Hurricane Risk Assessment of Power Distribution Poles Considering Impacts of a Changing Climate

Sigridur Bjarnadottir[1]; Yue Li, M.ASCE[2]; and Mark G. Stewart, M.ASCE[3]

Abstract: Storm-related power outages cause approximately $270 million per year in repair costs in the United States. As a result of increasing sea surface temperatures caused by the changing climate, hurricane patterns (i.e., intensity/frequency) may change; however, there is much uncertainty as to how climate change may affect hurricane patterns. Implications of the changing hazard patterns on hurricane risk warrants an investigation to evaluate the potential impact of climate change on power distribution pole failure. This paper proposes a probabilistic framework to evaluate the vulnerability of power distribution poles to hurricanes under the potential impact of a changing climate. Two methods for the design of distribution poles in the United States, the National Electric Safety Code method and the ASCE method, are considered to investigate the difference of the vulnerability of a distribution pole subjected to hurricane hazard. The framework includes a reliability analysis of the designed power distribution poles using fragility analysis, the effects of degradation of timber poles, probabilistic wind models, and an assessment of the potential impacts of climate change on the annual failure probability of power distribution poles. This paper finds that climate change may have a significant effect on the structural failure probabilities of distribution poles. The age of the poles has a significant impact on the reliability of power distribution poles, which warrants the exploration of cost-effective methods to determine when a distribution pole should be replaced to ensure adequate strength to withstand wind loads. **DOI: 10.1061/(ASCE)IS.1943-555X.0000108.** *© 2013 American Society of Civil Engineers.*

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Introduction

Storm-related power outages cause approximately $270 million in annual repair costs in the United States (Johnson 2005). Hurricane Rita, in 2005, left 500,000 customers without power in Louisiana [Louisiana Public Service Commission (LPSC) 2005] and 1.5 million customers without power in Texas [Public Utility Commission of the State of Texas (PUCT) 2006]. After Hurricane Katrina in 2005, approximately 82% of customers in the Gulf Coast region lost power (Guikema et al. 2010), and approximately 50% of customers in New Orleans, Louisiana were without power for six weeks after the hurricane made landfall (Kwasinski et al. 2009). In 2004, during a six-week period, four hurricanes made landfall in Florida, causing over $1 billion in damages to power systems, making it the most destructive hurricane season in Florida for utility companies (Johnson 2005). In 1992, Hurricane Andrew caused the failure of approximately 10.1% timber distribution poles, resulting in the loss of power to 44% of Florida Power and Light Company's customers [Florida Power and Light Company (FP&LC) 2006; Larsen et al. 1996].

Reed (2008) investigated damage to power systems as a result of winter storms that were accompanied by strong wind flows. The study found that hurricanes and winter storms produce similar failure probabilities to power systems. Reed et al. (2010) also found, through an analysis of the damage to the infrastructure of the power system after Hurricane Rita, that high hurricane winds caused the majority of the damage. Despite the evident vulnerability of the power system to hurricane damage, approximately 53% of the U.S. population lives in coastal counties (Crosset et al. 2004). Florida utility companies have experienced a 20% increase in customers from 1994 to 2004 (Johnson 2005).

The distribution systems (lines and poles) are the most susceptible to damage caused by high wind. Reliability-based design methods for distribution poles were introduced in 1990s, and have since been integrated into design standards (Dagher 2001). However, the performance of the distribution poles remained unknown, resulting in nonuniform reliabilities in time and space (Bhuyan and Li 2006; Li et al. 2006). Furthermore, the effect of pole strength degradation on hurricane risk assessment for distribution poles is not clear. Windborne debris and trees can also cause a considerable amount of damage to the power system (Winkler et al. 2010). Debris includes anything that has been broken or destroyed. Debris and fallen trees combined with intense hurricane winds can be a serious problem to surrounding infrastructure. However, debris will not be included in the proposed framework, but it will be incorporated into this framework at a later time.

Lastly, current hurricane risk assessments of the power system consider only current climate conditions (Bhuyan and Li 2006; Li et al. 2006), but various studies (e.g., Elsner et al. 2008; Emanuel 2005; White House 2009) are suggesting that the hurricane

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intensity/frequency may change as a result of the warming global climate, which may result in an increased repair and/or replacement cost of distribution poles. For example, Peters et al. (2006) found that elevated CO_2 levels caused by enhanced greenhouse conditions could result in an increase in hurricane wind speeds of 10%, which could, in turn, result in loads on power distribution lines that are 15–20% higher than the design criteria. Therefore, it is becoming increasingly important to explore the effects a changing climate may have on the failure probability of the power system.

This paper develops a probabilistic method to evaluate the risk of hurricanes on the failure and reliability of timber power distribution poles. Two methods dominate the design of distribution poles in the United States (Malmedal and Sen 2003). The first method is a deterministic approach outlined in the National Electric Safety Code (NESC) (2002). The second method was developed by ASCE (2006). The framework proposed in this paper is threefold: design, reliability analysis, and assessing the potential impacts of climate change.

A typical distribution pole will be designed, considering both design methods and taking into account various loading conditions. Fragility curves will be developed for the distributions poles, considering both new poles and existing distribution poles that have been subjected to degradation. Finally, both design methods will be used to assess the impacts climate change may have on the annual structural failure probability of the distribution pole. The purpose of this paper is not to examine whether there is a direct relationship between climate change and changing patterns of wind hazard, nor to endorse any specific scenario of climate change (or lack thereof). Instead, the authors aim to propose a framework for hurricane risk assessment of power distribution poles to account for the potential impact of climate change.

Design of Power Distribution Poles: ASCE and NESC

A typical timber distribution pole system is seen in Fig. 1 that consists of a solid pole, three conductors, one neutral wire, and one communication wire. Wood materials account for approximately 99% of distribution poles in the United States [Utility Solid Waste Activities Group (USWAG) 2005], and Southern pine is the most common material for distribution poles, accounting for 75% of poles in the United States (Wolfe and Moody 1997).

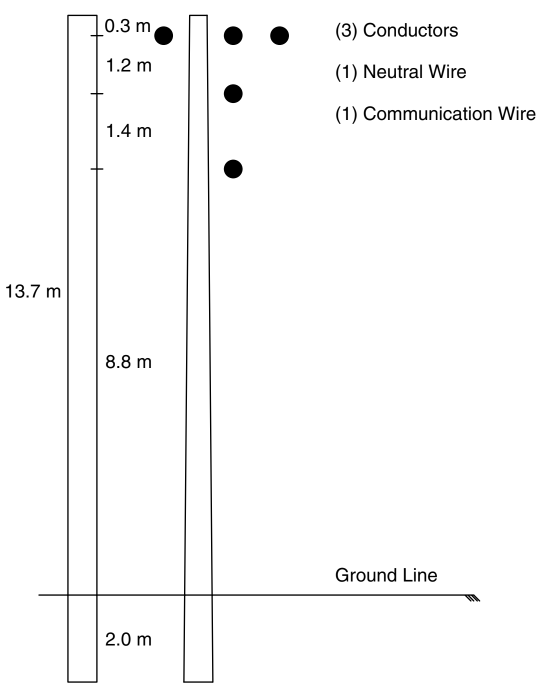

Fig. 1. Typical distribution pole system

Design on the Basis of ASCE versus NESC

Historically, distribution poles were designed by using the deterministic method prescribed in the NESC standard (NESC 2002). This allowable stress design (ASD) method is based on specific load factors and strength factors that are combined with zonal loading maps (Wolfe et al. 2001). The load and strength factors are determined on the basis of the grade of the construction of the distribution poles. To maintain consistent (or uniform) reliabilities for distribution poles, the load resistance factor design (LRFD) method was developed by the ASCE (Bhuyan and Li 2006; Dagher 2001), and is now typically used in distribution (utility) pole design (ASCE 2006). The LRFD is used to assess the performance of a distribution pole at various limit states. Although the ASCE design method is now often used in the design of new distribution poles, the NESC method is still used in the design of new distribution poles by some utilities in the United States, which is why both methods will be investigated. This paper will explore the differences between designing distribution poles with the ASCE and NESC design methods. The following formulation can be used for both methods:

$$R_n > \sum \frac{\gamma_j}{\varphi_j} S_{n,j} \tag{1}$$

where R_n = design (nominal) strength of the poles (e.g., design bending moment) and is identified by using design standards; φ_j = strength factor for load effect j; $S_{n,j}$ = design (nominal) load for load effect j; and γ_j = load factor for load effect j.

When designing distribution poles, the design method should be specified and the corresponding partial safety factors identified (i.e., values for φ and γ). Then, the design load (S_n) is determined, and consequently, the required design strength is determined by using Eq. (1). By means of the required design strength, the circumference of the pole at ground line can be estimated. From the required circumference, design standards [American National Standards Institute (ANSI) 2002] are used to determine the required class of pole and the corresponding design strength (R_n).

The two methods recommend different values for the strength (φ) and load (γ) factors. ASCE-111 (2006) recommends that the strength factor (φ) be 0.79 for a ground line bending moment with a coefficient of variation (COV) of 20% and that the load factor is $\gamma = 1.0$ for wind load. In addition, ASCE requires that the dead load should be adjusted with a factor of 1.1 when including the P-Δ effect. This case is referred to as Design Case 1 (Case 1 for brevity) in this paper.

The NESC method also uses strength and load factors for the design of distribution poles (NESC 2002). Both factors are determined on the basis of the grade of construction and the type of load designed for. Distribution poles are defined as Grade C construction. The strength factor is $\varphi = 0.85$ when wind load is considered (NESC 2002). There has been much discussion on what the value of the load factor should be. Malmedal and Sen (2003) recommended a load factor of $\gamma = 2.2$ for transverse wind loading (Design Case 2), whereas Bingel et al. (2003) suggested a value of $\gamma = 1.75$ (Design Case 3). Furthermore, Brown (2008) stated that distribution poles are typically designed with a load factor of $\gamma = 2.2$ but that the design load is reduced by half (i.e., $\gamma = 1.1$)

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Table 1. Four Design Cases

Design method	Design case	φ	γ
ASCE	1	0.79	1
NESC	2	0.85	2.2
	3	0.85	1.75
	4	0.85	1.1

(Design Case 4). Table 1 presents the four design cases on the basis of wind load conditions that will be investigated in this paper.

Design (Nominal) Load (S_n)

As shown in Fig. 1, the example pole system consists of a solid pole, three conductors, one neutral wire, and one communication wire; therefore, the design load of the distribution poles is affected by all these components and adjusted with an amplification factor to account for the P-Δ effect

$$S_n = \mathrm{amp} \cdot \sum_{i=1}^{N} F_i h_i \tag{2}$$

where S_n = design load (N-m); amp = amplification factor; F_i = wind force (N) on component i [Eq. (3)]; and h_i = distance (m) from ground line to the centroid of component i. The P-Δ effect refers to the deflected unbalance that occurs in a tapered distribution pole (ASCE 2006). More specifically, the pole leans over to resist the load, resulting in additional bending moments that affect the applied wind load (Bingel et al. 2003). ASCE-111 (2006) recommends utilizing the Gere-Carter method (1962) to account for the P-Δ effect in utility pole structures. The method involves determining an amplification factor that accounts for the additional bending moments caused by the P-Δ effect (Gere and Carter 1962), and this method is used herein.

The wind force acting on each component is described by using (ASCE 2008)

$$F_i = Q k_i V^2 I_{\mathrm{FW}} G_{\mathrm{RF}} C_f A_i \tag{3}$$

where Q = air density factor; k_i = terrain exposure coefficient for component i; V = 3-s gust wind speed; I_{FW} = importance factor; G_{RF} = gust response factor; C_f = force coefficient; and A_i = projected wind surface area normal to the direction of wind for component i.

Design (Nominal) Resistance (R_n)

The ANSI (2002) categorizes timber distribution poles into different classes on the basis of material. The ANSI (2002) assigns each class a permitted bending moment at ground line (i.e., 2.0 m from the base of the pole, see Fig. 1) depending on the height and the circumference of the poles. The circumference (C_g) of the poles is estimated from the required diameter (D_{req}), in which the required diameter is estimated from the design load (S_n) of the poles and the fiber stress of the species of timber (Brown 2008; Wolf and Kluge 2005)

$$D_{\mathrm{req}} = \sqrt[3]{\frac{32\left(\frac{\gamma}{\varphi}\right) S_n}{\pi \cdot F_0}} \tag{4}$$

where γ = load factor; φ = strength factor; and F_0 = designated fiber stress (ANSI 2002).

Once the circumference at ground line has been determined, the class of the poles is determined from design standards, and from the pole class, the design resistance (R_n) is found (ANSI 2002). Fig. 2 presents a flowchart of how the distribution poles are designed.

Degradation of Timber Poles

The strength of distribution poles degrades with age (Stewart and Goodman 1990; Gustavsen and Rolfseng 2000; Haldar and Tucker 2006). Fungal attacks are the main contributor to the deterioration of strength in timber poles because timber is an organic material (Baraneedaran et al. 2009). The key concern with fungal decay is referred to as in-ground decay because the poles are in direct contact with the soil creating, in many cases, optimal conditions for fungal attacks (Baraneedaran et al. 2009; Leicester et al. 2003; Wang et al. 2008b).

The modified resistance [Eq. (5)] is estimated for a specific age of pole to represent how strength deteriorates over time

$$R(t) = a(t) \cdot \mathrm{ME} \cdot R_n \tag{5}$$

where $R(t)$ = resistance after time t; $a(t)$ = degradation function; ME = the model error; and R_n = design strength of the pole. The design strength (found in design standards) is not equal to the actual

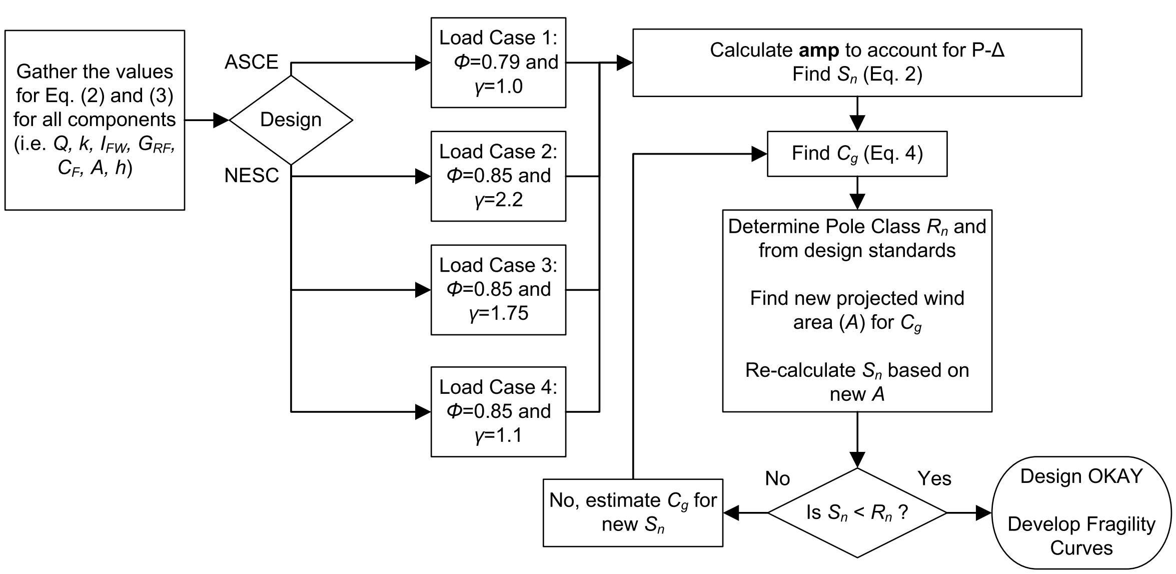

Fig. 2. Flowchart of the design process of the distribution pole

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strength of timber distribution poles because no distribution poles are identical, which can be attributed to factors, such as the cutting and material properties of each individual distribution pole. Within the estimations of the resistance at time t [Eq. (5)], this uncertainty is accounted for by including ME, which is relative to the design strength (Lupoi et al. 2006; Zhai and Stewart 2010).

Leicester et al. (2003) developed an engineering model that estimates the depth of decay in timber poles after a period of time, on the basis of a comprehensive study of timber degradation. Wang et al. (2008b) took the model further and developed a model that estimates the strength of timber poles after a specific time period on the basis of the decay depth. These models were developed on the basis of a survey of timber poles in Australia but will be used in this paper as a starting point to estimate the degradation of timber pole strength in the United States.

The degradation function $[a(t)]$ from Wang et al. (2008b) is used herein to represent how the strength of distribution poles deteriorates over time. In the following equation, the derivation of the degradation function is outlined; the interested reader is directed to Wang et al. (2008b) for details. The degradation function is estimated as (Wang et al. 2008b)

$$a(t) = \frac{\pi}{32}[D - d(t)]^3 \tag{6}$$

where D = initial diameter (mm) of the pole; and $d(t)$ = decay depth (mm) after time t.

Timber poles are typically composed of either solely heartwood or a combination of sapwood and heartwood. For simplicity, it is assumed herein that the timber distribution poles are composed of only untreated heartwood, so the degradation model can be implemented directly (Wang et al. 2008b). The future work will involve modifying the degradation model to account for a combination of sapwood and heartwood. According to the Australian Standard on Timber Durability (AS 56042005), the Durability Class of timber is assigned depending on the expected service life. The average service life of distribution poles in the United States is estimated at 32 years (Morrell 2005), corresponding to a Durability Class 2 according to AS 5604 (2005). The hardness of Southern pine is classified as soft (AS 5604-2005); these characteristics will be assumed to apply to Southern pine timber poles in the United States because no such data are obtainable in the United States. When more information becomes available, these assumptions can be modified.

The rate of decay can be estimated:

$$r = k_{\mathrm{wood}} k_{\mathrm{climate}}\ (\mathrm{mm/year}) \tag{7}$$

where k_{wood} = wood parameter; and k_{climate} = climate parameter. The wood parameter is 0.38, on the basis of that Durability Class 2. The climate parameter is assumed to be 1.5 (Climate Class of B) (Wang et al. 2008b). According to the Köppen-Geiger Climate Classification, Climate Class B refers to a temperate subtropical climate, and because Miami-Dade County, Florida is the location chosen for the illustrative example included herein, this climate class is assumed to be appropriate because Miami-Dade County has a subtropical climate (Peel et al. 2007).

Wang et al. (2008b) found that decay in timber poles does not commence immediately after a pole has been erected, but there is, in fact, a period of time during which the decay is negligible. This is referred to as the time lag (t_{lag})

$$t_{\mathrm{lag}} = 3\, r^{-0.4}\ (\mathrm{year}) \tag{8}$$

Then the time in which decay reaches its threshold can be determined (Wang et al. 2008a)

$$t_{d_0} = t_{\mathrm{lag}} + \frac{d_0}{r}\ (\mathrm{year}) \tag{9}$$

where d_0 = decay threshold. A value of d_0 = 5 mm is recommended when no data is available (Wang et al. 2008a). Once the decay threshold has been reached, the decay depth $[d(t)]$ is estimated on the basis of the rate of decay

$$d(t) = \begin{cases} ct^2 & t \le t_{d_0} \\ (t - t_{\mathrm{lag}})r & t > t_{d_0} \end{cases} \tag{10}$$

where $c = (d_0/t_{d_0}^2)$.

Fig. 3 shows the decay depth (mm) over time, by using the previously mentioned assumptions for the decay rate [see Eq. (7)] and Climate Class B. The figure clearly shows the time lag in which the decay depth is negligible, which is estimated to be 3.8 years. After the decay threshold has been obtained (5 mm), the rate of decay is constant, or 0.50 mm/year for this example.

For illustration purposes, Fig. 4 shows how the pole bending strength may change over time, considering a hypothetical Class 5 Southern Pine pole. The designated fiber stress of a Class 5 pole is 55.4 MPa with an initial diameter of 263 mm, according to design standards (ANSI 2002). The bending strength during the time lag is approximately 98,900 N-m, which is consistent with the design bending moment found in ANSI (2002) for this particular size and type of pole. Similar figures were developed for various classes of poles, and it was found that the bending strength estimated during the time lag with Eq. (5) is consistent with the design bending moment provided in the design standards (ANSI 2002).

The uncertainty which arises from using this predictive model to estimate the degradation of the strength must be accounted for. Wang et al. (2008a) stated that the uncertainty within the decay model [Eq. (6)], the timber and the climate parameters contribute to the uncertainty of the decay depth (COV_d)

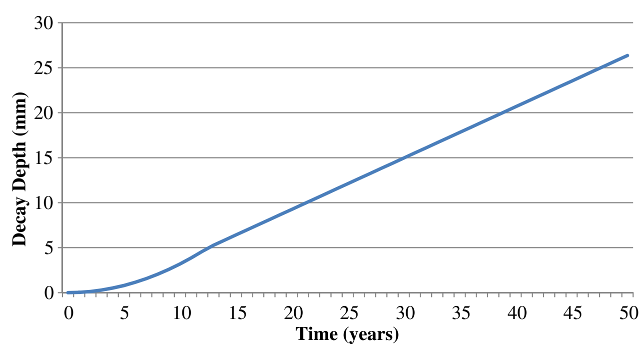

Fig. 3. Decay depth over time

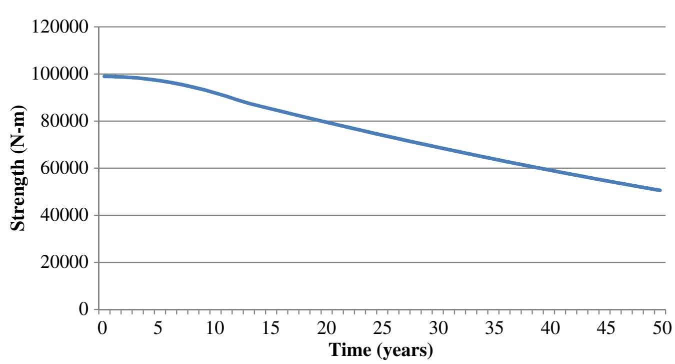

Fig. 4. Bending strength over time

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$$\mathrm{COV}_d = \sqrt{\mathrm{COV}_{\mathrm{wood}}^2 + \mathrm{COV}_{\mathrm{climate}}^2 + \mathrm{COV}_{\mathrm{dm}}^2} \quad (11)$$

where COV_d, $\mathrm{COV}_{\mathrm{wood}}$, $\mathrm{COV}_{\mathrm{climate}}$, and $\mathrm{COV}_{\mathrm{dm}}$ = the COV of the decay depth [$d(t)$], the wood parameter (k_{wood}), the climate parameter (k_{climate}), and the predictive decay model, respectively. The $\mathrm{COV}_{\mathrm{dm}}$ accounts for modeling errors within the decay model, similar to the ME found in Eq. (5). On the basis of the extensive testing of timber poles, $\mathrm{COV}_{\mathrm{wood}} = 0.55$ for timber poles of Durability Class 2, $\mathrm{COV}_{\mathrm{climate}} = 0.55$ for all climate classes, and $\mathrm{COV}_{\mathrm{dm}} = 0.50$ (Wang et al. 2008b). Therefore, for a Durability Class 2 timber pole, $\mathrm{COV}_d = 0.9$. The uncertainty arising within the strength estimations stems from the uncertainty of the decay depth and the initial uncertainty of the pole strength (Wang et al. 2008a). The COV of the pole strength at time t is given (Wang et al. 2008a)

$$\mathrm{COV}_R(t)^2 = \mathrm{COV}_R(0)^2 + \left[\frac{6\mathrm{COV}_d d(t)}{D - 2d(t)}\right]^2 \quad (12)$$

where $\mathrm{COV}_R(0)$ = COV of the initial strength; COV_d = COV of the decay depth; $d(t)$ = decay depth at time t; and D = initial diameter of the pole.

Risk Assessment

Reliability Analysis

The reliability of distribution poles is defined as the probability that the poles will fulfill the performance criteria (i.e., not fail) (Li et al. 2006). The performance of distribution poles is dependent on various intervening variables (e.g., wind speed, pole strength) and design parameters (e.g., pole height and size). There is much uncertainty associated with the variables, which must be accounted for in the analysis of the performance of the distribution poles. The general limit state function of the poles for both design methods is given as

$$G(t) = R(t) - S(t) \quad (13)$$

where $R(t)$ = actual capacity of the poles at time t [Eq. (5)]; and $S(t)$ = actual load at time t. The mean of the pole strength is determined from available test data (Vanderbilt et al. 1982). The actual load [$S(t)$] is determined by accounting for the uncertainties present within the variables of Eq. (2). The $\Pr(G > 0)$ is defined as the reliability of the pole, whereas the probability of failure is defined as $\Pr(G > 0)$. Monte Carlo Simulation (MCS) is utilized to estimate the probability of failure.

Hurricane Fragility Analysis

Hurricane fragility is defined as the conditional probability of failure of a structural member or structural system as a function of wind speed (Li and Ellingwood 2006). The structural fragility of infrastructure systems is often modeled as a lognormal cumulative distribution function (CDF) (Li and Ellingwood 2006).

$$F_D(V) = \Phi[\ln(V/m_R)/\xi_R] \quad (14)$$

where V = 3-s gust wind speed; m_R = median capacity or resistance; ξ_R = logarithmic standard deviation of the capacity or resistance; and $\Phi(\cdot)$ = standard normal probability integral. Fig. 5 presents a flowchart of the framework for developing the hurricane fragility curves on the basis of the designed poles (from Fig. 2). The terminology on Fig. 5, i.e., consider statistics, refers to assigning variables a statistical distribution, mean, and COV from either available test data or on the basis of previous studies or assumptions.

Annual Probability of Failure (P_f)

The expected annual probability of failure (P_f) caused by hurricane hazard can be determined by convolving the hurricane fragility $F_D(v)$ and the probability density function (PDF) of the annual hurricane wind speed model $f_v(v)$ (Li and Ellingwood 2006)

$$P_f = \int F_D(V) \cdot f_v(V) dV \quad (15)$$

The Weibull distribution is used to model the maximum annual 3-s gust wind speed in the United States (Li and Ellingwood 2006), which is assumed to be at a height of 10 m on open terrain. The PDF of the Weibull distribution, assuming that wind speeds are nonstationary because of climate change, is given as (Bjarnadottir et al. 2011)

$$f_v(V,t) = \frac{\alpha(t)}{u(t)} \cdot \left(\frac{V}{u(t)}\right)^{\alpha(t)-1} \exp\left[-\left(\frac{V}{u(t)}\right)^{\alpha(t)}\right] \quad (16)$$

where V = 3-s gust wind speed; and α and u = time-dependent and site-specific parameters.

Potential Impacts of Climate Change

Some studies have indicated that hurricane activity in the Atlantic Ocean has increased significantly since 1995 (Goldenberg et al. 2001). Emanuel (2005) found that an increase in sea surface temperature (SST) of 1°C could produce an increase of 5% in the peak wind speed of a tropical cyclone. The frequency of hurricanes may increase as well [Climate Change Science Program (CCSP) 2008]. Broccoli and Manabe (1990) found that if CO_2 levels double, the frequency of hurricanes may increase by 6%.

There is a great uncertainty in assessing the impact of climate change on hurricane intensity/frequency in the United States (Stainforth 2005). However, information on how climate change may affect hurricane patterns is available from other regions of the world. Studies have found that there may be an increase in high precipitation events (e.g., tropical cyclones) in Southeast Asia and Japan (Chang 2010; Webersik et al. 2010). Studies [Australian Greenhouse Office (AGO) 2007; Walsh et al. 2002] have found that wind speeds may increase by 5–10% in Queensland, Australia by 2050. Similarly, Vickery et al. (2009) suggested that, in the United States, increases in wind speeds of up to 10% may be possible. It can be expected that power systems may experience more failure as a result of the changing climate. Therefore, it is imperative to assess the potential impacts climate change may have on the annual P_f of distribution poles.

Illustrative Example

Typical Southern pine timber poles (i.e., 14-m high with spacing of 46 m) in Miami-Dade County, Florida are designed by using both the ASCE and the NESC design methods to illustrate the proposed framework. It is assumed that the pole supports three conductors, one neutral wire, and one communication cable. Timber poles are classified on the basis of the load capacity of the poles (ANSI 2002).

Design of Distribution Poles

The design load (S_n) is assumed to be the total ground line moment of the poles, which is the summation of the ground line moment of each component (i.e., pole, conductors, neutral wire, and communication cable). Table 2 shows the design values for typical poles

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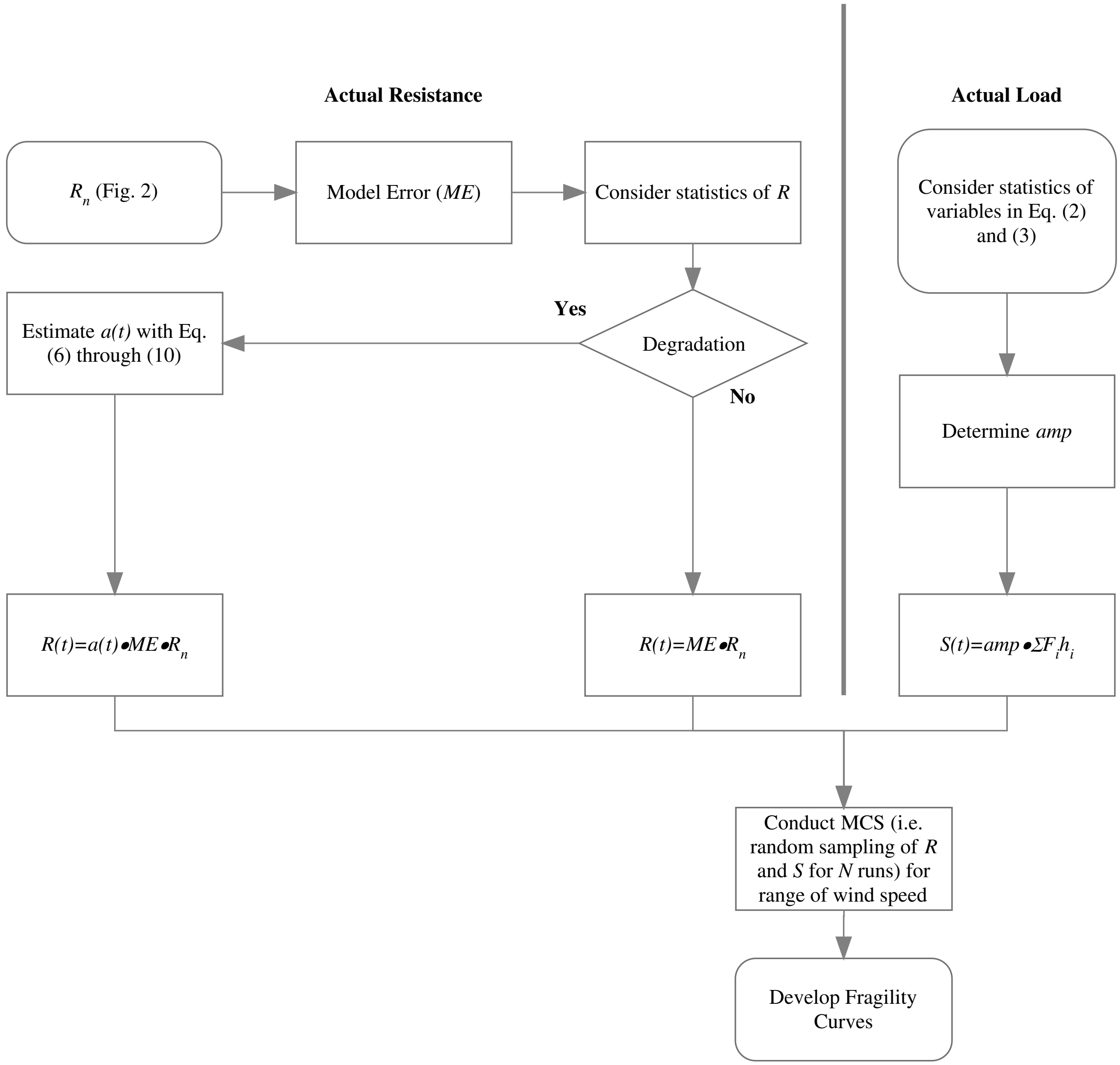

Fig. 5. Flowchart of the development of fragility curves

Table 2. Design Variables and Values

Variable	Description	Value	Note
Q	Air density factor	0.00256	Standard value
I_{FW}	Importance factor	1	Utility poles
G_{RF}	Gust response factor	0.96/0.81	Pole/wires
C_f	Force coefficient	1	Circle-shaped pole
V	Wind speed (m/s)	40	Assumed value
A	Projected wind area (m^2)	Varies/1.5/10/3.2	Pole/conductor/neutral wire/communication cable
k	Terrain exposure coefficient	0.98/1.05	Pole/wires
h	Distance from ground line to centroid (m)	5.4/11.5/10.2/8.8	Pole/conductor/neutral wire/communication cable

(ASCE 2008; NESC 2002). Factors Q, I_{FW}, and C_f are the same for all of the components of the distribution pole systems, but G_{RF}, A, k, and h vary between the poles and the wires (i.e., conductor, neutral wire, and communication cable) (Malmedal and Sen 2003). Factors Q, I_{FW}, G_{RF}, k, and C_f are found in design standards (ASCE 2008), while factors A and h are determined based on the geometric properties of the components of the poles.

The projected wind area (A) is defined as the area of each component that is exposed to wind (e.g., the wind area for the wires is determined by multiplying the diameter of the wires with the span of the wires). When designing the distribution poles, the projected wind area of the poles is assumed to be 2.8 m^2 initially; however, this value will change on the basis of the required circumference of the distribution poles, which will vary between methods. Current design practices assume distribution poles are designed to withstand a wind speed of 40 m/s (Malmedal and Sen 2003). The NESC design procedure for extreme wind conditions for distribution poles (i.e., Grade C construction) involves designing structures

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to withstand extreme winds of 38 m/s within loading regions, such as Miami-Dade County, Florida (Brown 2008; Quanta 2009); therefore, for simplicity, the design wind speed for both design methods is assumed to be 40 m/s to ensure that both methods are consistent.

The amp is determined for both methods by using the Gere-Carter method. The amp is estimated to be 1.133 for the distribution poles designed according to the ASCE method and 1.118 for poles designed according to the NESC method. The variation is attributed to the fact that when the amp is estimated using the Gere-Carter method, a dead load factor is implemented within the calculations. The dead load for the ASCE method is 1.1 (ASCE 2006), whereas the NESC (2002) recommends a dead load factor of 1.0, i.e., amp is dependent on which method is used.

Table 3 shows the required ground line circumference (C_g) determined from Eq. (4), and the corresponding class of the poles is found in design standards (ANSI 2002). Table 3 also presents the design resistance (R_n) that is found in design standards given the pole class (ANSI 2002). ANSI (2002) defines the minimum ground line circumferences for a range of pole classes, i.e., Classes H6-H1 and Classes 1-6, in which Class H6 is the largest with a minimum circumference of 1,500 mm, and Class 6 is the smallest with a minimum circumference of 762 mm (on the basis of a 14-m pole). In Table 3, Case 2 yields the largest pole, and this is attributed to the fact that the ratio γ/φ is 2.58, whereas for Case 3, the ratio is 2.06, and for Case 4, the ratio is 1.29. In comparison, for the Case 1, the γ/φ ratio is 1.27, which is similar to Case 4.

No Degradation of Poles: Fragility Analysis

Fragility curves are developed to examine the differences in vulnerability between the two design methods. The design resistance (R_n) listed in Table 3 is modified with the model error to estimate the actual resistance (R) [Eq. (14)]. The mean model error (ME) is assumed to be 1.12, with $\mathrm{COV_{ME}} = 14\%$, for the design resistance (Vanderbilt et al. 1982). As a starting point, the ME is assumed to be normally distributed. Table 4 shows the actual resistance for the four design cases.

The design strength for the four pole classes has a COV of 20% (ANSI 2002). The ASCE-111 (2006) recommends that the strength of the pole be described by three distributions: normal, lognormal, and Weibull. For this analysis, the strength of the pole will be assumed to have a lognormal distribution, as suggested in Bingel et al. (2003), Li et al. (2006), and Wolfe et al. (2001).

The wind load for the poles is determined on the basis of the projected wind area of the poles (which varies because of different ground line circumferences for the design methods) and the uncertain variables in Eqs. (2) and (3). Air density factor (Q) and importance factor (I_{FW}) are constants; their values are listed in Table 2. Table 5 summarizes the statistical parameters for the random variables in Eqs. (2) and (3).

To account for the uncertainty within amp, it is assumed to be normally distributed with a mean value, which is assumed to be the value estimated with the Gere-Carter method. The COV is determined by running a MCS, assuming that the modulus of elasticity (MOE) has a normal distribution with a COV of 20% (ANSI 2002), and the geometry of the poles has a normal distribution with a COV of 6% (Wolfe and Moody 1997). The mean value of amp is estimated to be 1.118, with a COV of 8% for the NESC method and to be 1.133, with a COV of 11% for the ASCE method.

Hurricane fragility curves show the probability of failure of a distribution pole conditioned at a specific wind speed, which increases monotonically. The probability of failure for the fragility curves is estimated by a MCS, as stated previously, which involves counting the number of times the load (S) exceeds the resistance (R). The load (S) and resistance (R) are randomly generated with the wind speed as deterministic within the selected range (i.e., from 0–120 m/s), by using the previously mentioned parameters. Fragility curves are developed from the MCS for the four design cases and are shown in Fig. 6.

The fragility curve for ASCE Design Case 1 and NECS Design Case 4 reach 100% probability of failure at about 83 m/s, whereas NESC Design Cases 2 and 3 reach 100% probability of failure at 112 and 103 m/s, respectively. Cases 1 and 4 reach 100% probability of failure at wind speeds that are approximately 35 and 25% lower than for Cases 2 and 3, respectively. It is also interesting to examine at what wind speed the distribution poles reach a 50% probability of failure. Case 1 has a 50% probability of failure at approximately 51 m/s, Case 2 at 69 m/s, Case 3 at 63 m/s, and Case 4 at 52 m/s. From the comparison, it can be seen that Case 1 has a 50% probability of failure at wind speeds that are about 27% lower than for Case 2.

These differences are attributed to the fact that Cases 2 and 3 are more conservative than Cases 1 and 4, because they implement a significantly higher ratio of load factor and strength factor (γ/φ). Cases 2 and 3 required larger ground line circumferences than Cases 1 and 4, which resulted in larger poles for Cases 2 and 3 that are able to withstand higher loads. Furthermore, the ratio of the load factor to the strength factor (γ/φ) implemented for Cases 1 and 4 are similar, or 1.27 and 1.29, which yielded the same class of pole; however, the fragility curves are not exactly the same

Table 3. Required Circumference at Ground Line and Pole Class for Design Cases

Design case	Method	C_g	Class	Design resistance (R_n)
1	ASCE	775 mm	5	99,000 N-m
2	NESC	940 mm	2	190,900 N-m
3	NESC	856 mm	3	151,600 N-m
4	NESC	777 mm	5	99,000 N-m

Table 4. Actual Resistance for Design Cases

Design case	Method	Class	Actual resistance (R)
1	ASCE	5	110,900 N-m
2	NESC	2	213,800 N-m
3	NESC	3	169,700 N-m
4	NESC	5	110,900 N-m

Table 5. Statistics for Wind Load

Random variable	Component	Distribution	Mean value	COV (%)	Source
A (m^2)	Pole	Normal	Varies	6	(Wolfe and Moody 1997)
	Conductor	Normal	1.50	6	
	Neutral wire	Normal	1.00	6	
	Communication cable	Normal	3.20	6	
k	Pole	Normal	0.98	6	(ASCE 2006)
	Wires	Normal	1.05	6	
h (m)	Pole	Normal	5.40	3	Assumed
	Conductor	Normal	11.50	3	
	Neutral wire	Normal	10.20	3	
	Communication cable	Normal	8.80	3	
G_{RF}	Pole	Normal	0.96	11	(Ellingwood and Tekie 1999)
	Wires	Normal	0.81	11	
C_f	Pole	Normal	1	12	(Ellingwood and Tekie 1999)
	Wires	Normal	1	12	

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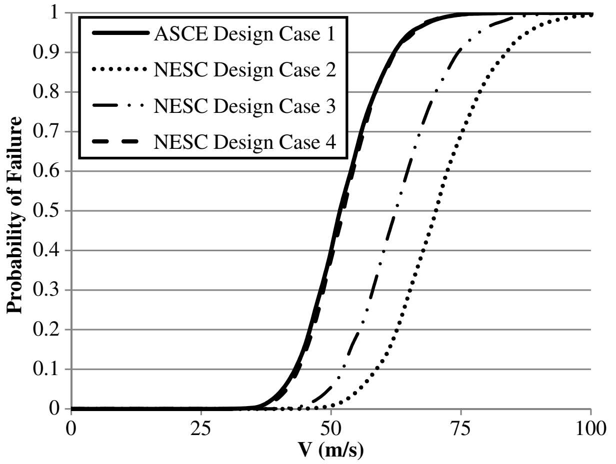

Fig. 6. Fragility curve (ASCE versus NESC)

Table 6. Lognormal Parameters for Hurricane Fragility

Design case	$\ln(m_R)$	ξ_R
1	4.74	0.146
2	5.05	0.135
3	4.94	0.140
4	4.76	0.137

for Case 1 and 4 because the methods resulted in different amp factors.

The statistical distribution of the structural fragility of timber poles needs to be identified to estimate the annual P_f using Eq. (15). Kolmogorov-Smirnov test goodness of fit test with a significance level of 5% is used to find that the lognormal distribution is appropriate for all fragility curves. Table 6 shows the lognormal parameters, ln m_R and ξ_R in Eq. (14) for each design case, for new poles.

No Degradation of Poles: Annual Probability of Failure

To estimate the annual P_f in Eq. (15), the Weibull distribution parameters for wind speed in Miami-Dade County are estimated to be $u(1) = 27.36$ and $\alpha(1) = 1.77$ using wind contour maps (Vickery et al. 2009). The annual P_f is calculated as 0.054, 0.009, 0.020, and 0.052 for the four design cases, respectively, at year 1. Quanta (2009) estimates that the timber pole failure rate of distribution poles in Southern Florida is approximately 0.045, considering an extreme wind criteria of 65 m/s (FP&LC 2006). The most common class of timber poles is Class 4 (Foedingur et al. 2002); therefore Case 2 and 3, which resulted in larger classes of poles, have an annual P_f that is significantly lower than the Quanta estimates, whereas Case 1and 4 (which resulted in a smaller class of pole) yield annual P_f that are higher than but albeit closer to the Quanta estimates.

Effect of Degradation on P_f

The average age of Southern Pine distribution poles in the United States is approximately 32 years (Stewart and Goodman 1990). To investigate a wide range of pole ages, fragility curves are developed for poles that are 0, 10, 30, and 50 years old. Eqs. (5)–(10) are used to estimate the strength for all design cases.

The resistance decreases with age because the degradation results in a decreased diameter of the pole caused by decay By using Eq. (10), the decay depth is found to be 0, 3, 15, and 26 mm after 0, 10, 30, and 50 years, respectively. Recall that the four design cases resulted in poles of various sizes; therefore, the decay depth will affect the bending stress of the poles differently depending on the design case (i.e., the strength of the smaller poles will be affected more by the decay depth than the strength of the larger poles).

Table 7 shows the actual resistance estimated with Eqs. (5)–(10) for each design case for poles that are 0, 10, 30, and 50 years old. The table also includes the COV for the resistance, estimated with Eq. (12). For each design case, the table shows that as the poles are assumed to age, the uncertainty within the estimated resistance increases, which translates to the increasing COV with age. It is also evident that the strength of the smallest poles (Cases 1 and 4) experience the greatest increase in uncertainty as the poles age. This is because the decay depth is the same despite the initial size of the pole, resulting in the largest proportional change in diameter for the smallest distribution poles.

Fig. 7 shows the fragility curves for distribution poles designed by using the four design cases considering the effects of aging. For example, distribution poles designed with Case 1 [Fig. 7(a)] reach 50% probability of failure at wind speeds of 51, 49, 43, and 34 m/s for poles that are 0, 10, 30, and 50 years old, respectively. The 30-year-old pole and the 50-year-old pole designed according to Case 1 have a 50% probability of failure at wind speeds that are approximately 15 and 32% lower, respectively, than for a new pole. On the other hand, distribution poles designed according to Case 2 [Fig. 7(b)] have a 50% probability of failure at wind speeds of 70, 68, 60, and 50 for 0, 10, 30 and 50-year-old distribution poles, respectively. The 30-year-old pole and the 50-year-old pole designed according to Case 2 have a 50% probability of failure at wind speeds that are approximately 15 and 28% lower, respectively, than the wind speed at which new poles have a 50% probability of failure.

From Fig. 7, it is evident that poles of various ages have a 100% probability of failure for lower wind speeds when using Cases 1 and 4. This is consistent with Fig. 6, in which it is apparent that Cases 1 and 4 resulted in poles that reach 100% probability of failure at a lower wind speed than the poles designed based on Cases 2 and 3. This is attributed to the fact that the ratio of load factor and strength factor (γ/φ) for Cases 2 and 3 is higher than the ratios implemented to design the pole for Cases 1 and 4. In other words, poles designed according to Cases 2 and 3 are significantly larger than for the other design cases. Moreover, the effects of degradation become apparent after more than 10 years. Hence, as the poles age, they become more vulnerable; therefore, the differences between the design cases become smaller.

Table 7. Mean and COV of Resistance $R(t)$ for Various Ages

	0 years		10 years		30 years		50 years	
Design case	Mean resistance (N-m)	COV (%)	Mean resistance (N-m)	COV (%)	Mean resistance (N-m)	COV (%)	Mean resistance (N-m)	COV (%)
1 (ASCE)	99,000	20.0	91,900	21.1	68,900	40.0	50,600	70.6
2 (NESC)	190,900	20.0	180,000	20.7	143,200	33.7	112,700	55.5
3 (NESC)	151,600	20.0	142,200	20.8	111,000	35.7	85,400	60.2
4 (NESC)	99,000	20.0	91,900	21.1	68,900	40.0	50,600	70.6

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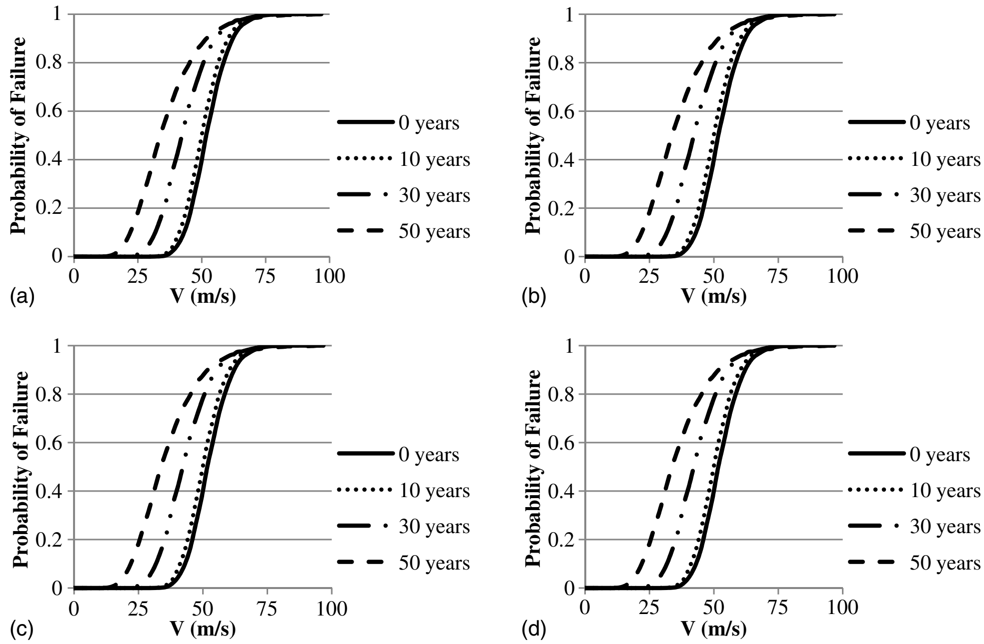

Fig. 7. Effects of degradation on the probability of failure: (a) ASCE Case 1; (b) NESC Case 2; (c) NESC Case 3; (d) NESC Case 4

The age of the poles appears, from a comparison of Fig. 7, to affect the probability of failure for the poles designed according to Cases 1 and 4 more than for the Cases 2 and 3. This is caused by the fact that the COV for the bending strength of the poles is higher for Cases 1 and 4 than for the Cases 2 and 3. For example, for a pole that is 50 years old, the COV of the bending strength for Cases 1 and 4 is 70.6%, but for Case 2, the COV of the bending strength is 55.5%, and for Case 3, the COV is 60.2%. The higher COV translates to more uncertainty, which, in turn, results in the probability of failure higher for Cases 1 and 4 than for Cases 2 and 3.

Impact of Climate Change on Annual P_f

The annual P_f will be estimated for all four design cases, considering new distribution poles for 100 years of service. However, the time frame is irrelevant for the point-in-time analysis given by Eq. (15) because this is not path dependent (i.e., the annual P_f will depend on the change in wind speed not the number of years investigated).

To estimate the annual P_f caused by the potential impact of climate change, the mean maximum annual wind speed is assumed to change linearly over the selected time period. The mean maximum annual wind speed is 23, 24, 26, and 30 m/s, assuming changes in wind speed of −5, 0, 10, and 25%, respectively, over 100 years. The COV for the mean maximum annual wind speed at year 1 is estimated to be 0.584, which is assumed to be constant for all years within the time frame (Bjarnadottir et al. 2011). Using the mean maximum annual wind speed and the COV, the Weibull parameters can be determined for each change in wind speed in each year during the 100-year time frame. Subsequently, the annual P_f of distribution poles can be estimated by using Eq. (15). This analysis assesses the impacts of climate change by assuming that the annual maximum hurricane wind speeds are nonstationary (i.e., changing) within Eq. (15); although, it is possible to assess the impacts of climate change by assuming a changing wind speed within the fragility. For example, the annual maximum wind speed (V) may increase from 24.3 to 26.7 m/s, which would result in an increase in annual P_f from the fragility analysis.

Table 8. Annual P_f for Design Cases (New Poles)

Change in wind speed (%)	Design Case 1	Design Case 2	Design Case 3	Design Case 4
−5	0.044	0.006	0.014	0.040
0	0.054	0.009	0.020	0.052
10	0.085	0.017	0.035	0.080
25	0.136	0.036	0.065	0.130

Table 8 shows the annual P_f after 100 years for the four design cases (not considering deterioration). From the table, it is evident that distribution poles designed with Case 2 are the least susceptible to wind damage and this attributed to the fact that this is the largest pole, able to withstand the largest wind load. In comparison, Cases 1 and 4 produced distribution poles that are the most vulnerable to wind speed. An increase in wind speed of 10% over 100 years could result in annual P_f of about 0.080 for Cases 1 and 4, whereas the annual P_f for Case 2 is estimated at 0.017 for the same increase in wind speed.

The larger poles, designed with Cases 2 and 3, experience the greatest change in annual P_f because the wind speed is assumed to change over the selected time frame. This is because the annual P_f was very low for these design cases initially; therefore, the changes in wind speed resulted in proportionally higher changes in the annual P_f than for Cases 1 and 4, which had considerably higher annual P_f initially. For example, for Case 2, an increase in wind speed of 10% could result in an increase in the annual P_f of 90% from the case of no climate change, whereas the same increase in wind speed could result in an increase in the annual P_f of approximately 50% for Cases 1 and 4.

Effects of Degradation and Climate Change on Annual P_f

The effects of degradation on the annual P_f considering the impacts of climate change are explored by integrating the reduced strength found in Table 7 into the annual P_f estimations. The annual P_f of

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distribution poles for the four design cases are estimated assuming poles that are 0, 10, 30, and 50 years old. Fig. 8 assumes that a percentage increase in wind speed occurs over 100 years and also includes the effects of degradation. As seen with the fragility curves, the distribution poles are more vulnerable to changes in wind speed as the age of the poles increases. As stated previously, the average age of distribution poles in the United States is about 32 years (Stewart and Goodman 1990). If wind speeds are assumed to increase by 10% in 100 years, the annual P_f for the 30-year-old pole could increase by 30, 60, 50, and 30%, for Cases 1, 2, 3, and 4, respectively, from the no climate change scenario (i.e., no change in wind speed). Cases 2 and 3 experience the highest change in annual damage probability for all scenarios of climate change, which is consistent with the previous discussion. Fig. 8 presents the annual P_f for poles of 0, 10, 30, and 50 years for all design cases considering changes in wind speed from -5 to 25% in 100 years.

Effects of Degradation and Climate Change on Service Proven Distribution Poles

It is important to investigate the service proven reliability of distribution poles by considering the effects load history may have on their reliability. An assessment of this nature takes into account the fact that the number of years that distribution poles have survived gives an indication as to the minimum structural capacity of the distribution poles. This analysis considers the updated annual P_f that a distribution pole will fail in the subsequent year given that the distribution pole has survived T years of service (Stewart 1997; Stewart and Val 1999), and the updated annual P_f is estimated at each subsequent year of the assessment by conducting a MCS. The MCS estimates the number of distribution poles that fail in a specific year given the load (S) and the resistance (R) of the poles, and then, the total number of remaining poles is updated for the next year by subtracting the failed poles for the previous year from the total number of poles. The updated annual P_f is then estimated at each subsequent year on the basis of the number of failed poles and total number of poles for that specific year. This service proven reliability analysis was conducted for distribution poles, considering climate change scenarios of 0 and 10% increases in wind speed over 100 years. For the sake of brevity, one case is presented in this paper, ASCE Design Case 1.

Fig. 9 shows the updated annual P_f for distribution poles, considering both degradation and no degradation, for two scenarios of climate change. The updated annual P_f, shown on Fig. 9 is a conditional probability that estimates the likelihood that the distribution poles will fail in year $T+1$ given that the distribution poles have survived T years

$$p_f(1|T) = \frac{p_f(T+1) - p_f(T)}{1 - p_f(T)} \tag{17}$$

where $p_f(t)$ = the cumulative probability of failure up to time t (Stewart and Val 1999).

Therefore, the updated annual P_f is quite similar for the first years of the assessment. From the Fig. 9, the updated annual P_f increases significantly for both climate change scenarios when degradation is accounted for. On the other hand, when degradation is not considered, the updated annual P_f decreases for 0% increase in wind speed, and this is because, for service proven structures, the reliability should increase if no degradation is accounted for (Stewart 1997; Stewart and Val 1999). However, when wind speeds are assumed to increase by 10%, the decrease in the updated annual P_f seen initially for 0% increase in wind speed is counteracted, and the updated annual P_f remains relatively constant for the 100 year service life of the distribution pole. In all cases, the updated annual P_f shown in Fig. 9 is less than the annual P_f shown in Fig. 8; this shows the advantage of updating reliabilities with new information. For example, from Fig. 8, a 50-year-old pole may have an annual P_f of approximately 0.30 if an increase in wind speed of 10% is assumed; in comparison, from Fig. 9(b), the updated annual P_f for a pole that has survived 49 years is approximately 0.25 for an increase in wind speed of 10%.

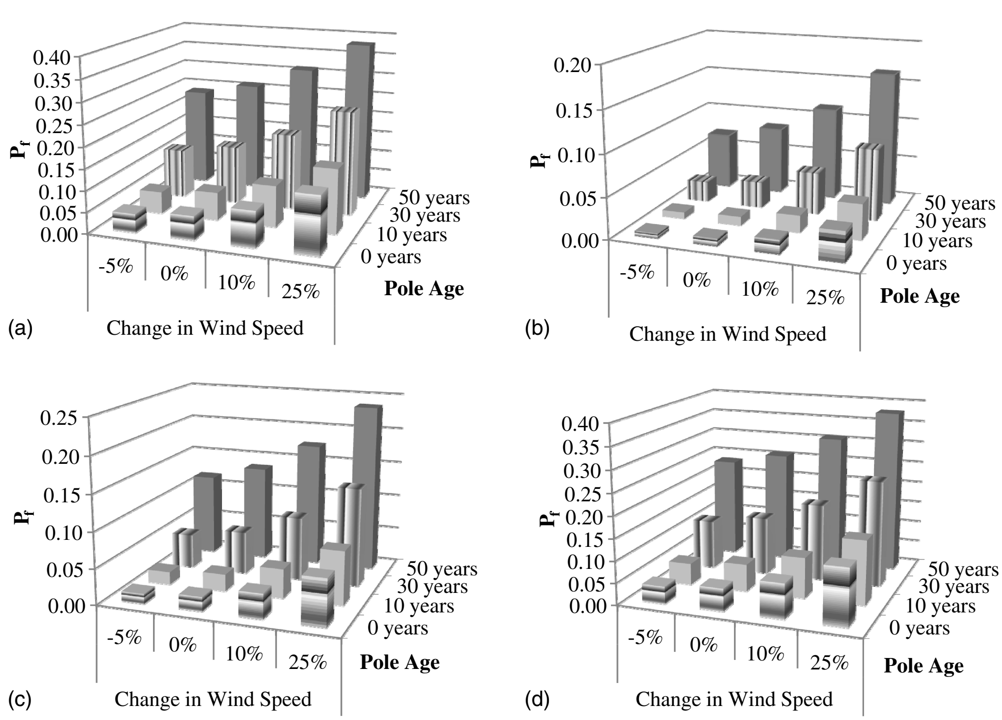

Fig. 8. Effects of degradation on annual P_f for various scenarios of climate change after 100 years: (a) ASCE Case 1; (b) NESC Case 2; (c) NESC Case 3; (d) NESC Case 4

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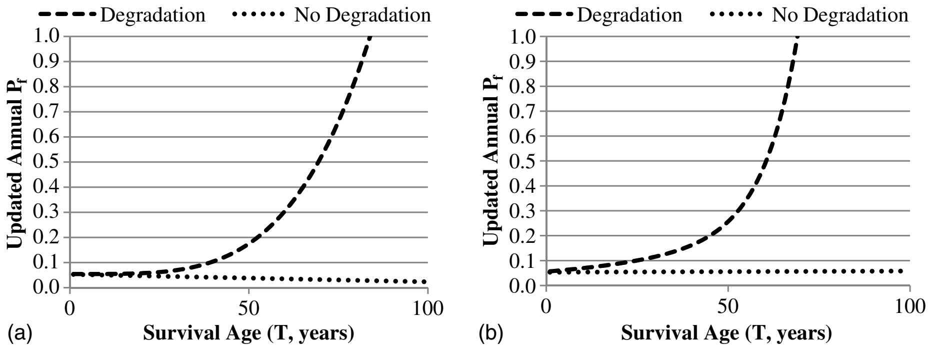

Fig. 9. Updated annual P_f: (a) no change in wind speed, and (b) 10% increase in wind speed in 100 years

For comparison purposes, recall that the annual P_f for the ASCE Design Case 1, was estimated at 0.054 for any year under current climate conditions (i.e., no climate change and no deterioration). However, the updated annual P_f at year 20 (i.e., the probability that a distribution pole will fail in year 20, given that it has survived the previous 19 years of service) is 0.047 if there is no degradation and 0.057 if there is degradation for no change in wind speed. If wind speeds increase by 10% over 100 years, the updated annual P_f at year 20 is 0.054 and 0.086 for no degradation and degradation, respectively. For a survival age of 50 years, the updated annual P_f is 0.038 for no degradation and 0.178 for degradation, for 0% increase in wind speed, and for 10% increase in wind speed, the updated annual P_f is 0.056 when degradation is not accounted for and 0.244 when degradation is accounted for. These are increases in the updated annual P_f of up to 50% if there is degradation, under an assumed increase in wind speed of 10% over 100 years. From this analysis, climate change could have a significant impact on the service proven reliability of distribution poles if there is deterioration, but if there is no deterioration, then a 10% increase in wind speed over 100 years will not increase updated annual probabilities of pole failure.

Conclusions

This paper proposes a comprehensive framework to analyze the hurricane risk of distribution power (utility) poles and assesses the potential impacts of climate change on the annual failure probability of the distribution poles. Damage risks of distribution poles designed by both NESC and ASCE methods were evaluated by using a fragility analysis. Climate change was found to have a significant impact on the annual failure probability of the distribution poles when they are subjected to changing patterns of hurricane hazard.

Four design cases were explored for the design of the distribution poles, and each design case included varying strength and load factors that are based on various practices within the design of existing and new distribution poles. Distribution poles designed with the new ASCE method were found to have the highest annual failure probability for all scenarios of climate change. The larger poles, often poles designed to NESC, experience the greatest percentage change in annual P_f from the case of no climate change (i.e., no change in wind speed), which is attributed to the fact the annual P_f was very low for these poles, initially. Therefore, any changes in wind speed resulted in proportionally higher changes in the annual P_f than for the other two design cases. An increase in wind speed of 10% over 100 years could increase annual failure probabilities by up to 90%, from the case of no climate change and assuming no deterioration of the pole.

Degradation was also found to be an important factor in damage probability. For a pole that is 30 years old, an increase in wind speed of 10% over 100 years could result in an increase in annual damage probability of 30–60% from the no climate change scenario. These increases in failure rates are not as dramatic as for newly constructed poles, but an increase in failure probability for the 30-year-old poles of approximately 30% or more, over a 100-year time frame, could indicate significant increases in replacement costs. Furthermore, the service proven reliability of distribution poles was investigated for distribution poles. It was found that the updated annual P_f could increase by up to 50% under an assumed increase in wind speed of 10% over 100 years, with deterioration, which further indicates increases in replacement costs. Therefore, if this or other climate change scenarios are believed to be possible, it is necessary to assess the economic viability of adaptation strategies to replace or strengthen distribution poles in hurricane-prone zones.

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References

American National Standards Institute (ANSI). (2002). "Wood poles specifications and dimensions." *ANSI 05.1.2008*, Washington, DC.

American Society of Civil Engineers (ASCE). (2006). "Reliability-based design of utility pole structures (No. 111)." *ASCE-111*, Reston, VA.

American Society of Civil Engineers (ASCE). (2008). "Substation structure design guide (No. 113)." *ASCE-113*, Reston, VA.

Australian Greenhouse Office (AGO). (2007). "An assessment of the need to adopt buildings for the unavoidable consequences of climate change." *Final Rep.*, Commonwealth of Australia, Canberra, Australia.

Baraneedaran, S., Gad, E. F., Flatley, I., Kamiran, A., and Wilson, J. L. (2009). "Review of in-service assessment of timber poles." *Proc., Australian Earthquake Engineering Society*, Newcastle, Australia.

Bhuyan, G., and Li, H. (2006). "Achieved reliability of the North American design approaches for transmission overhead structures." *Probabilistic methods applied to power systems*, IEEE, New York.

Bingel, N., et al. (2003). "Panel session: Structural reliability-based design of utility poles and the national electrical safety code." *Transmission and Distribution Conf. and Exposition*, IEEE, New York, 1088–1093.

Bjarnadottir, S., Li, Y., and Stewart, M. G. (2011). "A probabilistic-based framework for impact and adaptation assessment of climate change on hurricane damage risks and costs." *Struct. Safe.*, 33(3), 173–185.

Broccoli, A. J., and Manabe, S. (1990). "Can existing climate models be used to study anthropogenic changes in tropical cyclone climate?" *Geophys. Res. Lett.*, 17(11), 1917–1920.

Brown, R. E. (2008). *Electric power distribution reliability*, CRC Press, Boca Raton, FL.

Chang, C.-H. (2010). "Preparedness and storm hazards in a global warming world: Lessons from Southeast Asia." Natural hazards, ⟨http://services.lib.mtu.edu:2127/content/w2p683564371338h/fulltext.pdf⟩ (Feb. 10, 2012).

Climate Change Science Program (CCSP). (2008). *Impacts of climate change and variability on transportation systems and infrastructure: Gulf Coast study, Phase I. A report by the U.S. climate change science program and the subcommittee on global change research*, M. J. Savonis, V. R. Burkett, and J. R. Potter, eds., U.S. DOT, Washington, DC.

Crosset, K. M., Culliton, T. J., Wiley, P. C., and Goodspeed, T. R. (2004). *Population trends along the coastal United States: 1980–2008*, National Oceanic and Atmospheric Administration, Silver Spring, MD.

Dagher, H. J. (2001). "Reliability of poles in NESC grade C construction." *Rural Electric Power Conf.*, Institute of Electrical and Electronics Engineers (IEEE), New York.

Ellingwood, B. R., and Tekie, P. B. (1999). "Wind load statistics for probability-based structural design." *J. Struct. Eng.*, 125(4), 453–463.

Elsner, J. B., Kossin, J. P., and Jagger, T. H. (2008). "The increasing intensity of the strongest tropical cyclones." *Nature*, 455(7209), 92–95.

Emanuel, K. (2005). "Increasing destructiveness of tropical cyclones over the past 30 years." *Nature*, 436(7051), 686–688.

Florida Power and Light Company (FP&LC). (2006). "Technical report: Post Hurricane Wilma engineering analysis." KEMA, Inc., Arnhem, Netherlands.

Foedinger, R., Boozer, J. F., Bronstad, M. E., and Davidson, J. W. (2002). "Development of an energy-absorbing composite utility pole." *Transportation Research Record 1851*, Transportation Research Board, Washington, DC.

Gere, J., and Carter, W. (1962). "Critical buckling loads for tapered columns." *J. Struct. Div.*, 88(1), 1–12.

Goldenberg, S. B., Landsea, C. W., Mestas-Nuñez, A. M., and Gray, W. M. (2001). "The recent increase in Atlantic hurricane activity: Causes and implications." *Science*, 293(5529), 474–479.

Guikema, S. D., Quiring, S. M., and Han, S.-R. (2010). "Prestorms estimation of hurricane damage to electric power distribution systems." *Nat. Hazards*, 30(12), 1744–1752.

Gustavsen, B., and Rolfseng, L. (2000). "Simulation of wood pole replacement rate and its application to life cycle economy studies." *IEEE Trans. Power Deliver.*, 15(1), 300–306.

Haldar, A., and Tucker, K. (2006). "Condition based management of wood pole transmission lines using structural reliability analysis." *ASCE Conf. Proc.*, 218(28), 304–316.

Johnson, B. (2005). "After the disaster: Utility restoration cost recovery." Edison Electricity Institute, Washington, DC.

Kwasinski, A., Weaver, W. W., Chapman, P. L., and Krein, P. T. (2009). "Telecommunications power plant damage assessment for Hurricane Katrina—Site survey and follow-up results." *IEEE Syst. J.*, 3(3), 277–287.

Larsen, T., Porter, K., Zadeh, M., Van Anne, C., and Scawthorn, C. (1996). "Impact of Hurricane Andrew on performance, interaction, and recovery of lifelines." EQE International, San Francisco, CA.

Leicester, R. H., Wang, C. H., Minh, M. N., Thornton, J. D., Johnson, G. C., and Gardner, D. (2003). "An engineering model for the decay of timber in ground contact." *IRG/WP/03, 34th Annual Meeting*, International Research Group on Wood Preservation, Brisbane, Australia.

Li, H., Zhang, J., and Bhuyan, G. (2006). "Reliability assessment of electrical overhead distribution wood poles." *Probability Methods Applied to Power Systems, Int. Conf.*, Institute of Electrical and Electronics Engineers (IEEE), New York.

Li, Y., and Ellingwood, B. R. (2006). "Hurricane damage to residential construction in the US: Importance of uncertainty modeling in risk assessment." *Eng. Struct.*, 28(7), 1009–1017.

Louisiana Public Service Commission (LPSC). (2005). "Emergency operations center, daily service outage reports-September 24, 2005 through October 17, 2005." Excel spreadsheets for each day summarizing electric, natural gas, and phone outages, created December 9, 2005.

Lupoi, G., Franchin, P., Lupoi, A., and Pinto, P. E. (2006). "Seismic fragility analysis of structural systems." *J. Eng. Mech.*, 132(4), 385–396.

Malmedal, K., and Sen, P. K. (2003). "Structural loading calculations of wood transmission structures." *Rural Electric Power Conf.*, Institute of Electrical and Electronics Engineers (IEEE), New York.

Morrell, J. (2005). "Estimated service life of wood poles." Technical Bulletin for the North American Wood Pole Council (NAWPC), Vancouver, WA.

National Electrical Safety Code (NESC). (2002). *IEEE Standard*, IEEE, Piscataway, New Jersey.

Peel, M. C., Finlayson, B. L., and McMahon, T. A. (2007). "Updated world map of the Köppen-Geiger climate classification." *Hydrol. Earth Syst. Sci.*, 11, 1633–1644.

Peters, G., DiGioia, A. M. J., Hendrickson, C., and Apt, J. (2006). "Transmission line reliability: Climate change and extreme weather." *Electrical Transmission Line and Substation Structures, Proc., Electrical Transmission Conf.*, American Society of Civil Engineers (ASCE), Reston, VA, 12–26.

Public Utility Commission of the State of Texas (PUCT). (2006). "PUB investigation of methods to improve electric and telecommunications infrastructure to minimize long term outages and restoration costs associated with Gulf Coast hurricanes." *Final Rep., Project No. 32812*, Public Utility Commission of Texas, Austin, TX.

Quanta Technology. (2009). "Cost-benefit analysis of the deployment of utility infrastructure upgrades and storm hardening programs: Final report." Public Utility Commission of Texas, Austin, TX.

Reed, D. A. (2008). "Electric utility distribution analysis for extreme winds." *J. Wind Eng. Ind. Aerod.*, 96(1), 123–140.

Reed, D. A., Powell, M. D., and Westerman, J. M. (2010). "Energy infrastructure damage analysis for Hurricane Rita." *Nat. Hazards Rev.*, 11(3), 102–109.

Stainforth, D. A. (2005). "Uncertainty in predictions of the climate response to rising levels of greenhouse gases." *Nature*, 433(7024), 403–406.

Standards Australia. (2005). "Timber-natural durability ratings, Australia." *AS 5604*, Standards Australia, Sydney, Australia.

Stewart, A. H., and Goodman, J. R. (1990). "Life cycle economics of wood pole utility structures." *IEEE Trans. Power Deliver.*, 5(2), 1040–1046.

Stewart, M. G. (1997). "Time-dependent reliability of existing RC structures." *J. Struct. Eng.*, 123(7), 896–903.

Stewart, M. G., and Val, D. V. (1999). "Role of load history in reliability-based decision analysis of aging bridges." *J. Struct. Eng.*, 125(7), 776–783.

Utility Solid Waste Activities Group (USWAG). (2005). *Notice of availability of the preliminary risk assessment for wood preservatives containing pentachlorophenol reregistration eligibility decision*, USWAG, Washington, DC, 1–13.

Vanderbilt, M. D., Criswell, M. E., Folse, M. D., and Landers, P. G. (1982). "Probability-based design of wood transmission line structures electric power research institute project RP-1352-1: A status report." *IEEE Trans. Power Ap. Syst.*, PAS-101(8), 2451–2459.

Vickery, P. J., Wadhera, D., Twisdale, L. A., Jr., and Lavelle, F. M. (2009). "U.S. hurricane wind speed risk and uncertainty." *J. Struct. Eng.*, 135(3), 301–320.

Walsh, K., et al. (2002). "Climate change in Queensland under enhanced greenhouse conditions: Final rep., 1997-2002." A.R. Australian Commonwealth Scientific and Research Organization (CSIRO), Australia.

J. Infrastruct. Syst., 2013, 19(1): 12-24

Wang, C.-H., Leicester, R. H., and Nguyen, M. (2008a). "Decay in ground." *Forest and wood products Australia*, Australian Commonwealth Scientific and Research Organization (CSIRO), Australia.

Wang, C.-H., Leicester, R. H., and Nguyen, M. (2008b). "Probabilistic procedure for design of untreated timber poles in-ground under attack of decay fungi." *Reliab. Eng. Syst. Safe.*, 93(3), 476–481.

Webersik, C., Esteban, M., and Shibayama, T. (2010). "The economic impact of future increase in tropical cyclones in Japan." *Nat. Hazards*, 55(2), 233–250.

White House. (2009). "Global climate change impacts in the United States." U.S. Global Change Research Program, Washington, DC, ⟨http://downloads.globalchange.gov/usimpacts/pdfs/climate-impacts-report.pdf⟩ (Apr. 2, 2010).

Winkler, J., Dueñas-Osorio, L., Stein, R., and Subramanian, D. (2010). "Performance assessment of topologically diverse power systems subjected to hurricane events." *Reliab. Eng. Syst. Safe.*, 95(4), 323–336.

Wolfe, R., Bodig, J., and Lebow, P. (2001). "Derivation of nominal strength for wood utility poles." Forest Products Laboratory, U.S. Department of Agriculture, Washington, DC.

Wolfe, R., and Kluge, R. O. (2005). "Designated fiber stress of wood poles." Forest Products Laboratory, U.S. Department of Agriculture, Washington, DC.

Wolfe, R., and Moody, R. (1997). "Standard specifications for wood poles." Forest Products Laboratory. U.S. Department of Agriculture, Washington, DC.

Zhai, X., and Stewart, M. G. (2010). "Structural reliability analysis of reinforced grouted concrete block masonry walls in compression." *Eng. Struct.*, 32(1), 106–114.