-framed wood house were instrumented with load cells. It was found that under dead load alone, symmetric and theoretically identical truss reactions have significant variation. A similar degree of reaction discrepancy was found under the application of uplift pressures caused by hurricane winds. Analysis revealed that the majority of this discrepancy is caused by inherent uncertainties in load path. Although uncertainties in load magnitude and material resistance are accounted for in design by use of appropriate load and resistance factors, load path is generally taken to be deterministic. In this study, load path uncertainty in a test structure is statistically quantified and the effect on the reliability of wood structural members is investigated. Although large uncertainties in reactions were present, it was found that the resulting influence on reliability was modest, with decreases in component reliability index ranging from 5-15%.

KEY WORDS: Wood structure, Uncertainty, Load Path, Reliability

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1. Introduction

Uncertainties in loads and resistance have been formally recognized for decades in structural design. A significant body of information has been obtained over the years from empirical studies and analysis to estimate statistical parameters for various structural loads, such as dead load, occupancy live load, wind, earthquake, snow, and rain, among others. Similar parameters have been developed to characterize uncertainty in structural component resistance to moment, shear, axial tension and compression, and other force effects (see for example [1], and numerous others). This information was used to develop appropriate load and resistance factors in the various design standards for concrete, steel, wood, and other structural materials [2-5]. The study of wood structures has evolved significantly in the past 20 years, but only recently have reliability concepts taken a bigger role in the research of wood structures. A reliability-based design standard for wood structures was developed in 1988 with the publication of the ASCE and the National Forest Products Association's (NFPA) *Load and Resistance Factor Design for Engineered Wood Construction* [6]. The most recent version of the *National Design Specifications for Wood Construction* [5] incorporates both allowable stress design (ADS) and load and resistance factor design (LRFD).

Various studies have examined the reliability of wood structures for the general development of reliability-based design [7-14], as well as specific loads, components, and systems such as high wind events [15-18], wall and roof systems [10], [19-23], sheathing panels [24-28]; and shear walls [29-33]. The probabilistic models developed from this body of work for structural loads and resistance account for variations in load magnitude, frequency, and location, as well as variation in material strength and geometry that lead to uncertainty in component

resistance. However, they do not directly account for the uncertainty of interest in this study: uncertainty in how load is distributed throughout the structural system; i.e. uncertainty in load path.

Various modeling and experimental approaches have been suggested to predict and characterize the behavior of wood structural systems. Tuomi [34] discussed the full-scale load testing of structures, emphasizing the effect of variability of material, connection properties, orthotropy, and relative humidity of wood. Liu [35] suggested the need for developing better analytical procedures to predict the behavior of light-frame wood structures, while soon after, Kasal and Leichti [36] introduced a nonlinear finite-element model for light-frame stud walls, and later, investigated a full-scale light frame wood structure and presented a finite element model for predicting deformations and load distribution [37]. Other modeling advancements include Collins et al. [38], who suggested a 3-D finite element modeling approach to investigate various aspects of light frame building behavior under static and dynamic loading. Later, Asiz et al. [39] developed an advanced 3-D modeling approach to study the progressive collapse of wood structures, while Martin [40] used finite element analysis to study load paths through a wood structure. Doudak et al. [41] modeled wood light frame shear walls with openings using finite element models, and later demonstrated in a full-scale testing that system effects dominate the response for vertical and lateral loads, including the importance of considering the three-dimensional behavior, which originates from the relative stiff interconnection of roof, wall, and floor platform substructures [42].

The Forest Products Laboratory (US Dept. of Agriculture, Madison, WI) sponsored the construction of a light-framed wood house on the Florida coastline [43]. During construction,

the roof structure was instrumented with pressure gages, and load cells were placed between the connections of the roof structure and the supporting walls below.

Although the intended purpose of the structure was to measure wind loads, significant differences (greater than 50% in many cases) were found among symmetrically placed load cell reactions due to dead load alone. Similarly large discrepancies were found between the reactions caused by the application of uplift pressures caused by hurricane winds and those predicted from analysis when the same measured uplift pressures were applied on the analytical model. Although some discrepancy is expected, differences of this magnitude raised concerns about the predictability of wood structural behavior using deterministic modeling approaches.

Although a single case study is insufficient to generalize, results can be studied to raise issues of concern that may be relevant to other similar structures. The objective of this study is to identify the possible cause of this reaction uncertainty, to quantify the uncertainty in the load path (i.e. roof reactions) in the study structure, and to examine the potential effects on the reliability of wood components in general that a similar uncertainty in load path would entail.

2. Field Structure

The data are taken from a full-scale instrumented residential building that satisfies the structural construction requirements of the 2005 International Building Code (IBC) standards [44]. Sponsored by the Forest Products Laboratory, the structure was built by a local contractor using standard practices. The structure is located in Gulf Islands National Seashore Park in Gulf Breeze, Florida. Gulf Breeze is on a peninsula off of the Florida coast near the far western border of the state. The structure is sited in a clearing within a wooded area just north of Highway 98 (Gulf Breeze Parkway), as shown in Figure 1. Trees from 6 to 8 m (20 to 25 ft) tall

surround the clearing in which the structure is sited. The structure is assumed to correspond to an exposure category between B and C.

This is a single story, slab on grade structure, 13.3 m (44 ft) long by 8.5 m (28 ft) wide, with a floor-to-ceiling height of 3 m (10 ft). The structure has a hip roof which is constructed of clear-span engineered trusses spaced at 610 mm (24 in) on-center. The roof trusses are made of Southern Pine 2x4 (50 x 100 mm) dimensional lumber, ranging in grade from #2 to Dense Select Structural, depending on member location. The roof is sheathed with 13 mm ($\frac{1}{2}$ in) 4-ply CDX plywood decking, which is secured with 8d ring shank nails (length 60 mm, diameter 3.33 mm) spaced at 150 mm (6 in) throughout. This met the International Residential Code [45] requirements for hip roofs exposed to 100 mph or greater wind speeds. Note that gable roofs have more stringent fastener requirements at edge locations. The roof pitch is 4:12, with 600 mm (24 in) enclosed (box) overhangs on all sides and a mean roof height of 3.9 m (13 ft). Roof decking panel edges align on truss chords and are typically 4 x 8 (1.2 x 2.4 m) or 4 x 6 (1.2 x 1.8 m), as limited by the roof geometry. Wall studs are also spaced at 600 mm (24 in) on-center, and directly align with the truss supports above (see Figure 2). The interior ceiling and walls were later finished with 13 mm ($\frac{1}{2}$ in) gypsum board (not shown in the Figure). During construction, the house was instrumented with calibrated load cells between each truss reaction and the supporting wall (Figure 2 and Figure 3). Load cell locations are identified in a plan view of the structure by labels ending with “L” in Figure 4. The dead load data were collected when the wind speed was negligible, and thus represent reactions due to the roof dead load only. Because the house is bi-axially symmetric, the 68 load cells can be grouped into 17 sets of four data each that have theoretically identical values. For example, the data from load cells S22L, S01L, N22L,

and N01L (at the corners of the house; see Figure 4) are in a symmetric set and theoretically should have the same reactions.

In addition to dead load reactions, the reactions found from hurricane level wind uplift pressures were examined. The field structure was instrumented with 76 pressure taps on the roof surface, with locations indicated with labels ending with “P” in Figure 4. The pressure data made available for this study were recorded at 1 Hz from Hurricane Ivan. The instruments recorded the relative difference in pressure between the outer roof surface and roof interior. The data were continuously recorded from 9/10/2004 at 12:00 am until 9/23/2004 at 12:00 am. During that time, wind speed varied from 3.8 to 110 kph (2.4 to 68.6 mph). The peak wind speeds (averaged over 1-second gusts measured approximately 8 m above ground) 86 to 110 kph (53.9 to 68.6 mph) occurred from 9/16/2004 at 1:00 am to 9/16/2004 at 4:00 am (Greenwich Mean Time).

3. Computational Model

To obtain theoretical reaction values, a finite element (FE) model of the field structure was constructed. NASTRAN [46] was used for solution, while PATRAN [47] was used for pre- and post-processing. The structure is modeled as two substructures, the roof and the walls (see Figure 5). Note that in this study, the only response values of concern are the truss reactions (i.e. bearing point reactions on the supporting walls). Therefore, various modeling simplifications were introduced that, although do not significantly affect roof reactions, would not be suitable to model a wide variety of other responses such as connections, nonlinearities, and stress distributions within individual structural members.

For the roof, two-node beam elements are used to model the truss members. Connections between members and splices are modeled as pin joints. To explore the effect of joint stiffness, fully-fixed joints were also considered, which produced differences in truss member stresses, but no significant difference in truss bearing reactions were found. Sheathing panels are modeled with 4-node plate elements, with a typical physical element dimension of 6 in square. The sheathing is taken as 12 mm (0.5 in) thick, with elastic modulus of 4620 MPa (6.7×10^5 psi) and Poisson ratio of 0.02 [48]. Although plywood is an orthotropic material, it is layered symmetrically such that an isotropic material model can be used if the primary mode of bending is normally-oriented to the layers, which is typical and is the case for this study [49-50]. Individual sheathing panels are modeled, with adjacent panel edges disconnected. Beam elements representing the nails connect the sheathing to the top truss chords. These elements are oriented normal to the panel and are pinned at the (top) node connecting the element to the plates and are fixed to the truss members. Altering nail element bending and joint stiffness affected panel stress but produced no significant differences in truss reactions. Truss member elastic modulus E ranged from 9,660 to 15,200 MPa (1.6×10^6 to 2.2×10^6 psi). Specifically, top truss chords (Southern Pine (SP) No. 2) were assigned $E=11,000$ MPa, while depending on the truss structure and location, bottom chords were specified by the truss fabricator as either SP No. 2, SP Dense Select Structural ($E=13,100$ MPa), or Machine Stress Rated 2700f-2.2E ($E=15,200$ MPa), while all truss webs were taken as $E=9,660$ MPa (SP No. 3). Weight-density is taken as 5.5 kN/m^3 (35 lbs/ft³). Material properties are taken from NDS [5]. Since no evidence of panel pull-off, nail pull-through, or other structural damage or permanent deformations of the roof structure were identified, all material models are taken as linear elastic. This was confirmed in

the analysis, as all pull-off forces and member stresses were measured below expected failure levels on the model.

Wall components include studs, a double top plate and wall panels, which were modeled with beam elements. The top plate is modeled as a continuous member. Connections are assumed pin-connected, with the bottom end of the studs pin connected to the ground. To reduce computational effort, wall sheathing panels were not included on the model, as they were found to have no significant influence on the truss reactions. Lateral stability of the model was achieved by providing two horizontal constraints normal to the long side of the structure and a third normal to this direction, at the top of the wall level. The model has approximately 10,500 nodes, 5000 beam elements, and 6,800 plate elements.

Note that a 3 m wide garage door opening in the far east end of the south wall of the structure was spanned by a beam. The FEA analysis revealed that, due to the flexibility of this beam, the south side reactions of the trusses supported by this beam would differ by 10% or more from the north side reactions. Thus, the symmetric data sets affected by this opening (S16L-22L, S01L-S07L, N16L-S22L, and N01-N07L), were eliminated from the data analysis in the next section. The remaining ten symmetric dead load data sets are given in Table 1. According to the table, there is significant variation in what should be theoretically identical reaction values in the actual structure.

For wind pressure analysis, the roof was divided into areas of uniform dynamic pressure application separating the pressure taps. Here there were 24 areas each on the North and South sides of the roof and 14 areas each on the East and West sides, for a total of 76 independent wind load areas. A transient dynamic analysis was conducted using the FEA model loaded with the measured wind pressure data, and it was determined that the 1-Hz wind pressure data resulted in

no significant dynamic effects; i.e. the load effect of the 1-Hz wind pressure data could be effectively modeled as a series of sequential linear-elastic static analyses. This is expected from the high natural frequency of the roof panel as compared to the rate of change of wind load applied [24]. Therefore, to examine the difference in theoretical and actual load distribution throughout the structure due to the wind loads, wind pressures measured from the pressure taps from a sample of the most critical wind speeds normal (90°), parallel (0°), and diagonally (45°) to the roof were applied on the FEA model, and the truss reactions obtained. These were compared to the measured reactions on the field structure. Proportional reaction discrepancies (actual / analysis) from about 0.65 to 1.4 were observed, similar to the range of differences found in most of the theoretically symmetric dead load reaction data (Table 2).

4. Causes of Reaction Variation

Some possible sources of the reaction variation include: member size and shape variations such as member curvatures; material stiffness variations; roof weight variation (due to variations in component density or component volume); structural geometry variations producing eccentricities; and lack of plumb, square, and level members due to construction error, foundation settlement, or other effects; and instrument/measurement error. Based on an inspection of the structure and its instrumentation calibration records, most of these sources can be reasonably eliminated. Sources of discrepancy which cannot be eliminated include the variability in member stiffness and variability in member geometry, particularly stud length.

The effect of material stiffness variation was studied by changing the modulus of elasticity (E) of individual truss, roof panel, and stud members. It was found that a reasonable change in E (up to about 50%) for various members, in various patterns on the structure, has a

negligible impact on the reaction forces (less than 1% difference). Similar changes in material density to individual members had little effect on reactions as well (less than 1% difference).

The effect that stud length variability may have on reactions can be investigated by enforcing vertical displacements on the stud elements. The assumption here is that studs are likely to be slightly different in height (i.e. fractions of an inch, within tolerable construction practice), either due to variations in the cut length, floor slope, or both, and the connecting structural members that are continuous over the studs will be elastically deformed to conform to the variable geometry. Members primarily affected are top plates and roof sheathing panels. The latter are affected since they will be flexed and secured over slightly non-aligning truss top members, which may not align vertically due to the variations in truss support (i.e. wall stud).

As expected, it was found that the structure is sensitive to small length variations of the studs. A small settlement of one stud can lead to a significant decrease of the load cell value corresponding to that stud. Meanwhile, the reaction forces beside this stud tend to increase. A representative range of studs, S08 to S15, were studied here. The dead load reaction forces on the FEA model, as compared to the measured reactions on the field structure, are shown in Figure 6 (left). Significant variability exists in the experimental results that theoretically should be nearly uniform. Figure 6 (right) shows the results of imposing a small downward displacement of 0.02 inches on the studs under load cells S8, S11, S14, where the experimental trend can be closely matched. Thus, it appears reasonable that a significant source of variation among load cell reaction values may be attributable to differences in stud dimensions. Similar results were found when considering the reactions caused by wind pressures; an example result is shown in Figure 7.

5. Uncertainty Analysis

The variation in reaction value cannot be predicted by theoretical modeling, where consistent, idealized properties are assumed. Generally, differences between actual performance and the analysis prediction of performance is accounted for in probabilistically-calibrated (i.e. Load and Resistance Factor Design; LRFD) code development with inclusion of a professional factor (P) random variable in the reliability calibration procedure. However, the development of P does not consider variation in load path examined in this study, but rather the differences between component ultimate capacity as tested and as predicted from code design equations [1]. The additional uncertainty caused by load path variation may significantly affect (lower) the calculated reliability, and can be accounted for by considering an additional random variable similar to P . Using this approach, for consideration of the dead load reactions, the reaction value for each symmetric data set i can be regarded as a random variable R_i with mean value $\bar{R}_i$ and coefficient of variation (COV) V_{Ri} , statistical parameters which can be directly computed from the data belonging to set i . Note that COV is a normalized measure of variance used for convenience, and is found by dividing standard deviation σ by mean value; i.e. $V_{Ri} = \sigma_{Ri} / \bar{R}_i$. For this study, an estimate of the statistical parameters for an overall load path uncertainty random variable R_{all} considering all truss reaction data is desired. This is determined by first normalizing the data in each set to avoid weighting the uncertainty associated with one symmetric set of reactions more than another. Specifically, this is done by dividing each datum in a set by the mean value of the set. Thus, the mean value of each normalized set $\bar{R}_i$ has (dimensionless) unit value of 1.0. The mean value for R_{all} , $\bar{R}_{all}$, and the COV for R_{all} , V_{Rall} , are then taken as the means of $\bar{R}_i$ and V_{Ri} of the ten data sets shown in Table 1, respectively. That is, in this study,

$\bar{R}_{all} = (\sum \bar{R}_i)/10$ and $V_{Rall} = (\sum V_{Ri})/10$, where $i=1,10$. This results in a mean value of $\bar{R}_{all} = 1.0$ and $V_{Rall} = 0.24$.

For consideration of reaction variation due to the wind pressures, a similar process as described above can be followed, but since there exist no symmetric data sets (as the wind loads are asymmetric), COV is calculated based on differences from the measured and theoretically expected reaction results from the measured roof wind pressures. In this case, the FEA model is loaded with the recorded wind pressures, then the truss reactions calculated. The results are then normalized by dividing the actual truss reaction values by the reactions found from the analysis. This process is repeated for six critical wind pressure samples, where two most-critical cases each from wind directions of 0° , 45° , and 90° to the structure were considered, then the COV of the resulting data set is calculated. Similar results to dead load were found, with the final uncertainty RV statistics of $\bar{R}_{all} = 1.0$ and $V_{Rall} = 0.23$.

A cumulative distribution function (CDF) of the normalized dead load and wind load reaction data is given in Figure 8. The Chi-Square, Kolmogorov-Smirnov, and Anderson-Darling goodness-of-fit tests were used to determine if the distribution of R_{all} best fits a normal, lognormal or extreme type distributions. All three tests revealed that the data best fit a lognormal distribution.

A complication arises because V_{Rall} includes not only the reaction uncertainty due to load path, but also the reaction uncertainty due to variations in component dead weight as well. For accurate assessment of the effects of load path uncertainty, the effects of load path uncertainty alone must be isolated. This is accomplished by noting that a roof reaction R can be expressed as a function of two separate random variables D and U , where $R = D \cdot U$. Here D is roof dead load and U is the load path uncertainty factor. It can be shown that for the special case of a

product of two random variables, such as $R = D \cdot U$, the resulting variance of R , σ_R^2 , can be expressed as:

$$\sigma_R^2 = \left[(\bar{D} \cdot \sigma_R)^2 + (\bar{R} \cdot \sigma_D)^2 + (\sigma_R \cdot \sigma_D)^2 \right] (1 + \rho^2) \quad (1)$$

where ρ is the correlation coefficient for D and U . Assuming that dead load D and load path uncertainty U are independent, $\rho=0$ and eq. (1) can be simply written as the COV of R , V_R , as a function of the COVs of D and U [51]:

$$V_R = \sqrt{V_D^2 + V_U^2 + V_D^2 V_U^2} \quad (2)$$

Based on empirical studies, the COV of dead load for wood components is taken to be 0.10 [26]. From above, V_R was estimated to be $V_R = V_{Rall} = 0.24$. Solving eq. (2) for V_U results in $V_U = 0.22$. Similar results are obtained using the wind load pressure reactions. Note that in this case, although the actual dead loads are not known with certainty (due to geometric and member density variations), there is no uncertainty in the specific wind load data used in the analysis, as these were recorded directly from the pressure tap data. Therefore, the only uncertainty that must be extracted from load cell reactions to determine the COV of R when the recorded wind load is considered remains the uncertainty due to dead load. Thus, the same expression (eq. 2) is used when the wind load record is considered, but V_R is replaced with the COV found from the truss reactions under the wind loads ($V_R = 0.23$). As expected, the reaction COVs are almost identical for both cases (i.e. wind or dead load), indicating that load path uncertainty is about the same regardless of the load configuration applied. Thus, the random variable U , representing an uncertainty factor in load path for the truss reaction values, is represented with a (dimensionless) mean value of 1.0, COV of 0.22, and is estimated to be lognormally distributed.

6. Reliability Analysis

For components carrying roof dead, live and wind loads, as in the study house, the governing load combinations are: $1.2D + 1.6L_r + 0.8W$ and $1.2D + 1.6W + 0.5L_r$ [2], where it was found that the lowest reliability indices for components considered in this study (corresponding to the study structure) resulted from the latter combination. To examine the effects of load path uncertainty on wood structural component reliability, a reliability analysis can be conducted considering this governing load combination. For this analysis, dead load is taken as a normal RV [1, 10] with bias factor λ (i.e. ratio of mean value to nominal code value) of 1.05 and COV of 0.10. Fifty-year maximum wind load is modeled as an extreme type I RV, with λ of 0.78 and COV of 0.37 [1], with CDF as given by eq 2:

$$F_x(x) = \alpha \exp(-\exp(-\alpha(x - \mu))) \quad (3)$$

where parameters α and μ can be determined from the mean ($\bar{x}$) and standard deviation (σ) of the data: $\alpha = \pi / (\sqrt{6} \sigma)$ and $\mu = \bar{x} - \gamma / \alpha$, where γ is Euler's constant.

For roof live load, the 50-year maximum is considered, also modeled as an extreme type I with bias factor of 1.0 and COV of 0.25 [1, 10]. Structural resistance is modeled as an extreme type III distribution, with CDF shown in eq. 3, where the statistical parameters for sawn lumber and glued-laminated (glulam) beams and columns are taken from Rosowsky et al. [10], and given in Table 3.

$$F_x(x) = 1 - \exp((\varepsilon - x)/(\mu - \varepsilon))^k \quad (4)$$

where parameters μ , k , and ε are determined as a function of the mean value and standard deviation as well as the gamma function.

Moreover, since the RV statistics for U have been found to be the same for dead load reactions as well as the highly non-uniform [28] wind load reactions, U is taken as a structural characteristic independent of the type of load on the roof, and thus its statistical parameters are assumed to be similar when roof live load L_r is also applied.

Two cases are considered for reliability analysis. The first case does not consider load path uncertainty, and is used to determine baseline safety levels for wood components designed according to current code standards [5] using the limit state above. The second case considers load path uncertainty. The limit state function representing the first case (not considering load path uncertainty) is given by g (eq. 5), while the second case (considering load path uncertainty) is represented by g_U (eq. 6).

$$g = R - (W + D + L_r) \quad (5)$$

$$g_U = R - U(W + D + L_r) \quad (6)$$

Random variables are component resistance (R), dead load effect (D), live load effect (L), wind load effect (W) and load distribution uncertainty (U), the statistics for which were given earlier. The nominal value for resistance, R_n , is determined according to the current design procedure [5] for the governing limit state; i.e.

$$R_n = (1.2D + 1.6W + 0.5L_r) / \phi \quad (7)$$

Where ϕ is taken as 0.85 for flexure, 0.75 for shear, and 0.90 for compression. The mean values of the random variables are then obtained by multiplying by the appropriate bias factors. Note that W represents an uplift load on the roof.

Although the above analysis specifically applies to members subjected to loads applied to the roof as measured in the study structure, a similar process can be repeated for the typically governing limit state for members not specifically subjected to roof loads; i.e. $1.2D + 1.6L$, with

the assumption that the statistics for U are representative of a typical indeterminate wood structural system carrying dead, wind, or live load combinations.

7. Results

Probability of failure p_f for each limit state is determined from 1×10^6 Monte Carlo simulations. The results are converted to generalized reliability index, β , using the standard normal transformation $\beta = -\Phi^{-1}(p_f)$, where Φ is the standard normal cumulative distribution function. The results are given in Table 4 for eqs. 5 and 6, directly applicable to the study structure, and in Table 5 for the more general design case $1.2D + 1.6L$. In the tables, reliability indices are presented for limit states g (β_g) and g_u (β_{gu}). Note that the baseline values for reliability index β_g in Table 4 are somewhat lower than those reported by Rosowsky et al. [10] (approximately 2.3 compared to 2.7), although the same RV statistics are used. This difference occurs because in the previous work, reliability index was evaluated using a first order, second moment approach (FOSM), which assumes all RV distributions as normal, while values computed in this study are based on the actual distributions as described earlier (i.e. live load modeled as extreme type I and resistance as extreme type III).

As shown in the tables, reliability indices decrease with the inclusion of load path uncertainty, as expected. However, as shown in Table 3, differences are modest, with a uniform decrease in reliability index of about 5% for all cases considered. In Table 4, results are more significant, with decreases from 10-15%. This implies that, when considering the estimation of load path uncertainty, wood structural members placed in indeterminate load paths have a slightly lower reliability than expected, while the reliability of members placed in determinate load paths is unchanged from the expected value.

8. Conclusions

Roof reaction data were collected from a wood house structure under dead loads and wind pressures. It was found that significant variation was present in truss reactions which have theoretically identical values. From the data, load path uncertainty was characterized as a random variable U , and its statistical parameters were developed. It was found that U is best characterized with a mean value of 1.0, a coefficient of variation of 0.22, and is modeled with a lognormal distribution. Based on these results, reliability indices of sawn lumber and glulam beams and columns were estimated including this factor. Although large variations existed in the reaction data, this had only a modest impact on reliability, with decreases in reliability index ranging from about 5-15%. As variations in reliability of similar magnitude currently exist in most LRFD code formats, depending on factors such as dead to live load proportion, for example, variation due to load path uncertainty does not appear to be an immediate concern. Although the differences are not large, a small inconsistency in reliability may exist between members in determinate and indeterminate load paths. As these results represent a case study only, it is inappropriate to draw general conclusions across all wood structures without additional information. Ideally, load path variation information from multiple wood structures of different types, as well as at different locations within those structures, should be gathered. This may reveal different results or confirm the values found here. If confirmed, a slight decrease in resistance factor (for example, for sawn lumber in flexure, from approximately $\phi=0.85$ to $\phi=0.75$, based on the results here) may be appropriate for members in indeterminate load paths to provide a more consistent level of reliability for all structural members.

REFERENCES

- [1] Ellingwood B, Galambos TV, MacGregor JG, Cornell CA. Development of a probability based load criterion for American national standard A58 : building code requirements for minimum design loads in buildings and other structures. Washington, DC: U.S. Dept. of Commerce, National Bureau of Standards; 1980.
- [2] Minimum Design Loads for Buildings and Other Structures (ASCE 2010). Reston, VA: American Society of Civil Engineers; 2010.
- [3] Specifications for Structural Steel Buildings (AISC 2005). Chicago IL: American Institute of Steel Construction; 2005.
- [4] Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary. Farmington Hills, MI: American Concrete Institute; 2011.
- [5] National Design Specification for Wood Construction (NDS 2005). Washington, DC: American Forest & Paper Association, American Wood Council; 2005.
- [6] Load and resistance factor design for engineered wood construction: A pre-standard report. New York: ASCE; 1998
- [7] Ellingwood B. Reliability of wood structural elements. ASCE J Struct Div 1981; 107(1):73-81.
- [8] Goodman JR. Load and resistance factor design for engineered wood construction. Struct Des, Anal Test 1989; 736-740.
- [9] Ellingwood B. Load combinations for LRFD with wood construction. Struct Des, Anal Test 1989; 741-749
- [10] Rosowsky D, Gromala DS, Line P. Reliability-Based Code Calibration for Design of Wood Members Using Load and Resistance Factor Design. Journal of Structural Engineering. 2005;131(2):338-44.
- [11] Svensson S, Thelandersson S, Larse, H.J. Reliability of timber structures under long term loads. Materials and Structures 1999; 32:755-760.
- [12] Srensen, JD, Svensson, S, Dela Stang, B. Reliability-based calibration of load duration factors for timber structures. Structural Safety 2005;27(2):153-169.
- [13] Kohler J, Svensson S. Probabilistic representation of duration of load effects in timber structures. Engineering Structures 2011;33(2):462-467.

- [14] Kohler J, Fink, G. Reliability based code calibration of typical eurocode 5 design equations. World Conference on Timber Engineering 2012;4:99-103.
- [115] NAHB Research Center. Reliability of conventional residential construction: an assessment of roof component performance in hurricane andrew and typical wind regions of the united states. National Association of Home Builders; 1998
- [126] Ellingwood BR, Rosowsky D, Li Y, Kim, JH. Fragility assessment of light-frame wood construction subjected to wind and earthquake hazards. ASCE Journal of Structural Engineering 2004; 130(12):1921-30.
- [137] Li Y, and Ellingwood, BR. Hurricane damage to residential construction in the us: importance of uncertainty modeling in risk assessment. Engineering Structures 2006; 28(7): 1009-18.
- [148] Li Y, and Ellingwood, BR. Risk-based decision-making for multi-hazard mitigation for wood-frame residential construction. Australian Journal of Structural Engineering 2009; 9(1):17-26.
- [159] Salinas JJ, Gillard RG, and McMartin KC. Strength and structural safety of long-span light wood roof trusses. Reliability analysis using safety index. Canadian Journal of Civil Engineering 1989; 12(1):114-25.
- [20] Mtenga PV, Cramer SM, Pevrot AH, Wolfe RW. System Factors for Light Frame Wood Truss Assemblies. ASCE Journal of Structural Engineering 1989; 121(2): 290-300.
- [216] Zhao W, Woeste FE, Bender, DA. Effect of span-length on the reliability of truss bottom chords. Transactions of the American Society of Agricultural and Biological Engineers; 35(1): 303-310.
- [22] Bulleit, WM, Rosowsky, DV, Fridley, KJ, Criswell, ME. Reliability of wood structural systems. ASCE Journal of Structural Engineering 1993; 119(9): 2629-41.
- [23] Rosowsky DV, Yu G. Partial factor approach to repetitive-member system factors. Journal of structural engineering 2004; 130(11): 1829-41.
- [174] Rosowsky DV, Cheng N. Reliability of light-frame roof systems subject to wind uplift. National Association of Home Builders 1998.
- [25] Rosowsky DV, Cheng, N. Reliability of light-frame roofs in high-wind regions. i: wind loads. ASCE Journal of Structural Engineering 1999; 125(7): 725-33.
- [26] Rosowsky DV, Cheng, N. Reliability of light-frame roofs in high-wind regions. ii: reliability analysis." ASCE Journal of Structural Engineering 1999, 125(7): 734-39.

- [187] Lee KH, Rosowsky D. Fragility assessment for roof sheathing failure in high wind regions. *Engineering Structures* 2005; 27(6):857-68.
- [198] Rocha D, Eamon C, and Murphy J. Reliability analysis of roof sheathing panels exposed to a hurricane wind, *Structural Safety* 2011; 33:74-81
- [209] Li M, Lam F, Foschi R. Seismic reliability analysis of diagonal-braced and structural-panel- sheathed wood shear walls. *Journal of Structural Engineering* 2009; 135(5):587-96.
- [30] Rosowsky, DV. Reliability-based seismic design of wood shear walls. *Journal of Structural Engineering* 2002; 128(11):1439-53.
- [31] Van de Lindt JW, Rosowsky DV. Strength-based reliability of wood shearwalls subject to wind load. *Journal of Structural Engineering* 2005; 131(2): 359-63.
- [321] Van de Lindt JW, Huart JN, Rosowsky DV. Strength-based seismic reliability of wood shear walls designed according to AF&PA/ASCE 16. *Journal of Structural Engineering* 2005; 131(8):1307-12
- [33] Noory M, Smith I, Asiz A. Tests and numerical models for shear-walls with various layers. *World Conference on Timber Engineering* 2010.
- [34] Tuomi, RL. Full-scale testing of wood structures. *Full-scale load testing of structures ASTM STP 702*. ASTM, Philadelphia, Pa 1980: 45-62.
- [225] Liu H, Gopalaratnam VS, Nateghi F. Improving resistance of wood-frame houses. *Journal of Wind Engineering and Industrial Aerodynamics* 1990, 36: 669-707.
- [236] Kasal B, Leichti RJ. Nonlinear finite-element model for light-frame stud walls. *Journal of structural engineering* New York, NY. 1992;118(11):3122-35.
- [247] Kasal B, Leichti RJ, Itani RY. Nonlinear Finite-Element Model of Complete Light-Frame Wood Structures. *Journal of Structural Engineering*. 1994;120(1):100-19.
- [258] Collins M, Kasal B, Paevere P, Foliente GC. Three-dimensional model of light frame wood buildings. II: Experimental investigation and validation of analytical model. *Journal of Structural Engineering*. 2005;131(4):684-92.
- [39] Asiz A, Chui CY, Zhou L, Smith I. Three-dimensional numerical model of progressive failure in wood light-frame buildings. *World Conference on Timber Engineering* 2010.
- [40] Martin KG. Evaluation of system effects and structural load paths in a wood-frame building. M.S. thesis, Oregon State University, Corvallis, OR 2010.

- [41] Doudak G, Smith I, McClure G, Mohammad M, Lepper P. Tests and finite element models of wood light-frame shear walls with openings. *Progress in Structural Engineering and Materials*. 2006;8(4):165-74.
- [42] Doudak G, McClure G, Smith I. Experimental evaluation of load paths in light-frame wood structure. *Journal of Structural Engineering*. 2012;138(2):258-65.
- [43] Murphy, JF. Measuring wind reaction forces in structures. Durability of wood-framed housing, lessons learned from natural disasters. Biloxi, MS: Forest Products Society Conference; 2007.
- [44] International Building Code. Country Club Hills, IL: International Code Council; 2005.
- [45] International Residential Code. Country Club Hills, IL: International Code Council; 2005.
- [46] MSC/NASTRAN Users Guide. Los Angeles, CA: MacNeal-Schwendler Corporation; 2005.
- [47] MSC/PATRAN Users Guide. Los Angeles, CA: MacNeal-Schwendler Corporation; 2005.
- [48] Saliklis, EP and Falk, RH. Correlating off-axis tension tests to the shear modulus of wood based panels. *ASCE Journal of Structural Engineering* 2000; 126(5): 621-25.
- [49] Smulski, S. Engineered Wood Products: A Guide for Specifiers, Designers, and Users. Madison, WI: PFS Research Foundation; 1997.
- [50] Faherty, KF and Williamson, TG. Wood Engineering and Construction Handbook. New York: McGraw-Hill Companies; 1999.
- [51] Melchers, RE. Structural Reliability Analysis and Prediction. New York:Wiley; 1999.

Table 1. Symmetric Data Sets

Load Cells in Set	Load Cell Reactions (N)			
S15L, S08L, N15L, N08L	1273	823	1584	485
S14L, S09L, N14L, N09L	797	1015	1114	970
S13L, S10L, N13L, N10L	961	992	1380	1215
S12L, S11L, N12L, N11L	1113	716	992	939
W12L, W01L, E12L, E01L	481	405	552	458
W11L, W02L, E11L, E02L	814	587	641	743
W10L, W03L, E10L, E03L	263	334	525	352
W09L, W04L, E09L, E04L	432	623	347	512
W08L, W05L, E08L, E05L	641	405	298	276
W07L, W06L, E07L, E06L	294	343	494	556

Table 2. Typical Wind Reactions

Wind Direction (degrees)	Reaction (N)			
	N10L	N13L	S10L	S13L
0	1344	1700	1055	1206
45	1308	1664	1166	1317
90	1340	1691	1059	1242
FEA Result / Reaction				
0	1.20	1.36	0.95	1.15
45	1.25	1.19	0.75	0.77
90	0.86	1.02	0.68	0.83

Table 3. Resistance Statistics

Member	λ	COV
Sawn lumber beam, flexure	1.29	0.20
Sawn lumber beam, shear	1.11	0.20
Glulam beam, flexure	1.12	0.15
Glulam beam, shear	0.99	0.15
Sawn lumber column	1.09	0.15
Glulam column	1.09	0.15

Table 4. Reliability Results: $D+L_r+W$

load ratio &	sawn lumber		glulam	
failure mode	β_g	β_{gu}	β_g	β_{gu}
flexure	2.87	2.73	2.77	2.63
shear	2.87	2.73	2.78	2.64
compression	2.60	2.48	2.63	2.50

Table 5. Reliability Results: $D+L$

load ratio &	sawn lumber		glulam	
failure mode	β_g	β_{gu}	β_g	β_{gu}
flexure				
3.0	2.33	2.12	2.36	2.03
4.0	2.34	2.13	2.37	2.04
5.0	2.35	2.14	2.38	2.05
shear				
3.0	2.28	2.07	2.38	2.04
4.0	2.29	2.08	2.38	2.05
5.0	2.30	2.09	2.38	2.06
compression				
3.0	2.14	1.81	2.13	1.81
4.0	2.14	1.81	2.14	1.82
5.0	2.14	1.81	2.14	1.84

Fig. 1. Experimental house location.

Fig. 2. House construction, interior view.

Fig. 3. Load cell attachment.

Fig. 4. Instrumentation location.

Fig. 5. FEA model.

Fig. 6. Before adjustment (left) and after adjustment (right) dead loads.

Fig. 7. Before adjustment (left) and after adjustment (right), typical wind load.

Fig. 8. CDF of normalized reaction data.