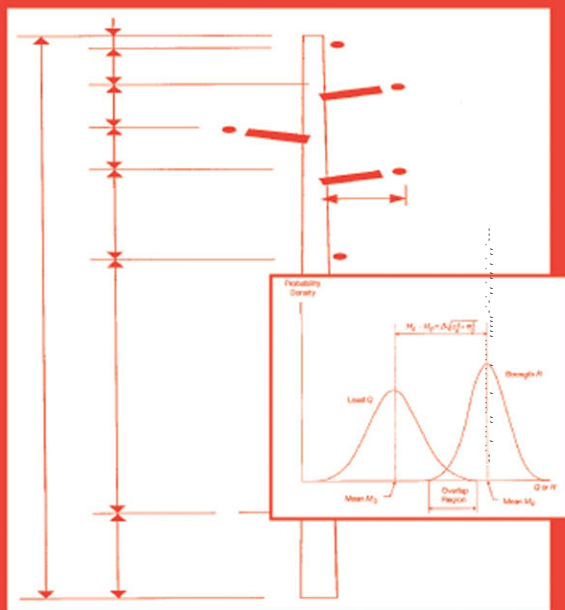


RELIABILITY-BASED DESIGN OF UTILITY POLE STRUCTURES



Reliability-Based Design of Utility Pole Structures

Prepared by
Reliability-Based Design Committee of the
Structural Engineering Institute (SEI) of the
American Society of Civil Engineers

Edited by
Habib J. Dagher



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Chapter 1

INTRODUCTION

1.1 CURRENT PRACTICE

There is a need to provide a design methodology for distribution and transmission poles that yields consistent structural reliabilities across all material types: wood, steel, concrete, fiberglass, and other new materials. Whereas this document is currently focused on poles, future editions will have an expanded scope to cover reliability-based design (RBD) for all utility structures and components.

Prevailing U.S. practice and most state laws require that transmission and distribution lines be designed to meet the basic safety requirements of whichever edition of the *National Electrical Safety Code (NESC)* is adopted by each state. The current *NESC* rules (IEEE 2002) for the selection of design load and strength factors are largely based on successful experience but they do not have a strong theoretical foundation. Some designers find the rules too restrictive, whereas others adopt more conservative criteria. In fact, individual utilities often develop their own loading criteria to supplement the *NESC* rules.

A desire to achieve more consistent structural reliabilities across materials was the impetus for the development of the American Society of Civil Engineers (ASCE) *Manual 74*, "Guidelines for transmission line structural loading" (ASCE 1991) and the International Electrotechnical Commission (IEC), "Design criteria of overhead transmission lines," *IEC 60826* (IEC 2002). Although *ASCE Manual 74* offers consistent methods to calculate loads, there is still a need to provide consistent methods to calculate strength across pole materials.

The nominal strength (R_n) of a pole is calculated using a variety of approaches and guides, depending on the pole material type. This includes, for example, *ASCE Manual 72* for steel poles (ASCE 1990), *ANSI O5.1-2002* for wood poles (ANSI 2002), and the Precast/Prestressed Concrete Institute (PCI), "Specification guide for prestressed concrete poles" (PCI 1999). *ASCE Manual 104*, "Recommended practice for fiber-reinforced polymer products for overhead utility line structures,"

has recently been developed for fiber-reinforced polymer (FRP) poles (ASCE 2003).

In addition to these guides, pole manufacturers typically use in-house models for predicting the nominal strength of prestressed concrete poles and FRP poles. These strength guides, standards, and in-house methods have essentially evolved independently so there is little consistency in the definition of nominal strength across pole materials. For some pole materials and design methods, the nominal strength value is close to a mean strength, whereas for other pole materials the nominal strength represents a more conservative value, such as a 5% or 10% lower exclusion limit (LEL). As a result, a wide range of possibilities currently exists concerning what the nominal strength of a pole really represents. This results in inconsistent and unknown reliability levels for different pole materials.

To illustrate the problem, refer to Fig. 1-1. Any strength property of a pole, such as the lateral strength of a Class 1 wood pole in a cantilever test, is a random variable (testing 100 identical poles in the same manner will result in 100 different values of the strength). The many sources of uncertainty in observed pole strength include inherent material property variability, geometric variability, manufacturing effects, and variation in the testing method. The strength of a pole may thus be characterized by a probability density function (PDF) with a mean value (m) and a coefficient of variation (COV), as shown in Fig. 1-1. The COV (the standard deviation divided by the mean) varies with the pole material and pole type. The COV is higher, for example, for wood poles than for steel poles. The higher COV causes the PDF for wood poles to be wider and flatter than for steel poles. The procedure used to calculate the nominal or characteristic strength of the pole (whether per *ASCE Manual 72* [ASCE 1990] or *ANSI O5.1-2002* [ANSI 2002]) yields a nominal strength R_n that falls somewhere on the horizontal axis of the PDF in Fig. 1-1.

Figure 1-1 shows three possible positions of the nominal strength: $R_n = R_1$, R_5 , and R_{50} , representing a 1%, 5%, and 50% LEL, respectively. For example, a 5% LEL nominal strength (a 5th percentile) is a value not achieved by 5% of the poles. For a normal density function, the 5th percentile is 1.645 standard deviations below the mean:

$$R_5 = m - 1.645 (m \times \text{COV}). \quad (\text{Eq. 1-1})$$

A 50% LEL or 50th percentile nominal strength is a value not reached by 50% of the poles. For a normal density function, R_{50} is also the mean strength. It follows that $R_{50} > R_5 > R_1$ and the distance between these values grows with increasing COVs. Clearly, unless strength design methods for different pole materials yield consistent nominal value definitions (e.g., all materials at the 5% LEL), it will be difficult to achieve consistent reliability among material types.

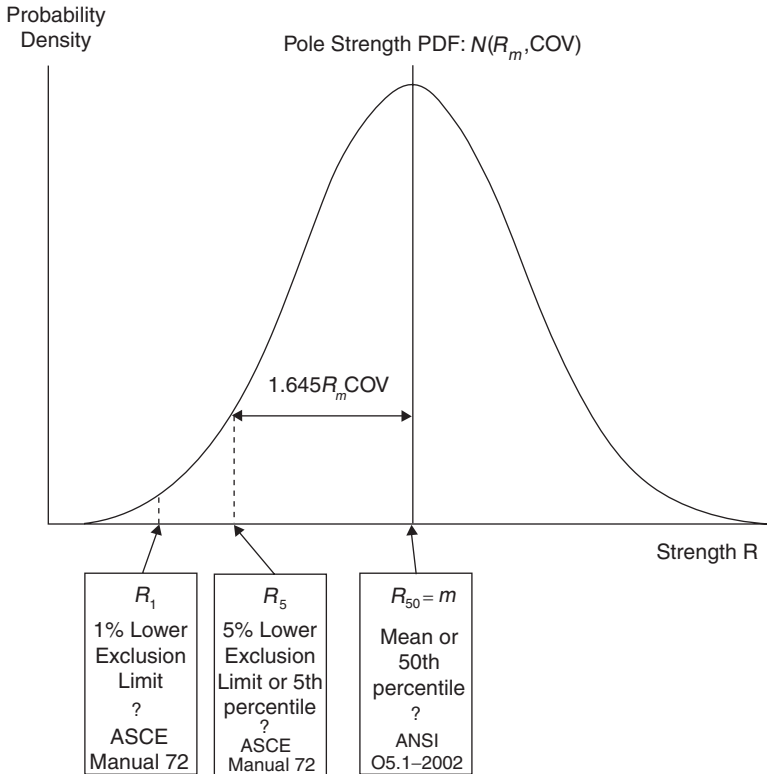


FIGURE 1-1. Probability Density Function (PDF) (Normal Distribution).

1.2 RELIABILITY-BASED DESIGN

Failure of a component or a structure occurs when the load (Q) exceeds the resistance (R). If the PDF of the strength and the load are known, then the probability of failure of a component may be estimated using the overlap region under the two curves (Fig. 1-2). Minimizing the overlap between the two curves is commonly achieved by moving the curves apart by moving the strength PDF to the right (e.g., by applying safety factors or partial safety factors to the mean or nominal design values). The distance between the means of the two curves ($M_R - M_Q$), measured in number of standard deviations of $(R - Q)$, is referred to as the reliability index β (Fig. 1-2).

$$\beta = (M_R - M_Q) / (\sigma_R^2 + \sigma_Q^2)^{0.5}. \quad (\text{Eq. 1-2})$$

The calculation of reliability index β in Eq. 1-2 and Fig. 1-2 is valid only when both R and Q are normally distributed and uncorrelated.

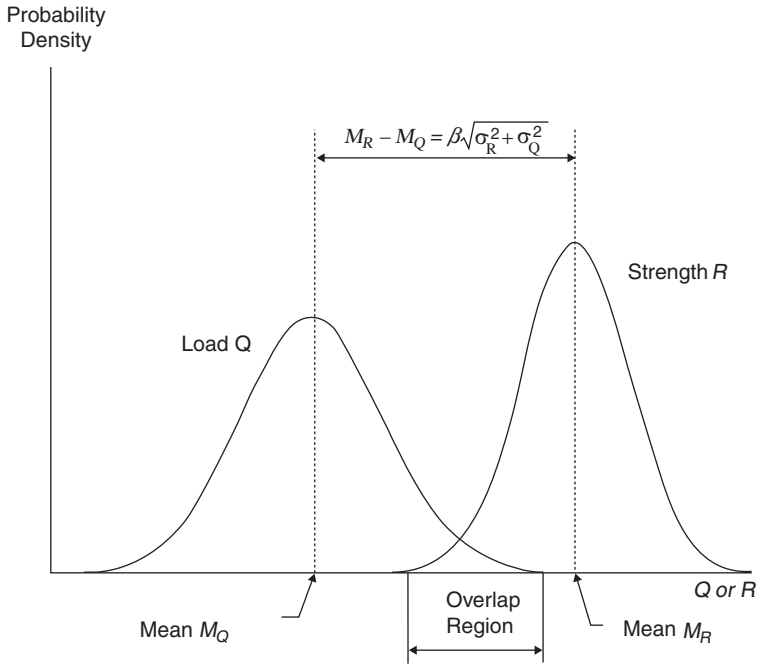


FIGURE 1-2. Failure Occurs in Overlap Region Where the Load $Q > \text{Strength } R$.

In this case, the reliability index β and the probability of failure are related as follows:

$$\text{Probability of Failure} = 1 - \Phi(\beta), \quad (\text{Eq. 1-3})$$

in which Φ is the cumulative density function (CDF) of the normal distribution. Larger reliability indices β mean more distance between the two curves in Fig. 1-2 and a smaller probability of failure. Components with equivalent reliability indices β have relatively equivalent probabilities of failure.

In RBD, load and strength factors (also called partial safety factors) are selected so that components have relatively equivalent reliability indices β (Ellingwood et al. 1980). Target reliability indices for conventional designs are typically selected by reliability calibration. The RBD methods for transmission lines are described in "Design criteria of overhead transmission lines," IEC 60826 (IEC 2002), "Reliability-based design of transmission line structures" (Peyrot and Dagher 1984), and "Reliability-based design of foundations for transmission line structures" (EPRI 1995). These documents provide additional references and details for

conducting structural reliability evaluations, including reliability index calculations, reliability calibrations of new RBD methods, and load and resistance factor design (LRFD) methodologies.

1.3 OBJECTIVE

The objective of this manual is to outline a methodology to achieve relatively consistent structural reliability across all pole materials. In future editions, this document will be expanded to cover RBD for other types of utility structures, components, and related load considerations.

1.4 SCOPE

The structural reliability of poles designed according to the 2002 *NESC* (IEEE 2002) provisions and existing strength design guides such as *ASCE Manual 72* (ASCE 1990) or *ANSI O5.1-2002* (ANSI 2002) varies depending on the material type. Reasons for the discrepancy relate to the historical development of the deterministic design approach used in the *NESC*, as well as inconsistencies in the nominal strength definitions in the various material-specific strength design guides.

This manual provides a design methodology for achieving relatively consistent structural reliability among poles of different materials. Specifically, this manual provides the following:

1. Target reliability levels for poles of *NESC* Grades B and C construction, applicable to all material types.
2. A consistent definition of nominal strength that is applicable to all pole materials, including the following:
 - a. Consistent test methods, sample sizes, and data analysis and selection procedures for establishing the statistical properties of pole strength.
 - b. Incentives (reduction in required load/strength factors and pole cost) for manufacturers to reduce variability and better define pole statistical properties.
3. Default statistical strength data (COV, % LEL of nominal strength) for the RBD of poles of all materials.
4. An RBD methodology that includes the following:
 - a. General LRFD equations.
 - b. Load factors applicable to all material types and material-dependent strength reduction factors.
 - c. Characteristic or design load values obtained using the procedures in *ASCE Manual 74* (ASCE 1991).

- d. A set of load and strength factors for the LRFD equations that achieve reliability levels corresponding to either *NESC* Grade B or Grade C construction, including the following:
 - i. A table to adjust strength factors depending on the LEL of nominal strength and COV of pole strength (see Table 2-2).
 - ii. A discussion of how to change reliability levels.

1.5 BENEFITS

This manual provides a design methodology for distribution and transmission pole structures that yields relatively consistent reliabilities across all materials. This manual also addresses the following:

1. Defines minimum reliability levels for both *NESC* Grades B and C construction on the basis of reliability analyses of existing *NESC* pole designs. Therefore, designers can still produce designs to *NESC* Grades B and C with more consistent results.
2. Provides a means for quantifiably adjusting reliability whenever needed or justified. An essential line should be more reliable than a less important line.
3. Achieves relatively uniform structural reliability across all pole materials, thereby allowing utilities to compare the cost of equivalent lines made of different materials.
4. Facilitates innovation by allowing the introduction of new materials into pole design.
5. Encourages manufacturers to continually improve their products by providing design incentives for more reliable poles and better databases for pole strength. A manufacturer that develops better statistical data on pole strength is allowed to adjust the strength factors accordingly.
6. Provides uniform procedures for defining the nominal strength of transmission and distribution structures to be used in conjunction with the available strength guides (such as *ASCE Manual 72* [ASCE 1990] and *ANSI O5.1-2002* [ANSI 2002]). This manual also suggests that all future editions of the strength guides provide nominal or characteristic strength values at or near the 5% LEL. Table 2-2 is provided to be used with existing strength guides to adjust load and strength factors depending on the LEL of nominal strength and COV of pole strength.
7. Complements *ASCE Manual 74* (ASCE 1991). This manual does not provide new methods for calculating loads and load combinations, instead referring to the *ASCE Manual 74* procedures for computing design loads and load factors that are independent of the materials

of the supporting structures. It is consistent with the *ASCE Manual 74* approach that a loading agenda should reflect uncertainties in the loads and the accepted risk that these loads may be exceeded.

8. Brings pole structural design in line with well-established RBD codes such as the American Association of State Highway and Transportation Officials (AASHTO) "Standard specifications for highway bridges" (AASHTO 2002), the American Institute of Steel Construction (AISC) "Load and resistance factor design specification for structural steel buildings" (AISC 1999), and the American Forest and Paper Association "Load and resistance factor design (LRFD) manual for engineered wood construction" (AF&PA 1996).

1.6 RELATION TO NATIONAL ELECTRICAL SAFETY CODE AND OTHER ASCE GUIDES

The *NESC* (IEEE 2002) rules contain basic provisions considered necessary for the safety of employees and the public under specified, deterministic load conditions. The *NESC* rules are not intended to be used as a design specification. This manual, which provides a design methodology, should be used in conjunction with any *NESC* safety requirements.

This manual provides reliability-based loads and strength factors that may be used with the *ASCE Manual 74* (ASCE 1991) load conditions.

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Chapter 2

RELIABILITY-BASED DESIGN METHODOLOGY

2.1 INTRODUCTION

This section describes a reliability-based design (RBD) methodology for transmission and distribution (T/D) line structures. This RBD approach is calibrated to yield average reliability levels consistent with years of history and practice with existing deterministic design approaches. Therefore, on average, designs will be nearly equivalent to past practice.

This methodology strives to correct the problem of inconsistent reliabilities among different material types. Minimum reliability levels are recommended for different grades of construction. Simply selecting different return periods in designing the T/D line quantifiably varies reliability levels. The method uses one set of load and load factors regardless of the material type. The strength reduction factors are a function of material type; the equations are based on initial strength before material degradation occurs.

2.2 STRUCTURAL SYSTEMS

A T/D line consists of two separate structural systems: the structural support system (towers, poles, and foundations) and the wire system (including insulators and hardware). Each system should be considered separately, even though it is evident they are closely related to each other.

The structural support system has the task of supporting the weight of the wire system and accumulated ice and of resisting the winds acting on both systems. The wire system consists of the conductors and shield wires, which may be severely tensioned by ice loading or extreme wind loading. The wire system includes all components such as dead-end insulators and hardware in series with the wires and all major angle and dead-end structures that are critically affected by the wire tensions. Major loads of a transmission line are generated on or by the wire system on the structures themselves, except for high-intensity-type winds such as tornados. Focus should be on the wire system to understand what happens to transmission

or distribution lines, especially when wires are loaded with ice. Whereas the support system can support very heavy vertical loads at relatively low cost, the strongest vertical capacity support system may be overstressed when unusual or unexpected loads, such as wind-blown or falling debris (branches), affect the wire system. A simple break in the wire system may promote a cascading failure that is difficult to control.

2.3 DESIGN OF WIRE SYSTEM

For the reasons discussed in the previous chapter and others, the components of the wire system should be selected conservatively, with loadings limited to damage levels and set well below the rated strengths of the components. The strength levels of the structures and foundations of the support system can be adjusted relative to each other. Generally, it is desirable to ensure that the foundations are more reliable than the supported structures. Some utilities, however, elect to have foundations for directly embedded structures fail (i.e., deflect excessively) prior to structure failure (i.e., buckling or rupture), since a foundation's deflection can be repaired quite easily compared to replacing a complete structure. In some cases the deflection can reduce the loads on the structure. The strength levels of important angle and dead-end structures can also be adjusted upward from those of tangent structures to make them more reliable.

2.4 TYPES OF LOAD-PRODUCING EVENTS AND RETURN PERIOD

When describing loads in a T/D line system, it is convenient to distinguish between the events that produce the loads and the resulting loads in the components of the subsystems. The loads are direct forces on the conductors, ground wires, or structures. The events producing the loads can be classified as weather-related, accidental, and construction and maintenance (C&M) events.

2.4.1 Weather-Related Events

Current design procedures, such as those found in the *National Electrical Safety Code (NESC)* criteria (IEEE 2002) for extreme wind, incorporate some probability information about weather-related events. This may be done intuitively through experience or by direct use of the return period (RP) concept, whenever data are available. Design nominal values are normally specified in a code or loading agenda in such a manner that their risk of

being exceeded is relatively low. A nominal value often used is that value of the variable that has the probability $1/\text{RP}$ of being exceeded in one year. Such a value is said to have an RP-year return period. A nominal value for a weather-related event or a load with an RP-year return period is indicated herein by the subscript RP. For example, a 50-year RP wind velocity, V_{50} , has a probability of 0.02 (2%) of being exceeded in any one year.

Since a direct relationship exists between weather-related events and corresponding load effects, the load effect from a weather-related event with an RP-year return period also has an RP-year return period. In this manual, design equations that constitute the *reliability* requirement use weather-related events and corresponding loads with a specified RP as the nominal events and loads.

The probability that an event or load with an RP-year return period will be exceeded at least once during the planned lifetime of a line (for example, 50 years) is given in Table 3-1 in Chapter 3. This probability is a useful indicator but it does not correspond to the probability of failure of the line or to that of any of its components. More information on the type of calculations yielding the results of Table 3-1 can be found in the commentary of *ASCE Standard 7-02* (ASCE 2002).

2.4.2 Accidental Events

Some of the events that produce loads in a transmission line system cannot be described statistically because of their nature or lack of data. Accidental events such as breaks of components from defects, wear, fatigue, or impact or failures of entire structures from landslides, tornadoes, sabotage, or any other unforeseen phenomena fall in that category. Design procedures do not control the occurrence of these events but attempt to minimize their consequences. Because of this, the designer must make sure that if a failure is triggered by an accidental or weather-related event, it will not propagate without control. This *security* requirement can be accomplished by designing for special loadings (longitudinal or torsional loads) at all or some structures by load-limiting devices such as mechanical fuses or by inserting stop structures at specified intervals in the line.

2.4.3 Construction and Maintenance Events

Some line components may be subjected to their critical loading during C&M operations. Once the magnitudes of the loads produced by the operations are established, they should be multiplied by a load factor to provide an adequate level of safety. Design for these factored loads, in addition to those specified by national regulations or codes of practice, constitute the *safety* requirement.

2.5 LIMIT STATE DESIGN

A limit state defines acceptable and unacceptable structural behavior evaluated in design. Limit states can be normally classified into the three categories of serviceability, damage, and failure (ultimate strength) limit states. For T/D structures, serviceability limit states include vibration, clearances, deflections, and alignment. Damage and failure limit states include any type of partial or complete failure. Designers have always checked limit states in design by comparing load effects with corresponding limits. Therefore, limit state design does not represent a new design method but, rather, is a collective term to describe all the limits checked in design.

In this document, design equations that deal with ice, wind, and temperature are intended to control the probability of occurrence of damage limit states; the equations are provided to prevent permanent damage of components from weather-related events. The design equations that deal with security-related loads address failure limit states; the concern is not with the damage of one or two structures because the line has already failed but, rather, with the possibility of cascading-type failures.

2.5.1 Loads and Load Effects

Loads on a T/D line, denoted herein as Q , are forces applied on the wires or forces applied directly to the supporting structures. Forces, displacements, and stresses caused by the loads in the various components of the subsystems are called load effects. A load effect herein will be denoted "Effect of []," where the terms inside the brackets are the loads (Table 2-1). Loads are further discussed in Chapter 3.

2.5.2 Component Strength

A component is normally selected in such a way that its strength multiplied by a strength reduction factor exceeds the corresponding design loads multiplied by load factors. The actual strength of a given component is a random variable. In current design applications, the strength of a component is identified by a unique value called its nominal strength. The nominal strength of a component, R_n , is normally calculated with equations described in the appropriate design document (AISC 1999; ANSI 2002; ASCE 1992; PCI 1999). (ASCE *Standard 10-97* [ASCE 2000] was previously American Society of Civil Engineers [ASCE] *Manual 72*, [ASCE 1990].)

The nominal strength may also be provided by a manufacturer in the form of a minimum or guaranteed strength or as a percentage of an

TABLE 2-1. Load Conditions That May Be Considered in Design

Equation	Load Case Description
2-1a and 2-1b Weather Loads	Extreme wind from any direction Extreme ice with reduced wind (combined ice and wind) Unbalanced ice without wind (where applicable) Substantial wind on reduced ice (where applicable)
2-2 Security Loads	Failure containment criterion or loading (for example, broken conductor load)
2-3 Construction and Maintenance Loads	Structure erection loads Stringing loads Worker load (250 lb)
2-4 Legislated Loads	Legislated loads (National Electrical Safety Code)

estimated breaking load. For example, the nominal strength in compression of a given type of steel angle in a lattice tower is given by a compression formula in American National Standards Institute (ANSI)/ASCE *Standard 10-90* (ASCE 1992). A nominal value, R_e , is said to be the $e\%$ exclusion limit of strength if it has an $e\%$ probability of not being achieved. Chapter 1 described where the value R_e is located relative to the probability density function (PDF) of the strength. Usually, the nominal values are set in a way that most components tested at loads equal to the nominal strength would survive the test. The percentage of components not surviving is equal to the exclusion limit.

Unfortunately, the exclusion limits of R_n for different types of components are not the same and are often unknown. The exclusion limit for the design strengths of many metallic components of lattice or tubular structures are in the range of 1% to 10%, with corresponding coefficients of variation (COV_R) in the range of 5% to 20%. The strength values specified in *ANSI Standard O5.1-2002* (ANSI 2002) for wood poles have exclusion limits that are significantly higher than 5% (Bodig et al. 1986). Design equations for the strength of foundations may have exclusion limits as high as 50%. This manual recommends that, in the future, published nominal values for the strengths of all transmission line components be established at the lower exclusion limit (LEL) of 5%. In any case, if statistical data are available on the strength of a particular component, the strength value's exclusion limit can be determined as shown in Chapter 4.

2.5.3 Load and Resistance Factor Design Format

2.5.3.1 Load and Resistance Factor Design Equations. Load and resistance factor design (LRFD) describes one way of assessing behavior at various limit states. The following set of LRFD design equations is recommended for the design of components in a transmission line:

Weather-Related Loads (Reliability-Based). Equation 2-1a (or 2-1b) is the design equation that controls reliability for weather-related events. The limit state considered is damage of a component caused by extreme wind or combinations of ice and wind.

$$\phi R_n > \text{effect of [1.1 DL and } \gamma Q_{50}\text{]}. \quad (\text{Eq. 2-1a})$$

The 1.1 load factor for self-weight is less than the 1.2 factor used in ASCE 7 (ASCE 2002b), recognizing that self-weight is more predictable in utility structures than it is in building structures.

Or,

$$\phi R_n > \text{effect of [1.1 DL and } Q_{RP}\text{]}. \quad (\text{Eq. 2-1b})$$

where

ϕ = strength factor (Table 2-2)

γ = load factor for wind (Table 3-2 in Chapter 3) or ice thickness factor for combined ice and wind (Table 3-3 in Chapter 3)

R_n = nominal strength of the component

DL = dead loads (not including weight of ice)

Q_{50} = wind or combined ice and wind loads based on a 50-year RP

Q_{RP} = loads similar to Q_{50} , based on RP.

If $RP > 50$ years, the structural reliability is increased. For example, Q_{100} wind loads are approximately 15% larger than Q_{50} wind loads. (Refer to Chapter 3, Section 3.4 for more details.)

Security Loads. Equation 2-2 provides for the security of the line. Ideally, the limit state considered in Eq. 2-2 should be an ultimate or failure limit state. The purpose of the equation is not to prevent localized damage but, rather, to prevent failure propagation. Assume, for simplicity, that damage and ultimate limit states are identical. With that conservative assumption, the same R_n can be used in all of the design equations. The conservative use of a damage limit with respect to security loads may impose a cost penalty on the utility.

$$\phi R_n > \text{effect of [DL and SL]}, \quad (\text{Eq. 2-2})$$

TABLE 2-2. Strength Factor ϕ^a

Nominal Strength Lower Exclusion Limit, e (%)	Strength Factor, ϕ for $COV_R =$		
	0.05	0.10	0.20
0.1	1.00	1.16	1.48
1	0.97	1.07	1.27
2	0.95	1.04	1.21
5	0.93	1.00	1.12
10	0.92	0.96	1.04
20	0.90	0.92	0.95
50	0.86	0.85	0.81
Mean	0.86	0.85	0.79

^a Assumes log-normal strength property.

where

ϕ = strength factor

R_n = nominal strength of the component

DL = dead loads

SL = security loads.

Construction and Maintenance Loads (Safety). Equation 2-3 considers the damage limit state of a component from C&M loads.

$$\phi R_n > \text{effect of } [\gamma_{CM}(\text{DL and C\&M})], \quad (\text{Eq. 2-3})$$

where

ϕ = strength factor

R_n = nominal strength of the component

γ_{CM} = load factor applied to the C&M load

DL = dead loads

C&M = loads produced by construction and maintenance operations.

Legislated Loads. Equation 2-4 is listed to emphasize that requirements from governing codes should always be considered.

$$\phi_{LL} R_n > \text{effect of } [LL], \quad (\text{Eq. 2-4})$$

where

ϕ_{LL} = legislated load strength factor
 R_n = nominal strength of the component
 LL = legislated loads.

The load factor γ in Eq. 2-1a (or the RP in Eq. 2-1b) allows a designer to modify the reliability of a line. The absence of load factor in Eq. 2-2 emphasizes that the design loads providing the security level of a line cannot be described statistically. However, simply increasing the nominal loads can increase the security of a line. Suggested minimum security loads (SLs) are presented in Chapter 3; also, refer to the *National Electrical Safety Code* (IEEE 2002) and the working draft of *ASCE Manual 74*.

The strength factor ϕ in Eqs. 2-1a through 2-3, obtained from Table 2-2, accounts for the nonuniformity of the exclusion limits in published information and formulas for nominal strength R_n for different materials. This factor also accounts for differences in strength coefficients of variation, COV_R . If all strength design guides published nominal strength values at a 5% lower exclusion limit (5% LEL), Table 2-2 could be effectively reduced to one row.

The strength and load factors to use with Eq. 2-1a (or strength factor and RP to use with Eq. 2-1b) could be developed by a number of different techniques. They could be chosen by consensus to represent current or projected practice. Preferably, they can be selected to control the reliability (or probability of failure) of the components of the line. That selection process is commonly referred to as reliability-based LRFD. Section 2.6 will describe a method for implementing RBD for T/D line structures.

2.5.3.2 Load Conditions. Equations 2-1a through 2-3 are generic equations to satisfy the basic requirements of reliability, security, and safety. Equation 2-4 represents design requirements for legislated loads. In practice, several load cases are considered in each of the categories of loads covered by the equations. Table 2-1 describes load cases normally considered in design.

Design loads on supporting structures are generally obtained by applying the selected load conditions to assumed maximum vertical span, wind span, and line angle. However, actual spans, line angles, and combinations of loads less than the maximum loads may be critical in some cases. The designer should be aware of conditions where the lesser values result in higher stresses in some components of the structures.

2.6 RELIABILITY-BASED DESIGN

The variability of loads and strengths can be formally considered in design through applications of probability theory. A probability-based design procedure is one that considers the probability of occurrence of a given limit state over a fixed period of time, usually one year or the

planned lifetime of the system. The procedure should address two essential points: (1) how the probability of occurrence of the limit state is to be estimated, and (2) how small it should be.

The ultimate goal of RBD is to control the reliability of the line system. Line reliability is directly affected by the reliability of each component in each subsystem. Because accurate control of a line system's reliability is beyond the current state of the art, the approach adopted in this manual is to control the relative failure probability (RFP) of different lines or of different structures within a line. Line RFP is approximately increased or decreased by a given factor by simply increasing or decreasing the RP of the load used in designing the line. For example, traditional *NESC* Grade B or C construction may be achieved by changing the load factor γ , as shown in Table 2-3.

2.6.1 Target Reliability Levels and Corresponding Load Factors

Different load factors are used to achieve relative target probabilities of failure for different types of construction. A larger design load factor results in a more reliable line. Table 2-3 provides the minimum design load factors for *NESC* Grades B and C construction with transverse wind force applied to the ice-covered conductors, as appropriate. Section 3.4 in Chapter 3 provides further information on load factors and their effects. The load factor γ is applied to the wind force and to the ice thickness (Table 2-3).

The RBD method for the load factors in Table 2-3 uses nominal climatic design loads based on a 50-year RP (Q_{50} in Eq. 2-1a). Historical *NESC* Grade C

TABLE 2-3. Minimum Load Factors for Transverse Wind Force and Ice Thickness Corresponding to *NESC* Grades B and C Construction

<i>NESC</i> Grade of Construction	Minimum Load Factors, γ Applied to 50-Year Events (Eqs. 2-1a and 2-1b)	
	Extreme Wind	Combined Ice and Wind
Grade B	Wind Force: 1.0	Wind Force: 1.0 Ice Thickness: 1.0 ^b
Grade C	Wind Force: 0.5 ^a	Wind Force: 1.0 Ice Thickness: 0.5 ^b

^a If any portion of the structure or its supported facilities exceeds 60 ft above ground or water level, a load factor of 1.0 should be used.

^b The load factor for the ice thickness is applied to the thickness of ice on the conductor, structure, or component prior to calculating the associated load (such as weight or transverse wind force). The wind force load factor is then applied to the calculated wind load on the ice-covered component. The ice weight load factor, if any, is applied to the resulting weight of ice, including the ice thickness load factor.

construction-factored loads are less than 50-year RP values (i.e., *NESC* Grade C construction is designed for less than a 50-year RP load). This results in RBD load factors less than 1.0 for Grade C construction in Table 2-3.

The load factors in Table 2-3 result from the author’s extensive effort to calibrate reliability to historic *NESC* designs. As part of the effort, three types of tangent poles were designed to full utilization at 40 U.S. locations for both *NESC* Grade B and Grade C construction (240 poles). A fully utilized design has a nominal design load effect equivalent to 100% of the nominal design strength. The reliabilities of these tangent poles designed according to *NESC* criteria were evaluated using the best available probabilistic wind and wind-on-ice models at each of these 40 locations. The reliability of *NESC* designs varied with location and pole height, even within the same loading district. Since this calibration study was restricted to tangent structures, line tension effects were not considered at this time. Future editions of this manual will consider angle and dead-end structures.

Using this information, the following target reliability levels for tangent structures were selected to provide equivalence to *NESC* construction in the majority of the United States: reliability index $\beta = 2.0$ for Grade B construction and $\beta = 1.5$ for Grade C construction (see Chapter 1, Section 1.2 for definition of β). These are annual β values obtained through Monte Carlo simulations of poles designed using *NESC* at 40 locations in the United States. The strength was assumed log-normal and the statistical models for wind and for wind-on-ice were the ones used in developing *ASCE 7* (*ASCE 2002b*).

The target reliability levels selected here will serve to maintain more consistent reliability while minimizing changes in current pole design. Departures from *NESC* traditional designs will result at locations where the wind maps or combined ice and wind maps used in *ASCE Manual 74* (or *ASCE 7*) (*ASCE 1991*; *ASCE draft*; *ASCE 2002*) are significantly different from the *NESC* district map (light, medium, and heavy). Departures from *NESC* designs will also occur for poles close to but less than 60 ft high that have not traditionally been designed for extreme wind.

Techniques for calculating the loads Q_{50} , Q_{RP} , SL , and $C\&M$ in Eqs. 2-1a through 2-3 are described in Chapter 3 of this manual as well as in *ASCE Manual 74* (*ASCE 1991*; *ASCE draft*). The load factor γ in Eq. 2-1a (or the RP in Eq. 2-1b) that corresponds to a given RFP can be obtained from Tables 3-2 and 3-3. The RFP is defined as follows:

Probability of Failure of a Component or Structure

$$RFP = \frac{\text{Designed to a Load Return Period RP}}{\text{Probability of Failure of the Same Component or Structure Designed Using a 50-Year Return Period Load}} \quad (\text{Eq. 2-5})$$

The factored load γQ_{50} (with γ from Table 3-2 or 3-3 in Chapter 3) is an approximation of the load Q_{RP} . This is further discussed in Chapter 3.

The commonly designated “15% rule” for wind loadings indicates that every time the wind load factor γ is increased by increments of 0.15 (e.g., 0.85, 1.00, 1.15, 1.30, 1.45 in Table 3-2 in Chapter 3), the return period of the wind load is doubled (e.g., 25, 50, 100, 200, and 400 years). This also results in approximately reducing the probability of failure against wind by a factor of 2.

If statistical data on weather-related events are available, the use of Eq. 2-1b (using Q_{RP}) is recommended over that of Eq. 2-1a (using γQ_{50}). In addition to meeting the minimum values given in Table 2-3, selection of an appropriate RFP should be based on the importance of the line and its location and length. For example, a higher reliability may be selected for a portion of a line located in an urban area.

The reliability of a long line is less than that of a short one, all design parameters being the same. The primary reason for the reduced reliability is that because a long line is exposed to a larger number of severe events, its likelihood of experiencing some kind of failure is greater. Also, weak components are more likely to be exposed in a larger population of components. The line designer may consider decreasing the RFP for long lines in order to increase the line’s reliability. Methodology to quantify the reliability of long lines is outside the scope of this manual but is addressed in Dagher et al. (1993) and IEC 60826 (IEC 2002).

Design for loads with a return period of 50 years (i.e., RFP = 1) is considered the basis for transmission line work. For temporary, noncritical components and for Grade C construction, an RFP >1 may be acceptable; that is, a design with RP <50 years may be used, as shown in Table 2-3, Grade C.

2.6.2 Selection of Strength Factor

The purpose of the strength factor ϕ in Eqs. 2-1a through 2-3 is to account for the nonuniformity of the exclusion limits that currently exist in published formulas for nominal strength R_n and for differences in strength coefficients of variation COV_R . The values of the strength factor ϕ can be obtained from Table 2-2. The factors indicated assume a log-normal distribution of strength, which is considered more realistic than the normal distribution.

In the future, the authors recommend that all T/D line strength design guides publish strength values at the 5% lower exclusion limit (5% LEL), or the estimate of this value—the 5% lower tolerance limit (5% LTL) with a minimum confidence of 50%. Chapter 4 discusses methods for obtaining the 5% LTL as well as the coefficient of variation COV_R , including sample sizes and data reduction techniques.

Larger strength values of the 5% LTL are obtained if more test data are available or product variability is reduced. This gives an incentive for manufacturers to conduct sufficient testing of their products so they can use higher design values and maintain product consistency. For large sample sizes or very small variability (COV_R), the 5% LTL approaches the 5% LEL value.

The use of Table 2-2 requires knowledge of the LEL corresponding to the nominal strength R_n of a component, as well as the COV_R of the component strength. (The 5% LTL, as determined by the method in Chapter 4, may be used for R_e for $e = 5\%$.) Both numbers will vary from one material type to another (steel versus wood pole) and will depend on the strength design guide or method used to calculate nominal strength (e.g., *ANSI O5.1-2002* [ANSI 2002] or *ASCE Manual 72* [ASCE 1990]).

The strength factors of Table 2-2 are to be applied to the design nominal strength used for the pole, using the exclusion limit of the nominal strength and the COV of the pole strength. For example, according to *ANSI O5.1-2002*, Douglas fir poles have a designated fiber stress of 8,000 pounds per square inch (psi) with a $COV = 0.20$. Since this designated fiber stress value represents the mean groundline fiber strength (i.e., not the 5% LEL strength), this stress level must be multiplied by a strength factor (ϕ) of 0.79 (Table 2-2) for design of the pole:

$$F_{b5\%LEL} = 0.79 \times 8,000 \text{ psi} = 6,320 \text{ psi}.$$

The factors in Table 2-2 are based on a log-normal strength distribution and have been derived to result in relatively consistent reliability (i.e., approximately the same probability of failure) when structures are subjected to extreme winds, independent of the type of pole product and material (i.e., COV_R). As a result, these factors do not directly correspond to the simple calculation that may otherwise be used to obtain a 5% LEL. (For example, assuming a normal strength distribution without regard to the failure rates for different materials, the simple calculation would have been $[1 - 1.645 \times COV_R]$ applied to a mean strength.) See Chapter 4 and Appendix A for additional discussion.

Information on how to calculate the LEL and COV from existing test data is given in Chapter 4 and the related appendices. Typical values of the LEL and COV_R for different components used in transmission and distribution line are also given in Chapter 4. If no data are available, the following default information may be used:

- 1. Steel Components and Prestressed Concrete Poles.** For components of steel towers and steel or prestressed concrete pole structures designed according to the ASCE and Precast/Prestressed Concrete Institute (PCI) publications (ASCE 1990; ASCE 1992; PCI 1997; ASCE 2000; PCI, 1999), assume that R_n (or R_e) has an exclusion limit in the

range of 5% to 10% and COV_R is in the range of 10% to 20%. Therefore, from Table 2-2, ϕ is in the range of 0.96 to 1.12 or, typically, $\phi = 1.0$.

2. **Reinforced Concrete.** For reinforced (non-prestressed) concrete components designed according to the American Concrete Institute (ACI) procedures (ACI 2002), the ACI strength reduction factors can be used in lieu of the ϕ factors given in Table 2-2, since the ACI factors already contain strength derating effects for the various concrete components.
3. **Wood Poles.** For wood pole structures, the statistical data in *ANSI O5.1-2002* (ANSI 2002) can be used to determine a value for R_n at the 5% LEL by using Eq. 4-4 in Chapter 4. The corresponding strength factor can then be obtained from Table 2-2. Alternatively, the COV_R and 5% LTL may be computed from actual data and the corresponding strength factor obtained from Table 2-2, for $e = 5\%$.
4. **Foundations.** Table 2-2 can be used for foundations for which statistical data are available. For foundations for which statistical data are not available, nominal strengths and strength factors based on established practice can be used. In such cases, however, the reliability of the foundation relative to that of the supported structure is unknown.
5. **Conductors and Ground Wires.** The LRFD format described herein is also applicable to the mechanical design of conductors or ground wires. If the nominal strength R_n for a conductor is defined as its rated ultimate strength T_u (AAI 1982), then a strength factor of 0.6 to 0.7 is recommended. Using a strength factor of 0.60 to 0.70 should prevent damage to the conductor and virtually eliminate the possibility of its rupture at that level. These suggestions are not reliability-based but represent current practice.

A summary of the RBD procedure is given in Fig. 2-1.

2.7 MOMENT MAGNIFICATION CONSIDERATION FOR FLEXIBLE POLES

Slender poles may have significant secondary moments that need to be considered in the design. These moments are caused by vertical loads acting through a horizontal displacement near the top of a deflected pole. These magnified moments may be obtained using an iterative calculation, using an amplification factor such as the Gere-Carter method (Appendix A, Example Calculation of P - Δ Effect Using the Gere-Carter Method) or using a computer program that accounts for geometric nonlinearity.

As a result of the reliability calibrations that included designing typical poles at 40 locations around the United States (described at the beginning of this chapter), the recommended load factors in this manual are less than or equal to 1.0 (Table 2-3). The ASCE Pole RBD Committee recommends

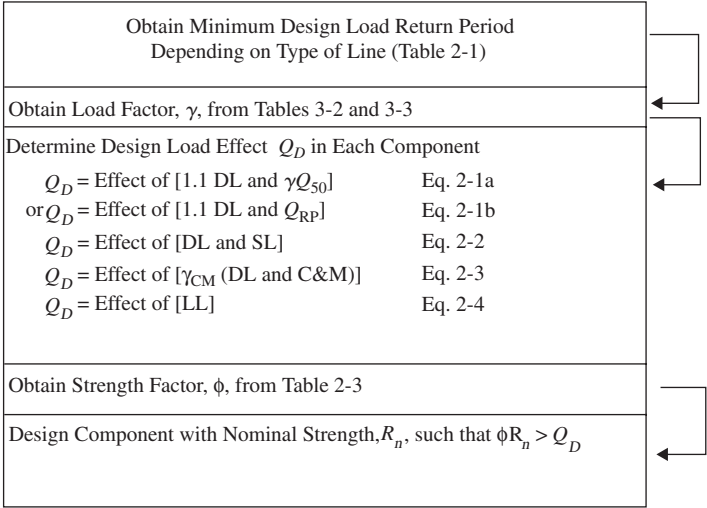


FIGURE 2-1. Summary of Reliability-Based LRFD Design Procedures.

that factored loads should be used in calculating secondary moments ($P-\Delta$ effects). The Gere-Carter method is typically conservative and is illustrated in Appendix A. From the reliability calibrations used to evaluate hundreds of poles, researchers determined that the moment magnification factors ranged from 1.0 to over 2.0, depending on the pole geometry, materials, load configuration, and load magnitude.

2.8 COORDINATION OF FAILURE SEQUENCES

There is a natural and well-founded desire to exercise some control over the sequence of failure of the different line components when one item fails. A casual event that cannot be avoided or a weather-related event that exceeds the selected RP load can, if not controlled, lead to unnecessary and excessive damage that will prolong the outage.

2.8.1 Structures versus Foundations

The components of the support system do require some attention regarding their relative strengths. In some cases, it may be desirable to design foundations that are generally more reliable than the structures they support. The rigorous approach to arranging this relative relationship would require knowledge of the dispersion characteristics PDFs and variability (COV_R) of these two major line components. By using their COV_R or using default

values such as those suggested here, it is possible to adjust the calculated strength of the foundations by applying a strength factor based on a LEL of 1% and using a factor based on a LEL of 10% for the structures. The failure rate of a LEL of 1% is theoretically equal to 1%, which is not significant in itself. The relative reliabilities of using a LEL of 1% and a LEL of 10% are, however, meaningful numbers that can be used in design. If the goal is to design foundations to be more reliable than the structures, then use a LEL of 1% for the foundations with the structures at a LEL of 10% (or a LEL of 0.1% for foundations and a LEL of 1% for structures). This will provide sufficient probability that the structure will fail before the foundation.

2.8.2 Wire System versus Support System

Structural failure of a component in the wire systems (wires, insulators, hardware) puts critical demands on the failure containment properties of the support system, especially if the threat is from heavy icing and the wires are highly tensioned. Fortunately, many of the wire system components are selected on the basis of their damage limits, usually from 50% to 70% of their rated strengths, so that the remaining strength margins (50% to 30% of their *nominal* strength) should ensure that they will not be the first to fail. For insulators, load duration effects may reduce the allowable design loads to a significantly lower level than their rated strengths, which should be taken into consideration in the design and selection of the insulator system. All of these components should not be part of the critical failure path, since all of the components of the wire systems are governed by their damage limits.

2.8.3 Tangent versus Dead-End Structures

There should also be a hierarchy of strengths of the different classes of structures with the tangent structures considered the base units, whereas heavy-angle and dead-end structures should have an extra margin of strength. Because this calibration study was restricted to tangent structures, line tension effects were not considered at this time. This edition assumes that insulators and hardware are designed so that they are not the weak links in the line. Future editions of this manual will consider angle and dead-end structures, as well as insulators and hardware.

The current edition of this manual addresses the reliability of a single pole subjected to a severe storm. It is outside the scope of this current edition to consider the reliability of an entire line comprising a group of poles subjected to a severe storm, including the reliability of the wires, insulators, hardware, and foundations. The reliability of a line is less than that of an individual pole, and techniques to evaluate line reliability are discussed in Dagher et al. (1993) and IEC 60826 (IEC 2002).