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Hurricane damage to residential construction in the US: Importance of uncertainty modeling in risk assessment

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Abstract

The severe damage to residential construction and the social disruption caused by hurricanes in the past two decades in the United States demonstrate the need for a better understanding of residential building performance and improved performance prediction tools if the goal of mitigating hurricane hazards is to be achieved. In this paper, probabilistic risk assessment methods are developed to assess performance and reliability of low-rise light-frame wood residential construction in the United States subjected to hurricane hazards. In a fully coupled reliability analysis, structural system fragilities are convolved with hurricane hazard models expressed in terms of 3-s gust wind speed. Alternate models for structural fragility used in such analyses are tested statistically. Sources of inherent variability and epistemic (knowledge-based) uncertainties due to competing plausible hurricane wind speed models and databases on structural resistance are explicitly included in the reliability analysis. The impact of epistemic uncertainties on structural reliability and their implications for risk-informed decision-making are examined. Comparison of reliabilities of building components that are essential to maintain the integrity of the building envelope highlights the need to design and detail those components to reduce the overall vulnerability of a building to hurricane effects.

Keywords: Buildings; Design; Fragility; Hurricanes; Probability; Reliability; Statistics; Structural engineering; Uncertainty; Wood

1. Introduction

Hurricanes are among the most costly natural hazards to impact residential construction in the southeast coastal area of the United States. Hurricane Andrew in 1992 alone caused more than $20 billion in insured losses (in 2001 dollars), while Hurricanes Hugo (1989), Iniki (1992) and Opal (1995) caused a total of approximately $10 billion. As population growth in hurricane-prone areas continues to increase, the vulnerability of housing is increasing, with the prospect of even higher damages and losses in the future. To advance building practices by improving predictions of damage to residential construction and thereby supporting implementation of strategies for enhancing performance economically, this paper presents a probabilistic approach to assessing reliability of low-rise wood residential construction in hurricane-prone areas of the United States.

Managing risks to buildings from natural hazards such as hurricanes requires analysis of uncertainties arising from demand and building response. There are two general categories of uncertainty that affect the manner in which engineering decisions are made from the reliability analysis of structural systems. The first source is inherent variability, describing factors that are inherently random (aleatory) in nature; such factors would include the resistance of a roof component or structural system to wind uplift, resistance of glass to missile impact, or hurricane wind speeds and pressure coefficients describing the effect of wind on low-rise building structures. This source of uncertainty is essentially irreducible at the customary scales of engineering modeling. The second source of uncertainty is knowledge-based (or epistemic), and is due to a lack of or imperfect knowledge, assumptions and simplifications in modeling, and databases to support probabilistic descriptions of resistance and loading that are

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incomplete. Sources of epistemic uncertainty in reliability assessment of light-frame wood construction include alternative databases on structural resistance developed by different investigators that may be difficult to reconcile by engineering analysis, competing probabilistic models of hurricane wind fields based on different assumptions or simplifications, and the small-sample statistics that are characteristic of structural reliability analysis. In contrast to aleatory uncertainties, the epistemic uncertainties generally can be reduced by more comprehensive and costly efforts. Both categories of uncertainty and their impact on reliability are addressed in this paper.

Fragility analysis methodologies for light-frame construction previously have been developed for assessing the response of light-frame wood construction roof components exposed to extreme winds [23,8]. The focus of those papers was on an uncoupled risk analysis, with the fragility model expressing the uncertainty in performance of structural components and systems as a function of scenario windstorms (expressed in terms of 3-s gust wind speed). In contrast, this paper develops a fully coupled reliability analysis that convolves the structural fragility models with hurricane wind hazard models (several models are considered, as described subsequently), and examines the role of epistemic (or knowledge-based) uncertainties in risk-informed decision-making. The limit states of interest relate to those components that are essential to maintain the integrity of the building envelope—roofs, windows and doors. Implications for decision-making also are discussed.

2. Reliability analysis of wood components and systems

Reliability analysis provides a framework to incorporate uncertainties in structural system and loading and a tool to measure the performance and reliability of a structural component or system subjected to hurricane hazard on a consistent basis. Conceptually, the probability of failure under a spectrum of possible hurricane winds is determined by convolving the structural fragility curve and hurricane hazard curve,

$$P_f = \int_0^\infty F_R(v) f_V(v)\,\mathrm{d}v \tag{1}$$

in which $F_R(v)$ is the structural fragility, defined as the conditional probability of failure of certain limit states given a certain wind speed and $f_V(v)$ is the probability density function for hurricane wind speed. The wind speed, V, can be expressed with reference to an annual extreme wind speed, a 50-year extreme wind speed, or an extreme for another reference period, depending on the purpose of the risk analysis. For building code development purposes, the reference period typically is 50 years [6,5]. For insurance underwriting, where customary business practice is to annualize the risk, the reference period normally would be one year. The wind speeds derived from all hurricane wind field models can be derived for whatever reference period is relevant to the decision process. While there may be some change in structural capacity (fragility) due to natural material degradation processes over that same reference period, such changes are not considered in the following. It is assumed that databases on resistance of new construction are sufficient for estimating the fragilities in Eq. (1).

Evaluation of Eq. (1) requires that $F_R(v)$ and $f_V(v)$, and their distribution parameters, are known or can be determined from available databases. In fact, this seldom is the case. Indeed, as noted above, while F_R and f_V are based on plausible models, they are supported by limited data. Accordingly, both F_R and f_V are, in fact, random functions (dependent on the uncertainties in their statistical parameters and, at a higher level, in the choice of probabilistic model) and the estimator of the limit state probability, P_f, is described by a probability distribution. It is this "frequency of limit state probability" that describes the epistemic uncertainty in the reliability analysis. If the structural and climatological models are highly refined (and perhaps costly to implement), then the standard deviation (or standard error) in P_f is likely to be small; conversely, if those models are crude, then the standard error in P_f may be relatively large. In either event, it may be useful to identify a confidence level associated with P_f to reflect the credibility of the estimate based on limitations that may exist in the underlying models: for example, that value of P_f where the decision-maker is 95% confident that the true (unknown) P_f is less. Such statements would become relatively more conservative for less accurate structural or wind field models, and therefore allow the decision to reflect the level of uncertainty in the analysis supporting the decision. We will return to this point later.

3. Building performance and limit states

Post-hurricane disaster surveys have shown that the building envelope is the part of residential construction that is most vulnerable to hurricane-induced damage [17]. Once the envelope is breached, the building and its contents are increasingly likely to suffer severe damage from water or wind effects. Research conducted by Sparks et al. [24] has shown that following removal of one roof panel by wind uplift, the magnitude of losses can be on the order of 80% of the total insurance claim. Moreover, breach of envelope invariably leads to a dramatic increase in wind pressure on interior surfaces, which causes more severe damage to propagate to other structural and non-structural components. In some cases, total or partial collapse may ensue from loss of essential supporting systems, such as the roof structure or wall. For example, Manning and Nichols [13] found that damage or destruction of the roof structural system might cause walls to lose lateral support and lead to building collapse. Even in cases where some residential building systems appeared to be intact following the hurricane, those residences were uninhabitable for weeks or months afterward [16].

Thus, for the reasons discussed above, the building performance limit state in this study is defined as the breach of the building envelope. This limit state is closely correlated to system performance, damage and insured losses, and serves as a surrogate for more sophisticated system models (e.g., [25]). Specific component limit states included in this general performance limit state include roof panel uplift, failure of roof-to-wall connections due to uplift, and breakage of windows and doors due to excessive wind-induced pressure or projectile impact. For the first three limit states, component failure is defined as the point at which wind pressure or force exceeds the capacity of the component; for projectile impact, failure is defined as the point at which the impact velocity exceeds the minimum breaking velocity of glass [12].

The governing limit state for roof performance can be expressed as,

$$R - (W - D) = 0 \tag{2}$$

in which R = structural resistance to wind uplift, and D and W are, respectively the dead and wind load effects, all terms expressed in dimensionally consistent units. Note that the dead load counteracts wind uplift to roof panel and roof-to-wall connection, and is beneficial in reducing the vulnerability of the roof structure. Since the roof system is the component most vulnerable to suction or pressure from strong winds, the neglect of the dead load effect in reliability assessment leads to a pessimistic appraisal of the system reliability. The wind load, W, is a function of the square of the wind speed (and other variables, as described subsequently); if the wind speed is fixed ($V = v$), then the conditional limit state probability, $F_R(v)$ in Eq. (1) becomes an increasing function of v. This conditional probability can be determined either by numerical integration or by first-order (FO) reliability methods (e.g., [6]); in the latter case, $F_R(v)$ can be approximated as $\Phi(-\beta(v))$, in which $\Phi(\,)$ is the standard normal probability integral and β is the first-order reliability index. This latter approach is adopted in the following.

4. Probabilistic description of wind load and component capacities

The wind pressure acting on a low-rise building is determined from,

$$W = q_h[GC_p - GC_{pi}] \tag{3}$$

in which q_h = velocity pressure evaluated at mean roof height, G = gust factor, C_p = external pressure coefficient, C_{pi} = internal pressure coefficient. Eq. (3) is the basis for the winds pressures in *ASCE Standard 7* [1]. The velocity pressure is calculated as

$$q_h = 0.00256 K_h K_{zt} K_d V^2 \ (\mathrm{lb/ft^2}) \tag{4}$$

in which K_h = exposure factor, K_{zt} = topographic factor (taken equal to unity in this study), K_d = directional factor, V = 3 s wind speed at the height of 33 ft (10 m) in an open-country exposure. [In SI units, $q_h = 0.613 K_h K_{zt} K_d V^2$ (N/m^2).]

Table 1 summarizes the wind load statistics for a typical low-rise residential structure. The wind load statistics were obtained from a Delphi study of wind parameters [7]. The mean value of the dead load effect is based on the weight of the roof: 1.6 psf (77 Pa) and 15 psf (717 Pa) and for roof panel and roof-to-wall connection, respectively, while its coefficient of variation

Table 1
Wind load statistics

	Mean	COV	CDF
K_z (Exposure B)	0.57	0.12	Normal
K_z (Exposure C)	0.8	0.12	Normal
GC_p (C & C) Zone 3	1.81	0.22	Normal
GC_p (MWFRS)	0.86	0.15	Normal
GC_p (C & C) Zone 4	0.9	0.1	Normal
$GCpi$ (Enclosed)	0.15	0.05	Normal
$GCpi$ (Partial enclosed)	0.45	0.09	Normal
K_d	0.89	0.14	Normal

C & C: Component and cladding.
MWFRS: Main Wind-Force Resisting Systems.

is assumed to be 0.1. The dead load can be modeled by a normal distribution [6].

Structural resistance statistics in this study have been determined from laboratory tests of prototypical structural components and details. The probabilistic descriptions of components considered herein are summarized in Table 2.

The statistics on roof panel capacity in Table 2 define wind load uplift resistance of 4 ft by 8 ft (1.2 m by 2.4 m) roof panels with two nailing patterns [22]: nominal nail diameters are 0.113 in. and 0.131 in. (2.9 mm and 3.3 mm) for "6d" and "8d" nails, respectively. Panels are nailed at a spacing of 6 in. (150 mm) at the edges of perimeter and 12 in. (300 mm) in the panel interior. Table 2 also summarizes statistics on capacities of two common roof-to-wall connection details: three 8d (3.3 mm diameter) toenails and a H2.5 hurricane clip connection, installed as per manufacturer's specifications. Two sources of resistance statistics for the H2.5 clip connection were located: one from tests at Clemson University [21], and a second from [4]. The difference in test results from two qualified and independent investigators is one manifestation of the epistemic uncertainty associated with the use of different sources of experimental data in reliability assessment. The impact of these differences on estimated reliability of roof-to-wall connections is examined subsequently.

The statistics in Table 2 describing glass breakage in windows and doors due to windborne debris shows the relationship between projectile impact velocity and glass breakage [14]. It was found that if a 0.005 kg missile accelerates over a distance of 15 m, the impact velocity is about 35% of the velocity of the wind transporting the missile. Minor et al. [14] provided statistics on the effects of thickness on the resistance of glass to small missile impact (e.g., pea-gravel), and suggested that the minimum breaking velocity does not appear to be dependent on the area of the glass pane due to the local character of missile-induced failure. Annealed glass is selected as representative of material for windows and doors exposed to damage from excessive wind pressure. The glass breakage velocity is described by a lognormal distribution [25].

For glass breakage due to uniform wind pressure, the nominal resistance value for annealed glass specified in ASTM Standard E 1300-03 is the 60-s load duration uniform load with a probability of failure of 0.008 [2]. The coefficient of variation in failure pressure is assumed to be 0.25 [18]. The 60-s strength can be transformed to the 3-s gust wind

Table 2
Capacity statistics

Component		Failure mode	Mean	COV	CDF	Source
Roof panel	6d (0.113 in.) nails @ 6/12 in.	Uplift	25 psf	0.15	Normal	[22]
	8d (0.131 in.) nails @ 6/12 in.		60 psf	0.2	Normal	
Roof-to-wall connection	3–8d (0.131 in.) toe nails		411 lbs	0.34	Normal	[21]
	H2.5 Clip		1312 lbs	0.1	Normal	
			1212 lbs	0.15	Normal	[4]
Annealed glass (window and glass door)	1/8 in.; 20 sq ft	Pressure	54.47 psf	0.25	Weibull	[2,18, 15]
	3/16 in.; 40 sq ft		32.04 psf	0.25		
	3/16 in.; 20 sq ft		96.12 psf	0.25		
	3/16 in.; 40 sq ft		51.26 psf	0.25		
	3/16 in. thickness	Missile impact[a]	22.8 mph	0.07	Lognormal	[14]

1 in. = 25.4 mm; 1 ft^2 = 0.093 m^2; 1 lb = 4.45 N; 1 psf = 47.9 Pa.
[a] Assume 15 m of acceleration of 0.005 kg missile, impact velocity is 35% of carrying wind speed. Glass resistance is adjusted to 3 s duration loading.

speed used in ASCE Standard 7 by multiplying by a factor of 1.2 [15]. A Weibull distribution provides a suitable model of the cumulative distribution function defining probability of failure of brittle materials such as glass, and is used to model the uniform pressure strength of glass [18]. The aspect ratios (height/length) of glass windows and doors are assumed to vary between 1:1 and 2:1. Two different thicknesses of annealed glass, namely 1/8 in. (3 mm) and 3/16 (5 mm), are assumed for the reliability analyses, with different areas representing small (20 sq ft or 1.9 m^2) and large (40 sq ft or 3.7 m^2) windows or sliding glass doors.

5. Fragility assessment of key components of light-frame wood construction

The fragilities for typical wood-frame residential construction in the southeast US are performed using first-order reliability analysis to calculate the probability of failure of components that are vital to maintain the integrity of the building envelope and to minimize economic losses [20]. The building prototype in the study is a one-story single-family residence. The fragilities are expressed as the probability of failure as a function of the 3-s gust wind velocity at 33 ft (10 m) elevation in Exposure C, providing a direct connection between the conditional probability of failure and the manner in which hurricane hazard is specified in ASCE Standard 7 [1] at designated coastal sites. Selection of 3-s gust wind velocity as the control variable also facilitates the subsequent reliability analysis involving the convolution of structural fragility with hurricane hazard defined by alternative wind field models.

Two typical one-story single-family houses, denoted types A and B, are considered. Both are rectangular in plan, with mean roof height 12.5 ft (3.75 m). The roofs are gable roof systems with roof trusses spaced 2 ft (0.6 m) on center and with 6:12 slopes. The roof on home type A has an overhang extending 2 ft (0.6 m) from the wall, while the roof on home type B does not have a roof overhang. The roof panels are 4 ft × 8 ft (1.2 m by 2.4 m) with various nailing patterns. The building configurations described above are typical of residential construction in the southeast United States.

Fig. 1 illustrates roof panel fragilities for the different roof configurations and nailing details summarized in Table 2 above. The most severe wind pressures occur at the corners and edges of the roof. The presence of a roof overhang increases the edge panel failure rates. Fig. 2 shows the roof truss-to-wall connection fragility for the same one-story residence without a roof overhang in Exposures B and C. The benefit of using the hurricane clip rather than toe-nailing the rafters to the upper plate is apparent; at a wind speed of 150 mph (67 m/s), the predicted failure rate decreases from 80% to 5%.

Fig. 3 illustrates fragilities of windows and doors with different glass thicknesses and areas, where failure can occur either by excessive wind pressure or windborne missile impact. Note that the glass thicknesses in this figure (and the one to follow) represent the available data summarized in Table 2 [18, 15]. The vulnerability of larger windows and doors is evident in this figure.

Fig. 4 provides further insight on the relative vulnerability of doors and windows to excessive pressure and impact. It is assumed that the glass is 5 mm in thickness in all cases. It can be seen that failure is dominated by wind pressure for wind speeds less than approximately 110 mph (49 m/s); conversely the failure rate due to missile impact becomes much higher than that due to wind pressure when wind speeds exceed 110 mph (49 m/s) for large windows or sliding glass doors (with areas of about 40 sq ft). These latter failure rates presume a ready supply of small projectiles (pea gravel, debris from adjacent damaged buildings, etc). While such projectiles sometimes can be eliminated, in most cases this is not a dependable or effective means of hazard mitigation.

6. Selection of fragility models for risk assessment

The fragilities presented in Figs. 1 through 4 were obtained directly (point-wise, at increasing v) from a first-order reliability analysis of the governing limit state equation, expressed as a function of 3-s gust wind velocity, v. No distribution assumptions were made in developing these figures beyond the load and resistance statistics in the basic variables summarized in Tables 1 and 2. On the other hand, if a particular distribution can be identified as an appropriate fragility model, fragility model development can be greatly simplified as it becomes necessary to estimate only the two or

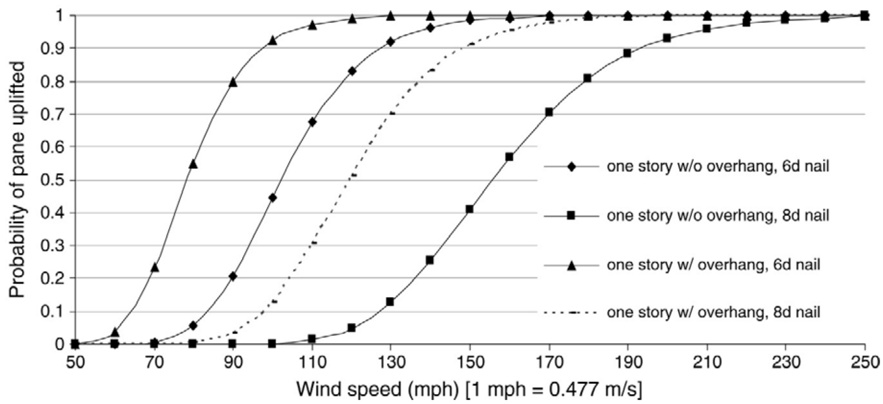

Fig. 1. Root panel fragility of two typical house (Exposure B).

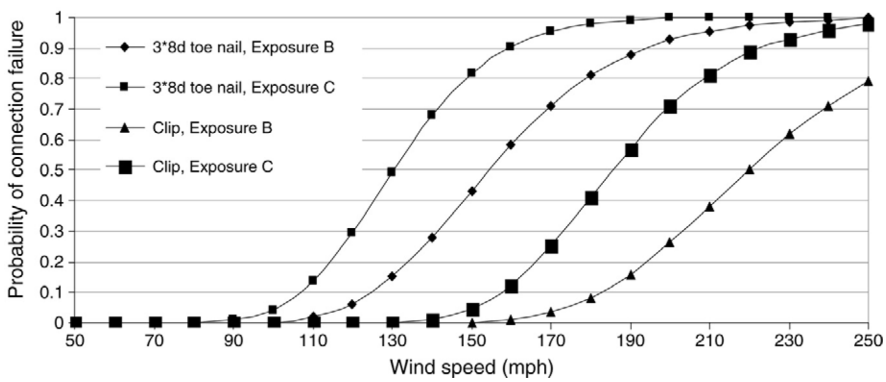

Fig. 2. Root-to-wall connection fragility of one-story house without roof overhang.

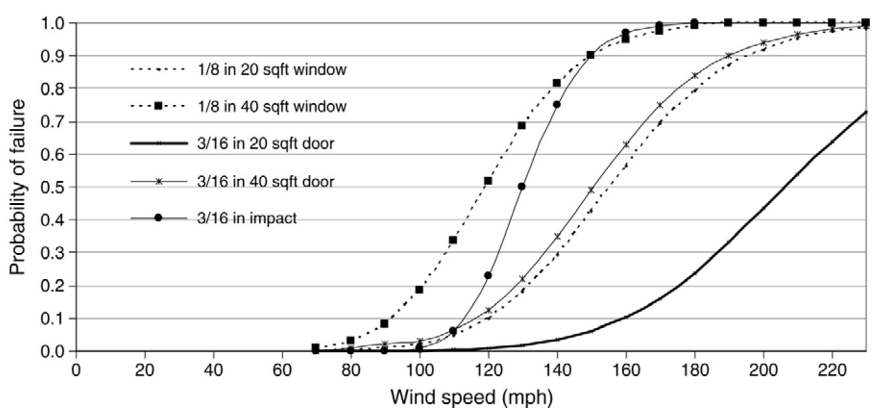

Fig. 3. Glass fragility due to pressure and impact.

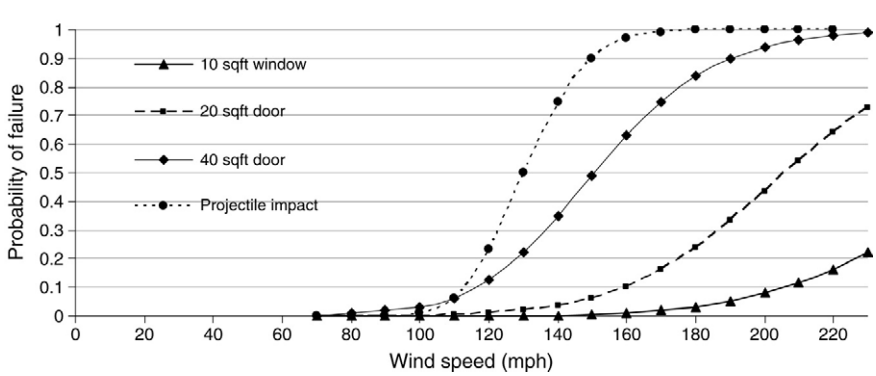

Fig. 4. 3/16 in. (5 mm) glass fragility due to pressure and impact.

three parameters that define that distribution. Development of fragility models (and their defining parameters) often requires a simulation-based finite element reliability assessment. Far fewer simulations are required to estimate the fragility parameters than to identify the entire fragility curve, making the fragility analysis and engineering decision analysis more computationally efficient.

The fragility of structural systems in other risk applications (e.g., concrete bridges; steel frames; nuclear power plant structures and components) commonly has been modeled by a lognormal cumulative distribution function (CDF). The question arises as to whether the lognormal distribution provides a suitable model to describe the fragility of light-frame wood structural components and systems. If the lognormal CDF can be shown to be a good fit for the fragilities determined from first-order analysis using the statistics in Tables 1 and 2, then its two parameters m_R and ζ_R, in which m_R = median capacity and ζ_R = logarithmic standard deviation of capacity, can be estimated by only two FO reliability analyses, and the previous approach, which required performing a sequence of FO analyses for increasing v, will not be necessary. The sampling errors associated with these estimates is much smaller than the sampling errors in the estimates of the lower fractiles (on the order of 5% to 10%) of the curves in Figs. 1 through 3, the values that are significant in code development, engineering decision analysis and hurricane risk mitigation. The lognormal CDF is described by,

$$F_R(v) = \Phi[\ln(v/m_R)/\zeta_R] \tag{5}$$

in which $\Phi(\cdot)$ = standard normal probability integral.

A series of tests were performed to test the hypothesis that the lognormal distribution is a suitable fragility model for light-frame wood construction. To begin, the CDFs $F_R(v)$ determined from FO analysis were plotted on lognormal paper. Such a plot is illustrated in Fig. 5 (on lognormal probability paper) for failure of a roof panel by wind uplift, where the linearity of the plot can be seen to provide support for the lognormal assumption. The probability plot correlation coefficient is close to 1, which is presumptive evidence that a lognormal CDF is a good fit for this fragility curve [9]. Similar results were obtained for capacities of other roof panels, roof trusses and for glass failure by excessive wind pressure. Indeed, it was surprising to find that the lognormal model for glass fragility appeared superior (in terms of linearity of probability plot) to a Weibull model, despite the fact that glass resistance to wind pressure is modeled by a Weibull distribution.

A subsequent series of Kolmogorov–Smirnov tests (at the 5% significance level) was performed on all fragilities developed by first-order methods, and confirmed that the lognormal CDF provides a reasonable model of fragility for light-frame wood construction.

7. Probabilistic models of hurricane hazard

The hurricane hazard is expressed by climatologists by a wind speed probability distribution for a standard averaging time (3 s), exposure (C-open), and elevation (10 m). Because of a lack of statistics of hurricane occurrence at specific mileposts along the coast and the need to estimate wind speeds at long return periods for design and risk analysis purposes, hurricanes are simulated probabilistically from fundamental climatological modeling principles and data (described in more detail below), and the resulting wind speeds at specific mileposts are derived from the wind field model. These wind speeds are post-processed statistically and are used to develop design wind speed maps for the *ASCE Standard 7* and for other purposes. Several hurricanes prediction models have been developed over the past two decades [3,11,10,27,28,26]. [The Vickery et al. [26] model is the basis for the wind speed contours in [1] in the southeast US.] The fundamental approach taken in these hurricane models is similar. Statistical distributions of key site-specific hurricane parameters, including rate of occurrence, central pressure, radius of hurricane, heading, crossing position along the coast, and dissipation following landfall are collected. Then Monte Carlo simulation is used to sample the hurricane parameters, and the wind speed is recorded when a mathematical representation of the hurricane passes the site. Of course, the physical models of the hurricane, such as filling rate models and wind field models, and the area over which the hurricane climatology is assumed to be uniform, which is used to derive the statistical characterization of the simulated hurricane, are different. In addition, some hurricane prediction models use a coast segment crossing approach, while others use a circular sub-region approach. These differences in wind field modeling affect the wind speed prediction.

Peterka and Shahid [19] found that the Weibull distribution provides a reasonable fit to the hurricane wind speed data at sites along the southeast coast of the US. Research conducted by Batts et al. [3], and simulations of hurricanes by Vickery et al. [26] using an empirical track model also confirm that a Weibull distribution is appropriate for hurricane wind speed prediction. The two-parameter Weibull distribution CDF is given

$$F_V(v) = P(V < v) = 1 - \exp[-(v/u)^\alpha]. \tag{6}$$

Parameters u and α are site-specific and can be determined from the published wind speed maps from the above hurricane studies that define the relationship between wind speed, v_T, and return period, T:

$$v_T = u[-\ln(1/T)]^{1/\alpha}. \tag{7}$$

Eq. (6) can be derived from Eq. (5) and the fact that

$$P(V > v_T) = 1 - (1/T). \tag{8}$$

Considering the area at the southern end of Florida as an example, the wind speed map developed by Vickery et al. [26] indicates that the 50, 100 and 1000 year return period peak (3-s) gust wind speeds in open terrain are 132, 150 and 182 mph (58, 67 and 85 m/s) respectively. The corresponding Weibull parameters are u = 61.07 and α = 1.769. Similar procedures are applied to the Batts et al. and Georgiou wind models for south Florida. The Weibull parameters for the three hurricane models are summarized in Table 3.

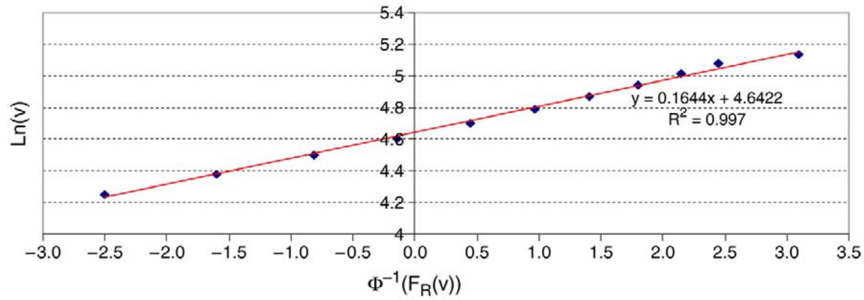

Fig. 5. Lognormal probability plot (6d nail panel fragility curve, exposure B).

Table 3
Weibull distribution parameters for different hurricane wind speed models (in mph) in south Florida

Hurricane wind speed model		Batts et al.	Vickery et al.	Georgiou
Return period	50 year	120	132	150
	100 year	130	150	162
	1000 year	155	182	208
Weibull distribution parameters	u	64.86	61.07	68.33
	α	2.218	1.769	1.738

1 mph = 0.447 m/s.

These distribution parameters are used in the following sections to complete several fully coupled analyses of limit state probabilities for the building components identified above.

8. Reliability analysis of key light-frame wood systems

Fragility models developed previously and the hurricane wind speed models summarized in Table 3 are used to determine point and interval estimates of limit state probabilities by Eq. (1). In this analysis, lognormal models of fragility are convolved with Weibull models of hurricane wind speed.

As a starting point, the probability of roof panel and roof-to-wall connections for several types of low-rise residential building in that community are determined, using the Vickery et al. [26] hurricane wind model. The results of this analysis are summarized in Table 4. [The log-median capacity, $\ln m_R$, is in units of log-velocity.] Note that the logarithmic standard deviations (dimensionless and approximately equal to the coefficient of variation in capacity) tend to cluster around 16%, despite the variety of structures, exposures and fasteners considered in this illustration.

A careful examination of the limit state probability in Eq. (1) reveals that a relatively small range of v contributes the major part of P_f. One can, for example, determine (by trial and error) the range of 3-s gust velocity that comprises (90%) of P_f; that range corresponds to 85–180 mph for roof panel uplift and 115–200 mph for toe-nailed roof-to-wall connection failure. Such results suggest that further meteorological refinements and risk mitigation efforts should be focused on storms with wind speeds in this range. It is interesting to note that wind speeds for hurricanes classified by the well-known Simpson–Saffir Scale as Category 5 storms are in excess of 191 mph (86 m/s) [based on 3-s gust over land [26]], indicating the relative importance of "lesser" hurricanes in overall risk management.

9. The role of epistemic uncertainties in reliability assessment

The fragility and reliability assessment presented in previous sections include uncertainties that are inherent in nature, to the extent that such uncertainties can be captured in the available databases. Epistemic uncertainties also play an important role in structural reliability and risk assessment, as noted in earlier discussions, as they impact the confidence with which decision-makers can make judgments from estimated limit state probabilities such as those presented in earlier sections. To illustrate this point, epistemic uncertainties in roof-to-wall connection resistance statistics from different data sources and in hurricane wind speed from the various wind field models are considered in this section.

Two sources of roof-to-wall hurricane clip resistance statistics obtained from experimental programs [21,4] are summarized in Table 2. Similarly, Table 3 summarizes the Weibull distribution parameters corresponding to three hurricane wind field models. The writers make no judgment as to the relative validity or credibility of these different sources of data that support reliability assessment, noting only that the investigators involved are all respected in their fields and the publications from which the data have been extracted all have been peer-reviewed. The purpose here is simply to illustrate the role of epistemic uncertainty in reliability assessment through a data collection and analysis effort that is similar to a challenge that might confront a typical decision-maker.

Table 5 summarizes the limit state probabilities associated with the roof-to-wall connection failure due to wind uplift obtained using these different wind speed models and sources of connection resistance statistics. When the sources of uncertainty for load and resistance are considered simultaneously, the uncertainty associated with the various wind speed models is far more significant than that associated

Table 4
Reliability of roof panel and roof-to-wall connection in south Florida

	Exposure	Type	Lognormal fragility model parameters		P_f
			$\ln m_R$	ζ_R	
Roof panel	Exposure B	Type A, 6d nail	4.6422	0.1644	0.0540
		Type A, 8d nail	5.0564	0.1600	0.0100
		Type B, 8d nail	4.7977	0.1675	0.0320
Roof-to-wall connection	Exposure B	Type A, Toe nails	5.0491	0.1757	0.0110
		Type A, Clip	5.3959	0.1428	0.0013
	Exposure C	Type A, Toe nails	4.8720	0.1576	0.0230
		Type A, Clip	5.2339	0.1341	0.0036

Table 5
Comparison of effects of hurricane models and sources of roof-to-wall clip connection resistance on P_f

Hurricane wind speed models		Batts et al.	Vickery et al.	Georgiou
Return period	50 year	120	132	150
	100 year	130	150	162
	1000 year	155	182	208
Weibull distribution parameters	u	64.86	61.07	68.33
	α	2.218	1.769	1.738
Probability of clip connection failure	Source [21]	0.000075	0.000438	0.00145
	Source [4]	0.000112	0.000478	0.00185

with the hurricane clip resistance model. The magnitude of difference in P_f due to source of connection data is about 30%–50%, while the difference in P_f resulting from the various wind speed models can reach an order of magnitude.

Similarly, Table 6 illustrates the effect of the different hurricane wind speed models on probabilities of failure by uplift of roof panels with 6d nailing and roof-to-wall connections with 3–8d toenailing for a house without roof overhang in exposure B. These panel and truss connection details correspond to minimum standards for residential construction practice. Here, the choice of hurricane wind speed model also has a significant impact on the estimated probability of panel failure, the highest P_f (0.137) being almost twice as high as the lowest P_f (0.077) for panels installed using 6d nails. Similar differences are observed for the 3–8d toenail roof-to-wall connection.

Finally, Table 7 illustrates the sensitivity of window failure probability to the choice of hurricane wind speed model.

The epistemic uncertainty (due to either hurricane and/or resistance) is displayed in a discrete form in Tables 5–7. As noted previously, the effect of epistemic uncertainty also can be displayed conveniently as a "frequency of probability", either discretely by a probability mass function or continuously through a probability density function. From such frequency representations of epistemic uncertainty, one can make confidence statements on the estimated P_f, which is a performance metric that is useful in the context of decision-making. For example, consider the results presented in Table 5, and assume that both data sets on clip capacity and all three hurricane wind field models are equally credible. The mean and standard deviation of estimated probability, P_f, are 0.00073, and 0.0007, respectively. The 95th percentile is estimated as approximately 0.0019. Thus, a decision-maker might say, "I am 95% confident that the annual failure rate of roof trusses attached to the wall with an H2.5 clip is less than 0.2%". More comprehensive test data or a better consensus on hurricane wind field models would push this value (0.2%) closer to the mean, 0.00073. Similarly, for a 40 sq ft glass panel 1/8 in. (3 mm) in thickness failing by excessive wind pressure, the estimated mean and standard deviation of P_f are 0.068 and 0.025, respectively, and one might state that he/she is 90% confident that the annual failure rate is less than 0.10. Differences in model credibility is taken into account in the assignment of likelihoods (weights) to the models. For example, if the Vickery [26] and Reed [21] models of hurricane wind speed and clip resistance are viewed as having twice the credibility of the competing models, then the mean and standard deviation of the estimated P_f are 0.00064 and 0.00057, respectively; the decision-maker might conclude that it is 95% probable that the annual failure rate of roof trusses attached to the wall with an H2.5 clip is less than 0.16%.

10. Conclusion

A probabilistic framework is proposed for evaluating the reliability of low-rise wood residential construction in hurricane-prone areas of the United States. It is assumed that contents damage and consequent structural damage result from breach of the building envelope, and the limit states are so defined: roof panels and trusses damaged by excessive wind uplift or breakage of window or door glass by excessive wind pressure or by windborne debris. Both structural fragility models to assess conditional reliability, given a certain level of wind speed, and hurricane wind hazard models are considered. Inherent and epistemic uncertainties are identified and displayed separately in the analysis. It was found that among all inherently random factors, wind speed is the most significant. This follows naturally from the fact that the wind

Table 6
Comparison of hurricane model effect on P_f: Type A house in exposure B

Hurricane wind speed models		Batts et al.	Vickery et al.	Georgiou
Return period	50 year	120	132	150
	100 year	130	150	162
	1000 year	155	182	208
Weibull distribution parameters	u	64.86	61.07	68.33
	α	2.218	1.769	1.738
P_f of 6d nail roof		0.077	0.09	0.137
P_f of 3–8d toe nails roof-to-wall connection		0.0053	0.011	0.024

Table 7
Comparison of effect of hurricane model and thickness and area of glass on P_f

Glass failure mode	Glass thickness and area	Lognormal fragility model parameters		P_f for different wind speed model		
		$\ln m_R$	ζ_R	Batts et al.	Vickery et al.	Georgiou
Pressure	1/8 in. 20 sq ft	5.0178	0.2086	0.0095	0.0170	0.0330
	1/8 in. 40 sq ft	4.7587	0.1974	0.0470	0.0600	0.0960
	3/16 in. 20 sq ft	5.3330	0.2248	0.0005	0.0022	0.0056
	3/16 in. 40 sq ft	4.9833	0.2096	0.0120	0.0200	0.0390
Impact	3/16 in.	4.9400	0.1070	0.0071	0.0170	0.0350

1 in. = 25.4 mm; 1 ft^2 = 0.093 m^2.

pressure is proportional to the square of the wind speed. In terms of epistemic uncertainty, the choice of wind model for risk assessment purposes is significant in terms of its impact on structural reliability and on engineering decision analysis. Developing a professional consensus on such models appears to be an important step toward risk-informed decision-making for individual buildings as well as for regional damage and loss estimation methods that utilize such models.

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