Chapter 4

STRENGTH OF SINGLE-POLE UTILITY STRUCTURES

4.1 INTRODUCTION

Single-pole utility structures are generally loaded as cantilever beam-columns. Their load capacity varies with geometry, structural material, manufacturing process, and support conditions. These parameters, often characterized with varying degrees of uncertainty, are used with mechanics-based as well as empirical-based models (that also introduce some uncertainty) to obtain load capacity or resistance estimates. For these reasons, the design procedure described in Chapter 2 incorporates resistance factors (ϕ factors in Eq. 2-1a) to account for the uncertainties inherent in the estimates of pole capacity.

Pole resistance is a random variable that may be characterized using a probability density function (PDF). For the typical range of pole resistance and load coefficients of variation (COVs), a relatively consistent reliability can be achieved across material types by setting nominal resistance values that represent a 5% to 10% lower exclusion limit (LEL) equal to the design load effect corresponding to a predetermined return period (RP) (IEC 2002; Peyrot and Dagher 1984). For example, setting the 5th percentile pole strength, $R_{5\%LEL}$, equal to the 50-year RP load effect Q_{50}, $R_{5\%LEL} = Q_{50}$, yields resulting reliabilities that are relatively insensitive to the respective PDFs and COVs of the load and strength parameters.

In this manual, nominal resistance (R_n) is defined as the strength that will be exceeded by 95% of poles in the target population (see Fig. 1-1 in Chapter 1). In statistical terms, this value is often referred to as the 5th percentile (5% of the population lies below it) or the lower 5% LEL. Because it is not practical to require a precise evaluation of the 5% LEL, statistical methods are commonly used to obtain an estimate that has an associated level of confidence. Confidence refers to the probability that a randomly selected sample will have an LEL greater than or equal to the target value. The lower confidence bound on a LEL is called a lower tolerance limit (LTL). In this manual, we refer primarily to a 5% LTL having a 50% or 75% confidence.

4.2 OBJECTIVE

This section presents three basic methods for deriving and documenting R_n as an LTL value along with the coefficient of variation (COV_R) for single-pole structures. These include the following:

1. An empirical analysis based primarily on tests of full-sized poles.
2. A theoretical analysis of mechanics-based models used in conjunction with Monte Carlo simulation.
3. A default assignment of material distribution parameters.

These three approaches are intended to address the range of complexity and experience associated with conventional as well as potential utility pole structural materials. The empirical approach relies heavily on test data to verify modeling assumptions. This is especially applicable for wood that is a nonuniform orthotropic material. The variable nature of wood leaves the designer with the options of using a minimum clear wood strength and designing to the minimum specification (for example, largest knot, minimum dimension, greatest grain angle), or referencing full-scale tests of a sample of poles that represent the quality range expected when ordering to a minimum specification (e.g., *ANSI O5.1-2002* [ANSI 2002]). If the critical stress in a wood pole occurs predominantly at the groundline and does not involve knots, strength can be predicted using distribution parameters published for clear wood (ASTM 1998). Knots, cross-sectional dimensions, and taper may present some questions about variability; in such cases a full-scale pole test database may provide a better estimate of the R_n for full-sized poles.

The uniform, isotropic nature of materials such as steel and concrete make them more amenable to the use of mechanics-based models. These models may be coupled with Monte Carlo simulation of material properties to predict their 5% LTL strength. In other cases, where model parameters exhibit little or no covariance the analytical models may be used to estimate mean behavior, and independent parameter variances may be summed to provide an estimate of the pole strength variance with no need for computer simulation.

Default assignment is used when there are insufficient data to characterize the pole strength PDF empirically and when demonstrably reliable models have not been developed to provide accurate estimates of structural performance for a particular pole material or configuration. The default method provides a conservative approach to assigning parameters for estimating R_n.

Market forces will likely control the evolution of the reliability-based design (RBD) approach and ultimately bring all pole configurations to a relatively uniform level of reliability. It is the responsibility of the pole suppliers to provide the parameters and supporting documentation for their poles' strengths, but it is the responsibility of the design engineer to select the correct pole for a given load condition. It is the responsibility of the purchasing agency to review and verify that the poles selected by the engineer are provided by the pole supplier. If competing suppliers are unsure of values being used by their competition, it is their responsibility to fully understand the competitive products and their evaluation. It is this system of checks and balances resulting from competition and documented system performance that will force evolution of increasing reliability.

4.3 SCOPE

The following discussion is related to the derivation of resistance PDFs for single-pole structures subjected to transverse wind and ice loading. In this application, the primary structural component (the pole) is considered to behave as a cantilever beam. Its resistance is characterized by a bending-strength PDF, although the actual failure mechanism may be a localized buckling or tensile failure.

Procedures outlined in this section rely on the fundamental assumption that all poles meet or exceed established minimum manufacturing and process quality standards for poles (AISC 1999; ANSI 2002; ANSI/ASCE *Standard 10-90* [ASCE 1992], which was previously *ASCE Manual 72* [ASCE 1990]; PCI 1999]). The organizations that set these standards provide for acceptable tolerance variations in the manufacturing or processing of all of the pole types. A pole's nominal strength shall be determined and reliability-based strength factors shall be calculated against these acceptable variations. This document does not address pole strength reliability associated with initial substandard quality poles or of pole deterioration because of damage in service or use in hostile environments. Pole deterioration, whether cumulative or due to a single event, is highly variable and should be handled as part of regular pole inspection and maintenance schedules.

4.4 CHARACTERIZING POLE STRENGTH

This document does not cover all possible design criteria for single-pole structures. It illustrates general methods to assign nominal design properties for common pole configurations. The general procedures outlined here can be adapted to less-common designs.

4.4.1 Loads

As previously noted, loads considered in this characterization of pole strength are limited to wind and ice. While the magnitude variablility of these loads are different within and between geographic regions, their effect is manifested on single-pole structures predominantly as cantilever-bending moments. Potential failure modes that result from bending vary with pole material type, geometrry, and suupport conditions. All potential failrue modes must be considered in deriving a PDF to characterize pole strength. For wood poles, extreme fiber-bending stress generally controls the pole's bending load capacity. For reinforeced concrete poles, the ultimate strength is often controlled by the compressive strength of the concrete. For tubular steel and fiber-reinforced polymer (FRP) structures, local buckling and bending are generally the governing factors.

4.4.2 Nominal Resistance

To achieve relatively uniform structural reliability across all material types, a uniform definition of characterstic of nomianal strength or resistance R_n is required. Resistance or strength (R_n) defined in terms of a limiting stress should represent a consistent estimate with regard to the strength PDFs for all pole materials. To this end R_n for single-pole structures is herein defined to represent the 5% LEL with a noted level of confidence (= LTL), regardless of material type. The following discussion describes three approaches to characterizing pole strength and identifiying:

1. The nominal resistance R_n corresponding to a designated LTL.
2. The resistance coefficient of variation (COVR).

Ideally, R_n (definded in terms of a limiting stress) should represent the same point estimate with regard to strength PDF for all pole materials. To this end, R_n for sigle-pole structures is herein defined to represent the 5% LEL regardless of material type. The strength factors in Table 2-2 can be used when pole resistance values are stated at other than 5% LEL.

4.4.2.1 Method 1: Empirical Basis. Empirical derivation of pole strength generally involves some combination of full-sized pole tests and machanics-based models. Because it is not economically feasible to test every possible combination of pole size, processing variable, and load configuration, standard tests are used to establish baseline evaluation of pole capacity. This baseline is then adjusted to account for influences specific to a given application.

For example, the wood pole industry has tradionally endorsed a conservative approach to the selection of wood poles by adopting a standard test procedure, American Society for Testing and Materials (ASTM) *Standard D 1036-99* (ASTM 1999b) that uses either a cantilever or a simply supported beam test to evaluate a maximum groundline moment capacity for a green pole (green wood has a moisture content above the fiber saturation point–roughly 35%). At the present time, the best source for information on empirically derived wood pole design values is *ANSI O52-2002,* Annex C (ANSI 2002). This annex includes mean and variance values for strength and modulus of elasticity for commercial pole species as well as adjustments for conditioning, height, and size.

This test imposes boundary conditions that are at least as critical as any imposed on a pole in service. Green values have traditionally been used in the design of heavy timber because drying during service has counteracting effects: wood shrinkage reduces the effective section property while fiber strength and stiffness increase. Pole strength has traditionally been assessed as stress at the maximum moment location (groundline in the standard test), regardless of the actual failure location. Failure above the groundline is normally associated with knots or the reduced section property due to natural taper. A reduction in fiber strength with height may also be an influence for fast-grown poles. Tests conducted to assess the effect of variable material quality along the length of the pole (Bodig et al. 1986) show that when this failure occurred above groundline, the groundline stress was generally within 10% of the stress at the failure location. The pole capacity may therefore still be determined on the basis of groundline stress. The ASTM *Standard D 1036-99* (ASTM 1999b) cantilever test method provides conservative estimates of pole groundline moment (GLM) capacity when the groundline circumference is more than 1$^1/_2$ times the circumference at the centroid of load application.

Similar standard test procedures exist for concrete and FRP poles (e.g., ASTM *Standard C 1089-97* [ASTM 1997] for spun-cast prestressed concrete poles, and ASTM *Standard D 4923-92* [ASTM 1992] and ANSI *Standard C 136.20* [ANSI 1996] for FRP poles). ASCE is planning to publish a standard test procedure for steel poles.

Methods used to assess nominal resistance should take a conservative approach in accounting for processing and common service conditions that might affect pole performance. The design engineer should request information on the assumptions made in the derivation of the nominal values and recommendations for modifying them in certain cases: where a pole will be used under uncommon conditions, such as extremely wet or arid conditions, salt air, high temperature, or alkali soil, or where loads or boundary conditions differ from those assumed in deriving R_n. Adjustments for the derivation of design stress in round timber are presented in ASTM *Standard D 2899-01* (ASTM 2001).

A well-documented database established with strict adherence to standard test procedures can provide long-term benefits for the development and evolution of design standards. Periodic updating provides a record of changing trends in pole production and pole performance sensitivity to influencing variables. Over time, this can lead to refined methods for pole classification, reducing variability, and increasing reliability.

For manufactured poles having uniform strength along their length, it may be advantageous to develop a simple beam-bending test procedure in lieu of the full-scale cantilever pole test used for wood. The point of maximum stress in a uniformly tapered cantilever beam can be derived theoretically. If engineering models are available to accurately predict critical stress location but verification data are required to predict effects of change in geometry with applied stress, the standard test could be conducted to concentrate maximum bending stress at a specific cross section. A simple beam test having a high enough span-to-depth ratio to minimize the significance of shear effects would be less costly than a full-size cantilever test and would yield similar results. Alternative test methods must be universally accepted by users and producers and the appropriate design code authority.

4.4.2.2.2 Probability Density Functions. A critical part of any empirical evaluation is ensuring that the referenced database accurately represents the target population. Test samples must be selected to truly represent pole production and test results should be classified so values are characterized on the basis of probability of occurrence.

For full-sized pole test data to be statistically valid, the test specimens must be representative, in terms of size and quality, of the poles to be used in service applications. This may require samples to be selected over time and to be selected on the basis of population proportions. Methods to ensure the statistical validity of the sampling conducted can be found in *ASTM Standard D 2925-99* (ASTM 1999a) and *Standard D 5457-93* (ASTM 1993).

Using a test sample to project a statistical probability of occurrence of pole strength requires the adoption of a PDF. PDFs are generally classified as either parametric (described by a mathematical function) or nonparametric and are assumed to have a frequency profile rather than a strength profile representative of the parent population of pole strengths.

The nonparametric approach imposes no assumed shape on the PDF; it sorts data by order of magnitude. The sequence number corresponding to a particular datum in the ordered set is referred to as its order statistic; the initial estimate of the probability of getting a value less than or equal to that datum is the order statistic divided by the number of data points in the data set. For example, the lowest value in a sample of 20 (1/20 = 0.05) or the second value in a sample of 40 (2/40 = 0.05) is assumed to represent the 5th-percentile order statistic. In other words, 5% of a sample will have values less than or equal to this value. This value is often referred to as a point estimate of the 5% lower exclusion limit (LEL) of the parent population.

It is generally accepted practice in the wood industry, as in *ASTM Standard D 2915-99* (ASTM 1999a) to select an order statistic that will provide a level of confidence in an estimate of the 5% LEL. For an infinite population, the value at the 5th-percentile order statistic is the 5% LEL. For small samples, there is roughly a 50% chance that the LEL will exceed 5% of the parent population. Statistical tables are available that provide nonparametric order statistics associated with confidence bounds on the estimates of lower exclusion values (FPL 2005). Table 4-1 lists the order statistics associated with the 5% LEL given the 50% minimal confidence value as well as 75% confidence. The higher the level of confidence, the lower the order statistic.

TABLE 4-1. A 5% LTL with 50% or 75% Confidence for a Nonparametric Estimate and for a Normal Distribution[a]

Sample Size	Nonparametric		Normal	
N	I_{50}	I_{75}	K_{50}	K_{75}
5	NA	NA	1.78	2.464
15	NA	NA	1.68	1.99
20	1	–	1.67	1.93
28	–	1	1.66	1.88
40	2	–	1.66	1.83
53	–	2	1.65	1.81
60	3	–	1.65	1.79
70	–	–	1.65	1.78
78	–	3	1.65	1.77
80	4	–	1.65	1.77
90	–	–	1.65	1.76
102	5.1	4	1.65	1.76
125	6.25	5	1.65	1.75
200	10	9	1.65	1.72
∞	15	13	1.65	1.71

[a] For a nonparametric distribution, the order statistic (I = order of magnitude, or rank in an ordered list) is used to identify the 5% LEL (I_{50}) and the 5% LTL (I_{75} = 75% confidence). For a normal distribution, the 5% LTL is derived with 50% (K_{50}) or 75% (K_{75}) confidence standard deviations below the mean.

As specified in *ASTM Standard D 2915-99* (ASTM 1999a), the non-parametric distribution is prefered for structural wood design value derivations when using a small sample (<100) basis. Unlike parametric distributions, this approach requires a minimum sample fo 21 to estmate a 5% LEL. Small samples may provide a resionable estimate of mean trend but they do not provide sufficient information to accurately assess variability or point estimates at low probability levels.

Parametric PDFs are defined by closed-form equations, which can be used to generate an entire population of values that fit a designated distribution shape. Parametirc PDFs therefore provide a means of extrapolating beyond the bounds of a given data set to estimate values with a low probability of occurrence. It is generally considered risky to extrapolate too far beyond the range of the available test data. Sample size (N) should therefore be selected to provide a prescribed level of assurance (x) that $100 \times \eta\%$ of a population will be included between the largest and smallest values. Wilkes (1944) derived a fuchtion to characterize the relationship (Eq. 4-1) between N, x, and η:

$$N\eta^{N-1} - (N-1)\eta^{N} = 1 - \xi. \qquad \text{(Eq. 4-1)}$$

This function indicates taht a sample size of $N = 93$ is required to encompass 95% of a population with assurance, and a sample size of $N = 472$ to encompass 99% of a population with that same level of assurance.

Many forms of parametric PDFs have been formulated. The three that are most commonly referenced to model strength are normal, log-nomal, and Weibull distributions. The PDFs demonstrate a range of control of shape and range of predicted values and imply varying degrees of knowledge about the population being represented.

The normal PDF has the flollowing mathematical form:

$$f_X(x) = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{1}{2}[(x-m)/\sigma]^2} \qquad \text{(Eq. 4-2)}$$

where

m = the mean value of x or first moment of the area under the PDF curve, where $-\infty < x < +\infty$

σ = the standard deviation. It is the square root of the second moment of the area under the PDF about the mean (variance) $\sigma^2 = /(x - m)^2 f_X(x)\ dx$. For siscrete data set, this relationship yields:

$$\sigma = \left[\frac{\sum_{i=1}^{N} x_i^2 - Nm^2}{N-1} \right]^{0.5} \qquad \text{(Eq. 4-3)}$$

in which N is the sample size.

If there are sufficient data to warrant its use, the normal PDF will generally provide conservative estimates of low tail values. Using the normal distribution, R_n may be determined by

$$R_n = R_m - K \cdot \sigma_R \qquad \text{(Eq. 4-4)}$$

where

R_m = the mean strength
σ_R = standard deviation of strength
K = the distance from the mean to the point on the PDF that corresponds to the target lower tolerance limit.

The K values given in Table 4-1 were derived using a noncentral t-distribution inverse approach discussed by Guttman (1970). These values are derived to consider either a 50% or 75% confidence in the 5th percentile of a normal distribution for sample sizes ranging from 5 to 300.

The normal distribution has historically been used for characterizing the strength of wood. It is easy to use, is widely recognized, and generally provides conservative estimates of low tail values that are referenced when deriving design values. When the available data show that strengths are not distributed symmetrically about the mean, other PDFs are referenced to provide a more accurate characterization. A weakness often cited for the normal PDF is that it presents the possibility of having strength values less than zero. This is not a problem when the standard deviation is less than 30% of the mean and the PDF is used only to provide an estimate of a value having greater than a 1% probability of occurrence.

If the PDF is known to be right-skewed (i.e., having values much farther above than below the mean), the normal distribution may be considered to be overly conservative in estimating LTL values. Wood utility poles are often selected from a truncated normal distribution, where the lower tail represents poles that do not meet the *ANSI O5.1-2002* (ANSI 2002) minimum specifications. An alternative to the normal PDF that addresses the issue of right-skewness (and negative values) is the log-normal PDF. This PDF is derived assuming that the logarithms of the test data are normally distributed. In general, however, when the standard deviation is in the range of 20% of the mean, the log-normal distribution will give a 5th percentile point estimate only slightly larger than that obtained assuming a normal PDF. The log-normal PDF is defined as follows.

The log-normal PDF is applicable if the natural logarithms of strength data (x) are normally distributed with a mean λ and standard deviation μ. In this case, the PDF is of the same form as Eq. 4-2, substituting y = 1n (x) for x, λ for m, and μ for λ.

$$f_y(y) = \frac{1}{\mu\sqrt{2\pi}} e^{-\frac{1}{2}\left(\frac{y-\lambda}{\mu}\right)^2} \qquad \text{(Eq. 4-5)}$$

where λ is the first moment of the area under the PDF or the mean of the 1n, (x) and μ^2 is the second moment or variance of the 1n, (x).

Because there are closed-form transformation equations to relate normal and log-normal PDF parameters, there is little need to actually work with the logarithms of data in order to make point estimates using the log-normal distribution. The mean strength and standard deviation conversion from normal to logarithm have the following form:

$$\text{Log-normal variance} = \mu^2 = 1\text{n}\,(\Omega^2 + 1). \qquad \text{(Eq. 4-6)}$$

$$\text{Log-normal mean} = \lambda = 1\text{n}(R_m) - \frac{\mu^2}{2} \qquad \text{(Eq. 4-6)}$$

where

1n ( ) = natural logarithm (base e)
Ω = COV_R = coefficient of variation of the strength test data (σ_R/R_m)
Rm = mean of the strength test data
σ_R = standard deviation of the strength test data.

Equations 4-8 and 4-9 provide a point estimate of the LTL nominal strength R_n for a log-normal distribution using the mean R_m and coefficient of variation Ω of the test data.

$$R_n = k_N x R_m. \qquad \text{(Eq. 4-8)}$$

$$k_N = \frac{1}{\exp(\frac{1}{2}1\text{n}(\Omega^2+1) + K_N\sqrt{1\text{n}(\Omega^2+1)})} \qquad \text{(Eq. 4-9)}$$

Note that K_N and k_N are different variables. The multiplier k_N in Eq. 4-8 converts the mean test data strength to a LTL with a confidence value dependent on K_N. The K_N is the normal distribution tolerance adjustment corresponding to sample size N as listed in Table 4-1.

The Weibull PDF is a versatile alternative that can also be used to represent a distribution of all-positive values. It can be made to fit a wide range of distribution shapes. The Weibull distribution may be characterized using either two or three parameters. The three-parameter function has the following form:

$$f_x(x) = \frac{\omega}{\theta}\left(\frac{x-x_0}{\theta}\right)^{\omega-1} \exp(-(\frac{x-x_0}{\theta})^{\omega}) \quad x \geq x_0 \quad \omega,\theta > 0 \qquad \text{(Eq. 4-10)}$$

where

- ω = slope or shape parameter that reflects the relative scatter in the data; the larger the shape parameter, the lower the spread. A shape parameter of 3.5 is symmetric while a value >3.5 gives a negative or left-skewness, and a value <3.5 provides a positive skewness.
- θ = scale parameter. As the scale parameter increases, the mode (location) where most events occur moves toward the upper end of the distribution.
- x_0 = location parameter. If the location parameter is set equal to zero, Eq. 4-10 reduces to a two-parameter Weibull PDF.

The added versatility of the Weibull PDF also allows a greater chance for misrepresentation. For poles that have a COV in the range of 20%, the two-parameter Weibull distribution is likely to give more conservative estimates of a lower fractile than will a normal distribution. When the COV of a data set has a value less than 30%, a two-parameter Weibull PDF will generally have a shape parameter greater than 3.5, resulting in a negative skewness. Including the third (location) parameter will shift the distribution, reduce the shape parameter, and change the skewness.

The point estimate for the 5% LEL of a Weibull distribution can be calculated using Eq. 4-11.

$$\begin{aligned} 5\%LRL_{Weibull} &= \theta \cdot (-1\text{n}(0.95))^{1/}{}_{\omega} + x_0 \\ \text{or } 5\%LRL_{Weibull} &= \theta \cdot (0.0513)^{1/}{}_{\omega} + x_0. \end{aligned} \qquad \text{(Eq. 4-11)}$$

4.4.2.1.2 Selecting a Probability Density Function. When selecting a PDF, it is important to consider how representative the data are of the population being modeled and how conclusions to be drawn from the data are to be used. Small data sets are generally assumed to be representative of mean trend but have a low probability of accurately representing variability in a parent population. When the PDF is being used only to select a lower tolerance value and not to characterize the shape of the low tail of the resistance distribution, there is limited value in attaining a precise fit. In such cases a normal PDF is the easiest of the established parametric functions to work with. For products subject to some level of quality control or quality assurance, there is generally some justification for assuming that the parent PDF will be skewed to the high side. Normal and log-normal PDFs give similar results with COVs under 20%. In this case, the normal PDF will give slightly more conservative 5% LTL values.

Documentation of the derivation of R_n should include discussion of the PDF and the process used for its selection. *ASTM Standard D 2925-99,* Section 4.5.7 (ASTM 1999a) suggests comparing a histogram or empirical cumulative distribution function to one or more overlaid parametric distribution functions as a means of justifying the PDF selection. Anderson (1952) discusses goodness-of-fit models and how they vary with distribution type.

Individual pole producers who maintain their own database on pole strength may select any PDF that can be supported by their data as a means of estimating a 5% LTL. The nonparametric PDF assumption is the most conservative and is best used with limited samples. If the sample size is large and pole strengths are supported by simple, conservative models that recognize basic material properties (from small clear tests, coupon tests, and cylinder tests) and permissible defects, the normal or log-normal assumptions are likely to give reasonably conservative estimates of a lower fractile of the PDF, as well.

Any organization interested in using a strictly empirical basis for the derivation of nominal resistance should maintain an up-to-date database for poles representative of those being used. Increasing the size of the database leads to greater confidence in the nominal resistance value. Increasing the sample size over time provides a basis for judging trends in materials and manufacturing that might affect the strength PDF. Larger samples also provide the opportunity for adopting a more rigorous approach to assessing the reliability of a utility line.

4.4.2.2.3 Empirical Analysis. Test data generally require some degree of interpretation. For example, ANSI wood pole dimensions are typically used in design, rather than the measured dimensions of the pole. If empirical strength values are derived using measured pole dimensions and applied using the ANSI size-class minimum dimensions, predicted GLM capacity will be less than the measured value. For this reason, values referred to by *ANSI O5.2-2002* (ANSI 2002), Annex C as "adjusted groundline modulus of rupture" are derived as the average failure moment at groundline, divided by the pole-class minimum groundline section modulus. Here the groundline section modulus was derived using the ANSI 6-ft-from-the-butt value adjusted to groundline using the ANSI-tabulated minimum dimensions to estimate taper. Pole modulus of elasticity estimates are also based on the class minimum dimensions at the butt and tip, assuming a linear taper and constant modulus of elasticity (MOE) value over the length of the pole. These values are therefore intended only for use with the ANSI-tabulated minimum dimensions

4.4.2.2.4 Confidence. A number of factors affect the confidence or assurance that an estimate based on a test sample provides a conservative representation of the target point of the parent distribution. The greater the sample size, the greater the probability that the sample mean and variance will closely approximate the parent population values. For a nonparametric distribution, confidence is characterized in terms of order statistics or the order of magnitude. The smallest value in a sample of 20 is the 5th percentile for that sample but only a 50% probability exists that it will be a conservative estimate of the parent population 5th percentile. The first-order statistic in a sample of 28, on the other hand, has a 75% probability of lying at or below the parent population 5th percentile. For a normal distribution, confidence/ tolerance adjustment factors represent the distance from the mean of a sample to the point estimate in terms of the number of standard deviations. Basically, the confidence bound is set to provide some level of assurance that values derived on the basis of a small sample will encompass or provide a conservative estimate of the value for the parent population.

Table 4-1 provides a listing of order statistics used to estimate a lower 5% tolerance limit with a nonparametric distribution and adjustment factors representing the number of standard deviations from the mean to the 5% LTL of a normal PDF.

It is apparent from this discussion that an empirically derived value for R_n will vary, depending on the PDF assumed to represent the data. It is imperative for the pole supplier to provide documentation to support the assumptions made in the selection of a PDF and the derivation of the nominal resistance.

In Appendix B, the Method 1 section provides examples of the application of the empirical method to obtain the 5% LTL R_n.

4.4.2.2 Method 2: Mechanics-Based Models Used in Conjunction with Monte Carlo Simulation. Maintaining a database of full-sized pole tests can be prohibitively expensive. As an alternative, basic material properties can be used in conjunction with mechanics-based models to estimate mean pole strength. Strength variability, however, is a more complex issue. If there is no covariance between any of the independent variables, variance of a strictly linear model can be estimated as the sum of variances of the individual input parameters, eliminating the need for simulation. When using a nonlinear model with no covariance, variance may be influenced by parameter effects on any nonlinear function. Simulation provides a tool for characterizing this effect. However, models that rely on covariant input parameters are more complex because, for example, wood fiber strength and stiffness both vary with density, age, and moisture content, and the variability in weld strength may be larger with thicker steel plate. Application of Monte Carlo simulation in these cases requires establishment of an accurate covariance matrix and interaction equations to ensure realistic combinations of input parameters. Any influence that one input parameter has on another should be recognized in the development of the virtual structures being evaluated.

Computer simulation routines are designed to randomly generate physical and mechanical properties from defined PDFs assumed to represent the properties found in service, and are parameters of theoretical models used to predict performance. The advantage of Monte Carlo simulation is that statistical strength data are obtained using relatively inexpensive material coupon tests (small, clear samples for wood; cylinder tests for concrete) rather than testing a large population of full-size poles. Basically, the simulation routine compiles a large sample of computer-generated pole strength estimates. The resulting samples are then treated similarly to the empirical data, with the added adjustments for modeling error.

A few pitfalls to simulation must be considered. The most obvious is the question of mechanics-model accuracy. It is difficult to develop and verify a model that accounts for all variables that may influence strength and variability of the full-scale structure. When a model is used to predict performance of a complex system, it should be verified over the full range of input parameters for which it will be used.

Although the model being used may be accurate at predicting performance for any known combination of parameters, it may not accurately represent expected behavior in the tails of a distribution. For this reason, verification tests should be conducted to assess the prediction accuracy at the extremes of the influencing variables. A verification test should include accurate measurement of raw material mechanical properties as well as physical properties of the test poles. The more variable the material and the wider the range of structural configurations to be modeled, the larger the verification database should be. The model verification database should be well-documented and included along with simulation results as support for nominal resistance values to be used.

Nonlinear mechanics-based models employ iterative techniques to predict failure. These models account for change in material as well as geometric properties with increased strain levels. Verification tests are conducted to assess the prediction accuracy at the extremes of the influencing variables. Confidence in simulated data varies with the accuracy of the models as well as the input data. Model accuracy should be verified by comparing model predictions to full-scale pole test data using the actual material and geometric properties of the corresponding test specimen. The data used to establish input PDFs for mechanics-based models should be subject to the same assessment of confidence as the full-scale pole test data.

The number of simulations required to get a satisfactory confidence on estimates of distribution parameters will vary with the complexity of parameter interactions and symmetry of their assumed distribution functions. These topics are discussed in greater detail by Law and Kelton (2000), Hammersley and Handscomb (1964), and Balci and Sargenti (1984). It is often preferable to run a number of trials, each consisting of 200 to 500 simulations, to generate a distribution of point estimates rather than one run of 10,000 simulations. This provides a better indication of variability and confidence bounds. The number of simulations conducted needs to be large enough, however, to provide stable predictions of the 5th percentile.

The PDFs used to characterize the raw data input for simulation models should be based on large enough sample sizes to ensure a standard error (SE) no greater than 10% of the estimated 5th percentile. If normality is assumed, the tolerance limit is estimated using Eq. 4-4 (Natrella 1963). The SE of this statistic varies with sample size (*N*) and sample standard deviation(s) of the sample. It can be approximated using the equation:

$$SE = s\sqrt{\frac{1}{N} + \frac{k^2}{2(N-1)}} \qquad \text{(Eq. 4-12)}$$

where

K = confidence level factor (Table 4-1).

In Appendix B, the Method 2 section provides examples on the application of Monte Carlo simulation along with mechanics-based models to obtain the 5% LTL R_n.

4.4.2.3 Method 3: Default Basis. The default basis is used if there are insufficient data to characterize the pole strength PDF empirically or if demonstrably reliable models have not been developed to provide accurate estimates of pole strength. The default method provides a conservative approach to assigning parameters for estimating R_n. The National Institute of Standards and Technology (NIST, formerly the National Bureau of Standards) proposed guidelines (Ellingwood 1980) for estimating strength variability as a function of so-called professional, material, and fabrication influences.

A simple approach is to obtain a best estimate of mean with some degree of confidence and establish a conservative estimate of variability until more data become available. Pole strength variability, expressed here as COV, is influenced by a number of factors that should be considered. These include inherent material variability (COV_M^2), which can be evaluated using standard material property tests. The geometric variability includes inherent or fabrication-related dimensional and thickness tolerances. Fabrication-induced variability (COV_{FA}^2) for steel, concrete, and FRP poles include manufacturing process effects on geometry and on material strength properties. Finally, the accuracy of the predictive model of pole strength is referred to as the professional factor or model accuracy (COV_P^2). In estimating the strength of a full-sized pole on the basis of raw material test data, confidence in the result is dependent on the accuracy of the model being used.

Finally, consider so-called other effects (COV_O^2) such as deterioration, design error, and environmental risk. Poles may be damaged due to mishandling during installation or they may experience deterioration from environmental exposure such as high temperatures, grass fires, ultraviolet radiation, decay, corrosion, cracking, and spalling. These effects are not generally included in a design model and they do not have the same effect on all poles in a line. Poles removed from a line after 30 years of service are likely to have neither the same strength nor the same strength variability they had when they were installed.

Combining these individual effects can provide an estimate of the pole strength COV_R:

$$COV_R^2 = COV_M^2 + COV_{FA}^2 + COV_O^2 + COV_P^2 \quad \text{(Eq. 4-13)}$$

Further information is given in the American Iron and Steel Institute's (AISI) "Specification for the design of cold-formed steel structural members" (AISI 1996) and "Development of a probability-based load criterion for American National Standard A 58" by Ellingwood et al. (1980). These and other publications support overall default values for COV_R of steel and concrete poles of 0.15, and 0.20 for wood poles. A number of variables with fairly broad ranges affect the strength of FRP poles; therefore, useful default values cannot be established for these poles at this time.

4.5 PROOF LOADING

Proof loading to a design value provides some degree of quality assurance but, in the absence of pole failure, this procedure provides little useful information on the strength distribution. Even when the proof loading does result in occasional failures, such results can only provide a basis for assigning some level of confidence about the relative proximity of the proof load and some fractile of the strength distribution. The drawback of proof loading to a level that results in occasional failure is that it provides some risk of causing undetected damage to the pole.

If backed by research to correlate nondestructive evaluation (NDE) parameters to strength, proof loading methods might be developed to enable estimates of LELs of strength. In general, however, NDE parameters are not used to define strength since NDE parameters are poorly correlated with strength.

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