Reliability of Poles in NESC Grade C Construction

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Abstract

The reliabilities of steel and wood poles designed according to the 1997 NESC provisions for Grade C construction were evaluated. It was found that the reliability of steel poles is significantly higher than that of wood poles. Reasons for the discrepancy relate to the historical development of the deterministic design approach used in the NESC specifications. To correct the discrepancy, fundamental principles of structural reliability were used to develop modified load factors (LF) and strength factors (SF), applicable to all materials under grade C construction. A load factor LF=1.75 is proposed for all materials, and a table of strength factors are proposed. The strength factors are a function of the coefficient of variation of the pole strength, and the lower exclusion limit corresponding to the nominal design strength value. As an interim fix, it is proposed that the modified load and strength factors for grade C construction be allowed as an alternate approach in the NESC. Ultimately a complete rewrite of the NESC to incorporate a reliability-based LRFD (Load and Resistance Factor Design) approach will be necessary to guarantee consistent safety levels across all materials and structure types.

Introduction

Prevailing US practice and most state laws require that transmission lines be designed, with a minimum, to meet requirements of recent editions of the National Electric Safety Code (NESC) (1). The NESC rules for the selection of design loads and load factors are largely based on successful experience, but they do not have a strong theoretical foundation. Some designers find the rules too restrictive, while others adopt more conservative criteria. In fact, individual utilities often develop their own loading agendas to complement the NESC rules. A desire to achieve more consistent reliabilities across materials was the impetus for the development of the reliability-based Load and

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Resistance Factor Design (LRFD) method published in the ASCE Guidelines for Transmission Line Structural Loading (2).

The need for fundamental changes in the NESC is perhaps best illustrated by the recent controversy surrounding load and strength factors for wood and steel distribution poles. A number of change proposals were submitted to NESC Task Force 5.1.4 of ACS C2 to achieve equivalent and consistent levels of safety between wood and steel poles for Grade B and C construction. The proposals were accompanied with significant discussion and correspondence on this subject among members of American Iron and Steel Institute (AISI) and the North American Wood Pole Coalition (NAWPC). A number of position papers were written regarding the subject from the wood and steel pole sides, as summarized in Table 1.

TABLE 1. DOCUMENTS DISCUSSING STRENGTH AND LOAD FACTORS FOR STEEL AND WOOD POLES

Date	Author	Item/Title	Presented to
1997	Dick Aichinger (AISI)	The Need for NESC Loading and Strength Revision	NESC Subcommittee 5
1998		NESC Change Proposals	Task Force 5.1.4 of ACS C2
6/98	NAWPC	Providing Equity in the NESC for Wood and Steel Pole Construction	
9/98	Ron Randle (EDM)	NESC Discussion on Reliability-Based Design	Letter to Western Wood Preservers Institute
10/98	Martin Rollins (Consultant NAWPC)	NESC White Paper	
10/98	AISI	AISI Response to the North American Wood Pole Coalition	
10/98	Bob Peters	Wood Poles, Steel Poles, and the NESC	Letter to the NAWPC
10/98	Dr. H. Dagher (Univ. of Maine)	Evaluation of AISI Proposed Changes and Mr. Rollins' White Paper	NESC Subcommittee 5 on October 20, 98
3/99	Dr. H. Dagher (Univ. of Maine)	Reliability-Based Load and Strength Factors for Steel Poles in NESC Grade C Construction	AISI, IUSI

The documents and correspondence in Table 1 contain many good arguments on both sides of the issue. To illustrate the problem, consider the current NESC load and strength factors in Table 2.

TABLE 2. 1997 NESC- TRANSVERSE WIND - POLE NOT AT CROSSING

		STEEL			WOOD	
	LF	SF	LF/SF	LF	SF	LF/SF
Grade B	2.5	1	2.5	2.5	0.65	3.85
Grade C	2.2	1	2.2	1.75	0.85	2.06
Grade C / Grade B			88%			53.5%

LF = Load Factor *SF = Strength Reduction Factor*

The following observations may be made:

1. For grade C, the overall safety factor (LF/SF) is less for wood (2.06) than it is for steel (2.2). For grade B, the opposite is true: The overall safety factor for wood (3.85) is more than for steel (2.5).

2. For steel, Grade C poles have 88% of the “capacity” of Grade B poles. In contrast, for wood, Grade C poles have only 53.5% of the “capacity” of Grade B poles. Therefore, it seems that wood strength is significantly smaller in Grade C than in Grade B, as compared to steel.

To correct the apparent discrepancy in steel and wood pole safety factors, AISI proposed NESC Grade C construction Overall Safety Factors (OSF) of 1.3 and 2.08 for steel and wood, respectively (see Table 3 and Reference 3). The ratio 1.3/2.08 = 0.625 was selected by AISI in order to maintain the 2.5/4 = 0.625 ratio between the OSF of steel and wood for Grade B construction in the 1993 NESC.

The AISI intention to correct the problem is well founded; however the factors proposed in Table 3 are not based on structural reliability principles. To address the problem, one should seek equivalent reliability levels between steel and wood poles within each grade of construction. This can be done by using well-established principles of structural reliability (4-12).

TABLE 3. CURRENT AND AISI-PROPOSED NESC FACTORS FOR GRADE C CONSTRUCTION (3)

Load	1997 NESC						Task Group 5.1.4 Change Proposal				
	LF		SRF		OSF= LF/SF		LF	SRF		OSF= LF/SF	
	Steel	Wood	Steel	Wood	Steel	Wood	Both	Steel	Wood	Steel	Wood
Vertical	1.5	1.9	1.0	0.85	1.5	2.24	1.5	1.0	0.625	1.5	2.4
Tr.Wind	2.2	1.75	1.0	0.85	2.2	2.06	1.3	1.0	0.625	1.3	2.08
Tension	1.1	1.3	1.0	0.85	1.1	1.53	1.1	1.0	0.625	1.1	1.76

The NAWPC contracted with Mr. H. M. Rollins to address the problem using structural reliability principles. Mr. Rollins' work resulted in a white paper, which was presented to the NESC Task Force 5.1.4 on two occasions in 1998 (13). The NAWPC's effort was a step forward since it advocated finding a solution using established principles of probabilistic mechanics. However, the reliability analysis described in reference (13) failed to include uncertainties in the loads.

Objective

The objectives of this paper are to

(1) evaluate the reliabilities of steel and wood poles under NESC grade C construction, and if necessary

(2) to propose an interim measure to reduce some of the discrepancies in safety levels between all pole materials under Grade C construction. It is important to emphasize that any modified load and strength factors developed here do not provide a substitute for a future reliability-based LRFD method in the NESC.

Basic Principles

The following basic principles were used in this study:

1. **Establish one Target Reliability Level for all Materials under Grade C.** It was necessary to establish a reference or target reliability level for a Grade C Construction pole independent of the material used. We used as reference the reliability level of wood poles designed for transverse wind loading according to the NESC Grade C

construction. This is based on many years of experience with wood pole performance. Wood poles are currently designed with a load factor of 1.75 and a strength factor of 0.85. Hence, we sought to develop load and strength factors for steel poles so that their reliability will match that of wood poles.

2. **Modify the OSF for Transverse Wind only**. Referring to Table 3, AISI proposed to change the steel Overall Safety Factor (OSF) for the transverse wind condition only, from 2.2 to 1.3. The OSF remains at 1.5 and 1.1 for vertical loads and transverse tension loads, respectively. Therefore, a focus of this study is on whether we can justify, based on structural reliability principles, a reduction in the OSF for steel from 2.2 to 1.3 for Transverse Wind. If not, what reduced OSF can be justified?

3. **All Reliability Calculations are Based on Initial Strength.** The reliability of steel poles (or poles made of any other material) will be matched with that of wood poles when first installed. In other words, a Grade C construction pole whether steel or wood or any other material will have approximately equivalent 'as-installed' reliability. It is recognized that all materials are susceptible to environmental degradation in service. The rates of degradation are complex functions of the materials, the environment, the foundation conditions, and the quality of the pressure treatment or corrosion protection methods used. The degradation processes for various pole materials are such that it is very difficult to develop accurate predictive models that can be used to modify design strength factors. The conflicting arguments on the degradation rates and their consequences on pole strength in the publications of Table 1 serve to reinforce this point. It is selected to move away from the idea of prescribing reduced NESC strength factors to account for unknown degradation rates. The NESC should instead make reference to inspecting, maintaining and replacing not only wood poles but all poles when their strength is reduced below a specified percentage of their installed strength.

4. **Select one Load Factor for all Materials**. Load factors (LF) account for the

uncertainties in our ability to predict extreme loads and load effects. Therefore, the same load factors will be used for all materials.

5. **Strength Factors will be based on Statistical Properties of Pole Strength**. The strength or resistance factors (SF) varies from one pole material to another. This is due to two major reasons. First, in the NESC steel and wood pole designs, safety factors are not applied to the same nominal design quantity. For wood poles, the ANSI 05.1-1992 publishes a mean ultimate bending strength (14). However, the design, material and testing requirements in ASCE Manual 74 for steel poles (15) has been thought to result in pole nominal bending strength values closer to the 5^{th}-20^{th} percentile (5 %-20% Lower Exclusion Limit, LEL) (8). This is important because a larger strength reduction factor is required for a mean-based value than a 5% LEL value. Second, safety factors do not have the same effect if applied to materials with different variabilities. The bending strengths of wood and steel poles have a coefficient of variation (COV) of approximately 16.9% and 8.7%, respectively (5,6). These numbers were used in this paper for consistency with the NAWPC report.

Probability of Failure of Wood and Steel Poles Designed using the 1997 NESC.

In this section, the relative reliability levels of steel and wood poles designed according to the 1997 NESC Grade C are estimated. This is necessary to justify any changes in the current safety factors. The focus is on poles subjected to transverse wind loading since the largest inconsistency between steel and wood poles appears to be under this load condition. AISI proposed reducing the overall safety factors (OSF) for steel poles under this condition from 2.2 to 1.3.

Properly estimating the reliability of a transmission pole structure is a complex problem. It requires knowledge of the joint PDF of the load-producing events such as ice, wind, temperature and wind direction, the PDF of the strength properties of the pole components, and the evaluation of multiple correlated failure modes including bending, compression, local and global buckling, moment amplification, connections failures, and foundation failures. In addition, evaluating the reliability over time requires good models of the physical and chemical degradation mechanisms.

Because of the lack of data on load, strengths, and degradation mechanisms, the probabilities of failure given here should be only viewed as estimates. In this report, they are only used as *relative* measures of the reliabilities of wood and steel poles.

Statistical Models for Load-Producing Events

The nature of events which produce loads in a transmission or distribution line may be used to classify these loads in 3 broad categories. Category I includes loads produced by climatic phenomena which can be described by probability laws. Loads produced by extreme winds and, where data exist, loads produced by severe combination of ice and wind fall into Category I. Category II includes loads produced by natural or accidental events which, because of lack of data, cannot be described statistically. Examples of loads in Category II are those produced by unbalanced ice or by failures of structures or components because of defects, wear, fatigue, landslides, earthquakes, sabotage, etc. Finally, Category III includes loads produced by construction or maintenance operations. Theoretically, these loads may be well predicted if one assumes that future operations will be performed according to strict guidelines. Like the loads in Category II, the loads in Category III cannot be described by a probability law.

Therefore, out of the many load cases normally considered in design, only those in Category I, i.e., loads produced by extreme wind and loads produced by the combination of ice and wind, can be considered in a formal reliability-based design procedure. It is interesting to note that the two load cases emphasized by the NESC are precisely those load cases which can be handled by the reliability approach. In keeping with the RBD design philosophy of the ASCE Loading Guide (2), the reliability analysis in this report did not consider loads in Categories II and III. These loads should be considered under line safety and security design requirements, as opposed to line reliability (2). For example, for loads in Category II, a designer must make sure that if a failure is triggered by an accidental event such as a vehicle impact, it will not propagate without control (2).

In summary, the reliability analysis of wood poles in this report was restricted to extreme wind loading. The reliabilities of wood and steel poles designed according to the 1997 NESC Grade C requirements were evaluated assuming that the maximum annual wind velocity follows an Extreme Type I (EXI) or Gumbel distribution. Since there is a relatively small dead load component, it was assumed that the maximum load effect in the pole Q is approximately proportional to the square of the wind velocity V:

$$Q = (\text{constant})\ V^2 \qquad [1]$$

$$V = EXI\ [a,u] \qquad [2]$$

It was assumed that the COV of the max annual wind velocity at a site can vary in the range of 10% to 30 % (4). It was also assumed that the extreme wind, which is measured regardless of direction, is applied perpendicular to the line. This assumption yields a conservative estimate of the probability of failure (4).

Statistical Models for Pole Strength

The ultimate bending strength R_s and R_w of both steel and wood poles were assumed to follow a Normal distribution with the properties given below (5,6).

Wood poles: $R_w = N\ [m_{Rw}, 0.169]$

The nominal design strength R_{wn} using ANSI 05.1 (14) is a mean-based value, with an LEL of 50%. The 16.9% COV and 50% LEL are representative for wood poles and they allow direct comparison of the results with Mr. Rollins' work (13).

Steel poles: $R_s = N\ [m_{Rs}, 0.087]$

The LEL corresponding to the nominal design strength R_{sn} using ASCE Manual 72 (15) has been generally assumed to be in the order of 5-20%. However, this assumption has not been verified in the published literature using full-scale test data. The 5-20% LEL assumption is essentially based on known statistical properties of steel: variability of the stiffness E, the yield strength F_y and wall thickness tolerances using mill test reports.

To verify the assumption on the LEL for steel poles, we have obtained six sets of full-scale test data on steel poles from three different sources:

1. Twenty-seven full-scale steel pole tests to failure conducted on behalf of IUSI (International Utility Structures Inc.) by an independent testing laboratory at three different occasions). Poles within each of the three sets of tests varied in length, but had the same nominal design strength. In all cases the steel poles tested above their design loads (Table 4).
2. Five full-scale tests to failure conducted by EDF (Electricite de France) on steel distribution poles manufactured by Petitjean, an IUSI subsidiary. Each of the five poles had different design strength, which were exceeded during testing (Table 4).
3. One-hundred proof load tests on tubular steel lighting poles by Valmont industries. These poles are manufactured using the same process and materials as the poles for the distribution market. The proof load tests were carried out to the nominal design load, and not to failure of the poles. In all 100 tests, all poles achieved the nominal design load.

The results of the full-scale tests summarized in Table 4 show that the LEL for these six sets of steel poles is less than 1%. For the 27 IUSI poles, the LEL is at most 0.003%, which is significantly less than 1%. The reasons for this may be explained by mill test reports of actual thickness and yield properties of the steel, which are higher than the minimum specified values. In order to take advantage of the low LEL on a continuing basis, one must insure that the steel will always be delivered with thickness and yield values higher than the specified requirements. This is currently achieved because structural steel is ordered to ASTM standards, which specify required thickness and yield. Steel suppliers exceed these standards to reduce the likelihood of rejects. In addition, one needs to verify that the quality of the manufacturing processes is consistent with that used to manufacture the poles listed in Table 4. If a Q/C program is implemented that verifies mill test reports on actual steel thickness and yield stresses, combined with a continuing full-scale test program, one should be able to take advantage of low LEL in design.

TABLE 4 STATISTICAL PROPERTIES FROM FULL-SCALE TUBULAR STEEL POLE TEST RESULTS

Test Series	Sample Size	Pole Length (ft)	Actual Strength		Design Strength (lbs)	Actual/ Design	LEL
			Mean (lbs)	COV			
IUSI -1	7	45-70	3,625	7.7%	2,312.5	1.57	<0.003%
IUSI -2	10	40-55	2,903	9.3%	1,875	1.55	0.007%
IUSI -3	10	25-45	1,860	8.7%	1,187.5	1.57	<0.003%
EDF	5	33-50	___[1]	___[1]	___[1]	1.85	<0.003%
Valmont	100	32-34.5	___[2]	___[2]	___	___[2]	< 1%[3]

[1] *Cannot report because each of the five poles has a different design strength*

[2] *Information not available because it is a proof load test*

[3] *Since this is a proof load test, the actual LEL is unknown except that it is less than 1%*

Structural Reliability Analysis

In the most general sense, a reliability analysis of any pole should consider all possible failure modes. Failure at any location along the pole can be triggered by axial tension or compression stresses, combined compression/bending stresses, combined shear/torsion stresses, combined shear/normal stresses, moment magnification effects, global buckling, local buckling for steel poles, weld failures, base plates and anchor bolt failures and foundation failures.

In a structural reliability evaluation, each possible failure mode can be expressed in the form of a g[.] function or failure function. The g function is an explicit or implicit function of many random variables such as wind, ice, wind direction, temperature, mechanical properties and pole dimension. In the simplest case, g is a function of two random variables, the strength R and the load Q.

$$g = R - Q \qquad [3]$$

If g <0 the component fails. If g >0 the component survives. A more complex g-function could for example evaluate the combined shear and normal stresses in a steel pole using the distortion-energy (Henky-Mises) yield criterion (15).

$$g = F_y^2 - [P/A + M_x c_y/I_x + M_y c_x/I_y]^2 - 3 * [VQ/It + TC/J]^2 \qquad [4]$$

In which P, M, V, T are themselves functions of other random variables like wind, ice, temperature, wind direction and the geometry and material properties of the structure. Additional background on conducting reliability analyses of transmission structures may be found in references (4-12).

In this report, it was assumed that the multiple failure modes of the pole shaft in bending can be approximately represented by a single Normal distribution so that the g function of Equation [3] applies. The probability of failure of the pole in bending was evaluated both by direct integration and by Monte Carlo simulations, depending on the loading case and the computational effort involved. The computer programs used in the Monte Carlo simulations are listed in the Appendix.

Results of Reliability Analyses of NESC Grade C Construction Designs

Results of the reliability evaluation of the NESC steel and wood pole designs are given in Table 5. The last column of the Table provides the ratios of the probabilities of failure (P_f) of wood and steel poles, assuming the steel pole nominal strength is at the 5% LEL. The 5% LEL is a conservative value compared to the numbers in Table 4.

TABLE 5. ANNUAL PROBABILITY OF FAILURE. MAX ANNUAL WIND SPEED HAS GUMBEL DISTRIBUTION, STEEL POLE LEL=5%

COV of Wind Speed	Steel Poles[1] Grade C	Wood Poles[2] Grade C	Ratio P_f Wood/Steel
0.10	0.000003	0.0002	33
0.20	0.000085 10^{-5}	0.0007	8
0.30	0.00023	0.0014	6

1. *Design LF= 2.2 SF=1. Nominal strength = 5% LEL ; COV strength = 8.7%*
2. *Design LF= 1.75 SF=0.85 Nominal strength = mean value ; COV strength = 16.9%*

<u>Table 5 show that regardless of the load model used to evaluate the reliability of grade C</u>

construction, the 1997 NESC appears to yield significantly and consistently more reliable steel poles than wood poles. The probabilities of failure of the wood poles are between 6 and 33 times those of the equivalent Grade C steel poles. The difference will be even greater if an LEL < 5% can be demonstrated for the strength of the steel poles. Once again, this is looking at initial reliabilities before material degradation occurs for either steel or wood.

This information appears to correspond to some field observations[2] of pole failures during hurricane and high wind events for the 1998 hurricane Georges in Puerto Rico, and 1997 summer storms and tornadoes in Minnesota. In both events, a disproportionately high number of wood poles were reported to have failed, as compared to steel poles likely subjected to similar forces. In hurricane George, 8,540 wood poles were reported damaged or failed by PREPA (Puerto Rico Electric Power Utility), but none of 1000 steel distribution poles installed before the hurricane failed. On July 1, 1997 a storm with sustained winds up to 120 mph hit Monticello and Big Lake, Minnesota. Anoka electric had steel and wood lines in the area, but only the wood lines sustained severe damage. More information is needed to properly evaluate this data and other similar data including the age and class of the steel and wood poles, and the fraction of poles that have failed in each age and class category.

Table 5 stresses the importance of evaluating the NESC designs using structural reliability principles. There appears to be ample evidence that a thorough evaluation of the NESC load factors for steel and wood pole needs to be undertaken.

Modified Load and Strength Factors for Steel Poles

It was demonstrated that the reliability of steel poles in NESC grade C construction was significantly higher than that of wood poles. Therefore, it is possible to reduce the overall safety factor OSF = LF/SF for steel poles in grade C construction. This will reduce the reliability of steel poles down to the current level for wood poles in the 1997 NESC.

Assuming that it is desirable to use the same LF=1.75 for both steel and wood, let us calculate the strength factor for steel ϕ_σ which will result in more consistent reliabilities between the two materials.

The target annual probabilities of failure for a steel pole are those in the third column of Table 5. These target values depend on the COV of the wind velocity. A 1.75 load factor was selected for both steel and wood poles. What is required is the value of ϕ_s which will yield the target values. Monte Carlo Simulations (See Appendix) were used to estimate the probabilities of failure for different values of ϕ_s, different COV of the wind speed, and different LEL and COV for the poles. Referring to Figure 1, the probability of failure of a steel pole (or any pole) designed with a LF=1.75 was obtained using the following methodology:

1. Select an arbitrary value for the mean of the annual extreme wind velocity m_V
2. Select a value for the strength reduction factor ϕ_s
3. Select a value of COV of wind velocity V_V = 10, 15, 20, 25, 30%
4. Calculate parameters of wind velocity distribution V= EXI[a,u]:

   $a= 1.282/(m_V * V_v)$ $\quad$ $u= m_V-0.577/a$
5. Calculate V_{50}= 50-year RP wind velocity = $m_V (1+2.594\ V_v)$
6. Calculate $Q_{50} = k\ V_{50}^2$ in which k is an arbitrary constant
7. Calculate parameters of the pole strength $R_s=N[m_{Rs}, s_{Rs}]$

   $m_{Rs}= 1.75\ Q_{50}/\phi_s (1-1.645*V_{Rs})$ (see Figure 1)

   $s_{Rs} = 0.087\ m_{Rs}$
8. Simulate g-function $g= R_s – k\ V^2$, check convergence, and obtain P_f

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[2] This information was obtained in the form of letters from PREPA and IUSI

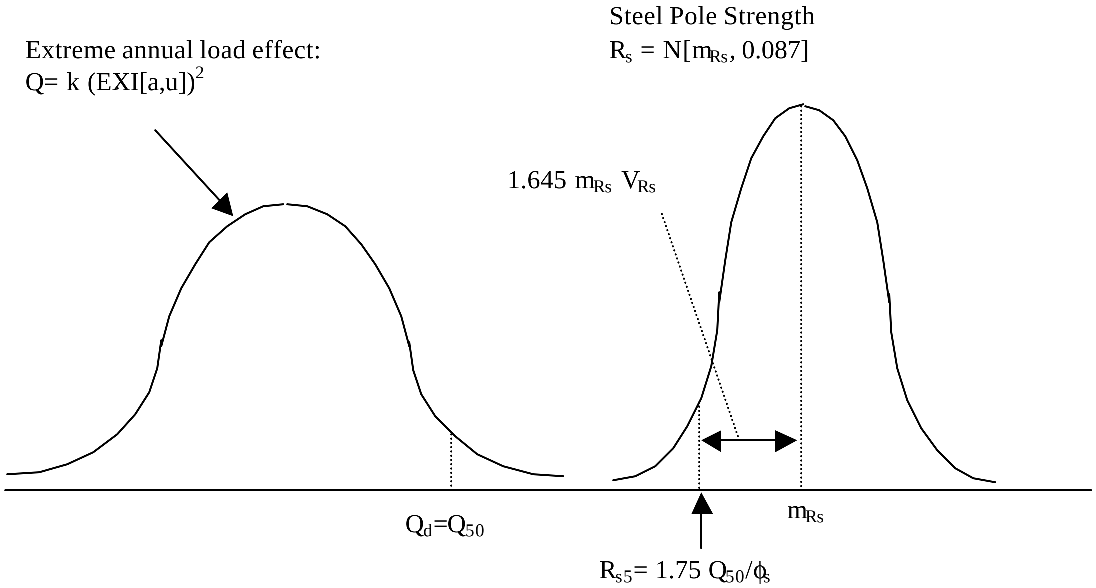

FIGURE 1. LOAD AND STRENGTH RELATIONSHIPS FOR STEEL POLES

A Monte Carlo simulation computer program that embodies these concepts is shown in Appendix A. The Appendix lists the source code for the program, which was slightly modified for each run conducted. It was found that a strength factor of 1.08 for steel along with a load factor of 1.75 yields consistent probabilities of failure between wood and steel poles. The results are summarized in Table 6.

TABLE 6 PROBABILITIES OF FAILURE OF STEEL AND WOOD POLES DESIGNED USING PROPOSED LOAD AND STRENGTH FACTORS FOR A STEEL POLE LEL=5%

COV of Wind Speed	P_f Steel Poles[1] Grade C	P_f Wood Poles[2] Grade C	Ratio P_f Wood/Steel
0.10	0.0001	0.0002	2
0.15	0.0003	0.0004	1.3
0.20	0.0007	0.0007	1
0.25	0.0010	0.0010	1
0.30	0.0014	0.0014	1

1. *Design LF= 1.75 SF=1.08 Nominal strength = 5% LEL ; COV strength = 8.7%*
2. *Design LF= 1.75 SF=0.85 Nominal strength = mean value ; COV strength = 16.9%*

Assuming an LEL=5% for steel pole strength, an examination of Table 6 shows that a Load Factor of 1.75 along with a strength factor of 1.08 for steel poles maintains a steel pole probability of failure P_{fs} equal to or less than that of a wood pole P_{fw}. The ratio $P_{fw}/P_{fs} = 1$ for a wind speed COV>0.20. For a wind speed COV< 0.20 the ratio P_{fw}/P_{fs} is larger than 1 indicating a larger reliability for steel poles.

As discussed earlier, it is possible for steel poles, or poles of any other material to have an LEL less than 1%. The data in Table 4 shows that an LEL< 0.003% has been achieved for steel poles, under the proper circumstances. Table 7 has been developed in accordance with this. The Table offers strength factors for poles made from any material, assuming the manufacturer can demonstrate and maintain the values of the LEL and COV of the pole strength. Equivalent reliabilities among as-installed poles of all materials for grade C construction can be achieved by

(1) Utilizing a wind Load Factor of 1.75 for all materials, and
(2) Employing the strength factors (SF) shown in Table 7. These strength factors insure that all materials have the same reliability as that of Grade C wood poles.

The SF factors in Table 7 were derived using fundamental principles of structural reliability, using the computer program shown in the Appendix. Essentially, the values of SF were modified through trial and error until the reliability of the pole was made equivalent to that of a grade C construction wood pole.

Conclusions

The objectives of this paper were to evaluate the reliabilities of steel and wood poles under NESC grade C construction, and if necessary to propose an interim measure to reduce some of the discrepancies in safety levels between wood and steel poles under Grade C construction. The study utilized fundamental principles of structural reliability while working within the context of the NESC design methodology. The following conclusions were reached:

1. To correct a discrepancy in steel and wood pole reliabilities, AISI proposed Overall Safety Factors (OSF) of 1.3 and 2.08 for steel and wood, respectively under NESC Grade C construction. The AISI intention to correct the problem was well founded; however the factors proposed by AISI were not based on structural reliability principles. As summarized below, it was found that the OSF for steel can be significantly reduced, depending on the LEL for the nominal strength of the steel poles.

2. The probabilities of failure of steel and wood poles designed according to the 1997 NESC were evaluated using structural reliability methods. It was found that <u>the 1997 NESC appears to yield significantly and consistently more reliable steel pole designs than wood poles under grade C construction</u>. With the current situation, and assuming a 5% LEL for the nominal strength of steel poles (a conservative value according to the tests in Table 4), the calculated probabilities of failure of the wood poles are between 6 and 33 times that of the equivalent Grade C steel poles. This information appears to correspond to some field observations of pole failures during hurricane and high wind events.

3. Equivalent reliabilities among as-installed poles of all materials for grade C construction can be achieved by (1) utilizing a wind Load Factor of 1.75 for all materials, and (2) employing the strength factors (SF) shown in Table 7. These strength factors ensure that all poles made of any material have the same reliability as that of Grade C wood poles. The factors in Table 7 were derived using fundamental principles of structural reliability. The NESC may wish to incorporate Table 7 as an alternate method to obtain the strength factor for grade C construction.

TABLE 7. ALTERNATE SF FOR GRADE C CONSTRUCTION, ALL MATERIALS (LF = 1.75)

COV of Pole Strength	Fraction of Poles with Capacity less than their Nominal Strength LEL = Lower Exclusion Limit of Nominal Strength						
	1/3000 0.033%	1/2000 0.05%	1/1000 0.1%	1/100 1%	1/20 5%	1/10 10%	1/2 50%[2]
5%	1.15	1.14	1.13	1.08	1.04	1.02	0.96
8.7% (steel)	1.31	1.29	1.26	1.16	1.08	1.04	0.92
13%	1.57	1.53	1.47	1.26	1.12	1.00	0.88
16.9% (wood)[1]	2.00	1.91	1.77	1.39	1.17	1.08	0.85
21%	2.63	2.42	2.13	1.46	1.14	1.02	0.75

[1] *Corresponds to COV of Wood Pole strength as defined in ANSI 05.1*

[2] *Corresponds to LEL of wood pole nominal strength as defined in ANSI 05.1*

4. In order to use the factors in Table 7, it is important to be able to guarantee the LEL of the steel pole (or any pole's) nominal strength. For example, the data from IUSI steel pole tests (27 poles tested to failure) indicates that the lower exclusion limit (LEL) for those tests is significantly less than 1 in 3000 (see Table 4). Also, proof load tests from Valmont steel poles show no failure in over 100 tests, indicating that the LEL is below 1%. It is therefore warranted to propose significant changes in NESC load factors for steel, assuming that the LEL and COV for the nominal steel pole strength can be guaranteed by the manufacturer. This can be achieved by implementing a routine full-scale test program, supplemented by mill reports certifying the minimum requirements of the steel supplied. If an LEL of 1 in 3000 can be guaranteed for steel poles, then the SF from Table 7 is 1.31, with a corresponding OSF of 1.34 (1.75/1.31).
5. The proposed load and strength factor changes for all pole materials as described under bullets (3) and (4) are summarized in Table 8.

TABLE 8. PROPOSED NESC FACTORS FOR GRADE C POLE CONSTRUCTION , ALL MATERIALS

Load	Proposed Values		
	LF	SF	OSF
Tr. Wind	**1.75**[1]	**Table 7**	**1.75/SF**

[1]*The bold font style is used to indicate numbers that have changed under this proposal*

Finally, it is important to emphasize that the best way to provide uniform reliability across all materials is to utilize a reliability-based Load and Resistance Factor Design (LRFD) methodology within the NESC. To reach this goal requires implementing fundamental changes to the NESC which were outside the scope of this study. The alternate load and strength factors proposed herein provide an intermediate step to improve reliability consistency across materials, while minimizing the impact on the existing NESC design methodology.

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APPENDIX

Listing of Pole Reliability Analysis Computer Program

```
140 'Annual Probability of failure of pole designed with a load factor
141 'LF,'strength factor SF
143 'Applied load = extreme annual wind = (Extreme Type 1
      Distribution)^2
144 'Pole strength = NORMAL with parameters XMEAN and XCOV
145 'Pole nominal strength exclusion limit e = CDF[-KS]
146 ' typical values of KS: for e= 50% (1/2) KS = 0
147 ' for e=5% (1/20) KS=1.645, e=1% KS=2.32, e=.1% (1/1,000) KS=3.09,
148 ' for e=.05% (1/2,000) KS=3.29,  e=.01% (1/10,000) KS = 3.72
149 '
150 'main
160 '
180 INPUT "WHAT IS THE NUMBER OF SIMULATIONS"; NSIM
190 RANDOMIZE (TIMER)
200 NF = 0: PF = 0: LF = 1.75: SF = 1.14: KS = 1.645  'KS # STDD DEV
      below mean
201 '
202 'parameters of load distribution
203 VMEAN = 50: VCOV = .1    'input mean and COV of max annual wind
      velocity
204 A = 1.282 / (VMEAN * VCOV): u = VMEAN - (.577 / A)'a, u param. EXI
      distrib.
205 V50 = VMEAN * (1 + 2.594 * VCOV): Q50 = (V50) ^ 2'50-year RP
      velocity & load
206 '
207 'parameters of pole strength distribution RNORM(XMEAN,STD)
208 XCOV = .21  'XCOV = COV of pole strength
209 XMEAN = LF * Q50 / (SF * (1 - KS * XCOV))'SF=strength factor
210 STD = XCOV * XMEAN    ' assume 8.7% COV of steel pole strength,
      16.9% wood
211 '
212 'probability of failure of pole
213 '
218 FOR I = 1 TO NSIM
220   GOSUB 340   'compute a value of RNORM = strength of a pole
250   GOSUB 440   'compute a value of EXI = maximum annual wind speed
252   Q = 1! * (EXI) ^ 2 'Q= maximum applied annual wind force
260   IF Q >= RNORM THEN NF = NF + 1
290 NEXT I
300 PF = NF / NSIM
305 GOSUB 570 'print results
310 END
320 '
340 '
350 'generate random deviates RNORM(XMEAN,STD) from a normal
      distribution
360 '
370 SUM = 0!
380 FOR K = 1 TO 12
390   SUM = SUM + RND
400 NEXT K
```

```
410 RNORM = XMEAN + STD * (SUM - 6!)
420 RETURN
430 '
440 'Generate value for max. annual wind speed using EXI(a,u)
543 'Generate random deviates from an extreme type I distribution
544 ' using inverse transform method
545 '
546 EXI = u - LOG(LOG(1! / RND)) / A
547 RETURN
549 '
570 'PRINT RESULTS
580 '
590 PRINT "MONTE CARLO ANALYSIS OF POLE RELIABILITY"
600 PRINT " "
605 PRINT "SF = "; SF; "    LF = "; LF; "   KS Stdd Dev below Mean = ";
KS
607 PRINT "Overall safety factor OSF = "; LF / SF
610 PRINT "STRENGTH = N["; XMEAN; ", "; XCOV; "]; "
620 PRINT "WIND VELOCITY = EXI["; VMEAN; ","; VCOV; "]"
630 PRINT "NUMBER OF SIMULATIONS= "; NSIM
660 PRINT "PROBABILITY OF FAILURE OF POLE = "; PF
730 RETURN
```