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Designated Fiber Stress for Wood Poles

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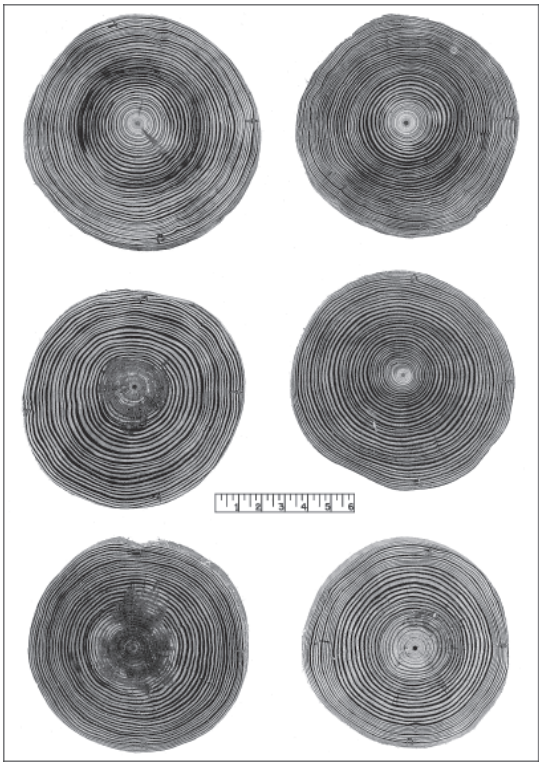

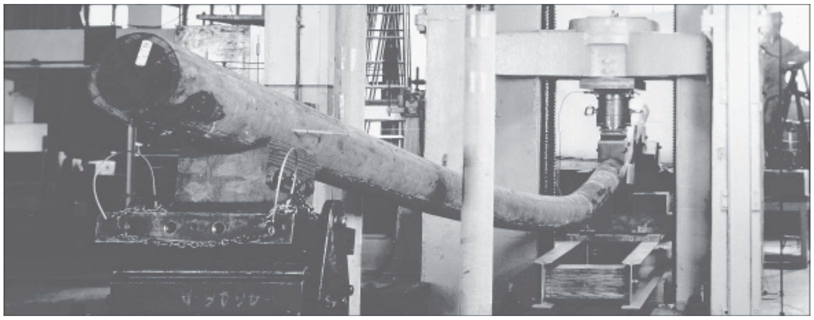

Abstract

Wood poles have been used to support utility distribution lines for well over 100 years. Over that time, specifications for a "wood utility pole" have evolved from the closest available tree stem more than 15 ft in length to straight, durable timbers of lengths ranging up 125 ft and base diameters of as much as 27 in. The continued success of wood poles in this application is due in part to the development of consensus standards. These standards define the phrase "minimum acceptable" to the satisfaction of both users and producers. They also encourage more competitive pricing by relaxing species as well as quality limitations, opening the market to a broader range of available timber resources. The American National Standards Institute (ANSI) standard ANSI O5.1 is an internationally recognized standard that has served as a guide for selecting the quality and size of wood utility poles for more than 70 years. From its inception, this standard has addressed issues of relative load capacity as well as physical quality to allow for species substitutions. In 2002, the relative strength evaluations previously published as a designated fiber stress took on added meaning when they were defined to represent the mean of the distribution of pole groundline strength values for various species. The change in meaning was accompanied by a more rigorous evaluation recognizing a change in strength with height and notation that pole strength distributions have a coefficient of variation of 20%. This paper reviews the history and philosophy of the ANSI designated fiber stress to help the reader more fully understand and appreciate the significance of changes adopted by the American Standards Committee O5 (ASC O5) in 2002.

Keywords: pole, ANSI standard, fiber stress, strength, history

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Abbreviations

ANSI	American National Standards Institute
ASA	American Standards Association
ASC O5	Accredited Standards Committee O5
ASTM	American Society for Testing and Materials
COV	coefficient of variation
CSA	Canadian Standards Association
DFS	designated fiber stress
EPRI	Electric Power Research Institute
LRFD	load-and-resistance-factor design
MOR	modulus of rupture
NESC	National Electrical Safety Code
REA	Rural Electrification Administration

SI conversion factors

English unit	Conversion factor	SI unit
inch (in.)	25.4	millimeters (mm)
square inch (in^2)	6.45	square centimeter (cm^2)
foot (ft)	0.3048	meter (m)
pound per square inch (lb/in^2)	6.895	kilopascal (kPa)
temperature (°F)	$[T_F - 32]/1.8$	temperature (°C)

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Introduction

Round timbers have been used structurally for centuries. Over the past 140 years, round timbers have been used primarily for utility structures. As in so many other areas of timber use, increased demand and competition from other materials have spurred development and efficiency in the structural use of poles. There is little doubt that Alexander Graham Bell and Thomas Alva Edison played key roles in initiating demand, but it is the influence of consensus standards (preservative treatment, strength tests, and material specification) that has enabled wood to retain a dominant role in support structures for telecommunications and electric power utilities.

Prior to 1930, there was little demand for standards that rigorously addressed issues of the "design" capacity of wood poles. The National Bureau of Standards (1927) began development of the National Electrical Safety Code (NESC) for this purpose in 1913. Early concerns focused on durability and cost. American chestnut and western redcedar were preferred for their durability. The second edition of the NESC (1916) recognized southern pine as a viable pole species after the development of creosote treatment. In that edition, southern pine was listed along with western redcedar and chestnut as having an ultimate bending strength of 5,000 lb/in^2. By 1927, northern white cedar was added, with an ultimate strength of 3,600 lb/in^2, and the bending strength of "dense" southern pine was rated at 6,500 lb/in^2. By this time, accumulating knowledge and experience related to the strength of wood poles indicated that somewhat higher values might be allowed (Wilson 1923). A Sectional Committee on Wood Poles was formed by the American Standards Association (ASA) to evaluate available data, and the NESC made provisions for use of new values as approved by the ASA. Decisions by this committee are discussed in detail by Colley (1932).

In June 1931, standard specifications for wood poles were approved by the ASA (Jones 1931). These standards incorporated a pole classification system originally developed by the American Bell Telephone Company in a standard specification aimed at maintaining uniform reliability for pole structures, despite a range of species. The standards quantified pole capacity using existing cantilever test data to assess groundline stress at failure for each recognized pole species. This value was then used to determine pole groundline dimensions required for each pole length and species. While changes have been made to the designated fiber stress (DFS) values referenced by the ASA standard, this basic approach to classifying poles remained unchanged for over 70 years.

As utility structure loads and importance increased, design became more of an issue. Most utility companies began using larger poles by the early 1960s, but they attracted little attention in the American National Standards Institute (ANSI) standard until 1972 when "H-class" pole sizes were introduced. Shortly thereafter, greater attention focused on the fact that the largest poles tested to that point were 55 ft in length with a butt diameter of 19 in., while the standard implied a knowledge of capacities for poles up to 125 ft long with a 30-in. butt diameter. Research conducted over the next 20 years pointed to the need for change to the traditional pole classification algorithm. As larger poles were shown to have a lower groundline stress at failure, the need to incorporate a pole "size" adjustment in the algorithm became apparent. This issue was added as an annex to the ANSI standard in 1986, but no changes were made to the pole size classification tables.

In the late 1990s, discussions about revising the NESC recommendations for wood pole design focused on discrepancies between the results tabulated in the ANSI annex and DFS reported in the standard. The Accredited Standards Committee O5 (ASC O5), which was responsible for maintaining the ANSI O5.1 standard, adopted a more rigorous definition of the ANSI DFS, which enabled them to indirectly recognize a size effect without changing the existing pole size classification tables. Redefining DFS to be the

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[1] Former chairman of ASC O5.

[2] Former chairman of ASC O5 fiber stress subcommittee.

actual groundline strength rather than the groundline stress at which a pole would fail and recognizing a change in strength with height basically results in reductions in the load capacity of poles where the failures are likely to be well above groundline. While the recommended strength change with height is easy to apply, it does not recognize variations between species, and that detracts from the premise of uniform reliability across species.[3]

The DFS values are all derived to represent "dry" strength, yet tabulated dimensions are explicitly declared to be "green" dimensions (sec 5.2 ANSI O5.1–1986). The standard suggests a 2% reduction in circumference for drying, which corresponds to a 6% reduction in section modulus or dry moment capacity. For poles that remain above fiber saturation in the groundline region, the "dry" strength gives a estimated moment capacity that is 86% (1/1.16) of the DFS. No adjustment is suggested for this situation.

Slight inconsistencies in the development of size classification are to be expected in a material specification. This has little impact on design when the design standards are independent of the material specification. In 2002, however, the ANSI O5.1 DFS values were considered to represent a basis for probability-based design of utility structures. The standard includes a statement about the DFS values that implies some degree of certainty that these truly represent average groundline strength values. It also states that poles have a bending strength coefficient of variation (COV) of 20% independent of species, species grouping, or size.

Changes made to standard definitions to justify changes to the standard could have some impact on the assumed reliability of existing as well as future utility structures. It is therefore essential that some documentation be provided to explain the basis for the past as well as the current interpretation of ANSI fiber stress values.

Objectives

This paper reviews the history of the ANSI O5.1 standard specification for wood poles from the time that poles were first used as utility structures up to 2002, when the ANSI standard made its initial transition from being just a material specification to being a pole design standard. First, we trace the growth of the wood pole industry in response to rapidly changing technology in the telecommunications and electric power utility industries. We next describe the evolution of the standard material specification, with emphasis on the evaluation and use of DFS values. Finally, we provide a summary and rationale for strength adjustments used to convert values derived from standard test procedures to values considered representative of utility poles.

The objective of this review is to provide documentation for future generations of the decisions made by consensus committees that have significantly influenced the evolution of the ANSI O5.1 standard. The change made in 2002 was the most significant of the standard's 70-year history. This paper is intended to highlight the significance of that change by offering a "before and after" glimpse.

Background

For over 140 years, wood poles have provided a low-cost solution to the problem of supporting utility wires for telecommunications and electric power. Wood poles presented a readily available resource when the first transcontinental telegraph line was constructed in 1861. By 1900, wood poles were being used to support electric power as well as telephone lines. Over time, utility companies gained experience at selecting poles that best met their needs for durability and load capacity.

The growing demand for durable wood poles spawned the need for nationally recognized standards. Bell Laboratories initiated efforts to develop test standards and material specifications for wood poles. In 1913, the National Bureau of Standards initiated the NESC to support design consistency for utility supply and communication installations throughout the United States. This was the first standard to publish pole strength values. In 1915, the American Society for Testing and Materials (ASTM) published the first standard material specification for round timber (ASTM D 25). This standard was intended for timbers to be used as piling but, except for the larger allowable knots and more lenient requirements for spiral grain afforded poles, it was identical to the standard specification for wood poles proposed by Bell Laboratories that was adopted as a national standard 15 years later.

In 1924, the American Bell Telephone Company and the U.S. Independent Telephone Association's American Standards Association (ASA) formed a Sectional Committee on Wood Poles. This committee used average pole strengths from the Bell Laboratories database to establish an interchangeable species–pole-size classification system. The Bell Laboratories strength data were obtained using a cantilever bending test that eventually was adopted by the ASTM and designated ASTM D 1036–49. This test provided estimates of relative groundline strength for each species. The ASA committee adopted a 10-class system comprising a series of tip load capacities increasing in increments of 25%. Groundline moments corresponding to these loads, divided

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[3] The pole specification was originally formulated by the American Standards Association (ASA) and was referred to as ASA O5.1. In 1972, the ASA was renamed the American National Standards Institute (ANSI). The designation for the pole standard was changed to ANSI O5.1 and the standard was managed by the Accredited Standards Committee O5 (ASC O5).

by the relative strengths determined from existing data, provided the groundline section property requirements for each pole species–class–length combination.

In 1931, the ASA adopted and published the first standard specification for wood poles. This standard contained minimum pole quality specifications as well as minimum required dimensions for 10 pole classes. The first seven of these classes were considered structural; classes 8, 9, and 10 were nonstructural. At that time, only four species were accepted for utility poles: western redcedar, Douglas-fir, southern pine, and northern white cedar. As new species and new data for accepted species were introduced, the committee refined its analysis of assigned DFS values and size class tables.

President Roosevelt's introduction of the Rural Electrification Administration in 1935 provided a definite impetus to the market for wood utility poles. By the end of World War II, annual production had grown to 8.1 million poles. To meet the growing demand, the ASA standard accepted a number of minor species in 1948. As there were no pole data for these species, the ASA committee assigned DFS values on the basis of published small–clear bending strengths adjusted for a relationship between full-size poles and small–clear (ASTM D 143, ASTM 2000) test values determined for lodgepole pine. Questions about the selection of these DFS values prompted ASTM to sponsor a wood pole research project, beginning in 1955, to assess the bending moment capacity of distribution poles. The final report of that program (Wood and others 1960) became known as the ASTM wood pole report, and it formed the basis for revision of the ANSI DFS values in 1963.

Large poles used for transmission structures became more of an issue with the introduction of six larger H-classes in 1972. These structures were used to support high voltage power transmission lines. The greater potential costs and safety hazard resulting from failure of these structures prompted design engineers to apply more rigorous design standards than had traditionally been used for distribution poles. The addition of these larger pole classes sparked discussion about the efficacy of the existing DFS values that had been supported by tests of smaller poles, most of which failed close to the groundline. One result of this discussion was the addition of a strength–height function introduced as Annex A to the ANSI O5.1 standard in 1979.

In the 1980s, a second major research project, sponsored by the Electric Power Research Institute (EPRI), was conducted to assess the load capacity of large poles. An initial effort was made as part of this research (Wolf 1979) to summarize published full-size pole test data in the form of a computerized database. Results of the ASTM- and EPRI-sponsored studies were compiled along with historic data. Statistical analyses of these data formed the basis for the probability-based-design guidelines that were added as Annex C to the ANSI O5.1 standard in 1986.

Wood maintained its dominance as a utility pole material throughout the 20th century but gradually lost market share. As power utilities built higher voltage lines with large conductors and longer span, more steel came into use. Spin-casting of steel-reinforced high strength concrete poles also had a major impact on markets for wood transmission structures. Poles fabricated from these materials are perceived by structural engineers as being more precisely engineered and therefore more reliable than wood poles.

The shift to steel and concrete in the more highly engineered transmission structures was accompanied by recognition among engineering professionals of the benefits of promoting the use of a load–resistance factor design (LRFD). This design methodology, when tied to a statistical or reliability basis, generally shows greater benefit for materials with lower variability when used in non-redundant systems. A move by the NESC to adopt an LRFD design basis increased industry pressure for better, more reliable strength data.

Efforts initiated by competing interests who supported the NESC led to a reevaluation of the ANSI O5.1 standard. Options debated by the ASC O5 included the following:

1. Retain the philosophy of classifying poles according to their load capacity under standard test conditions and recognize size effect and strength disparities highlighted by EPRI research (Phillips and others 1985, Bodig and others 1986a,b).
2. Adopt a more rigorous analysis that attributes observed disparities to taper differences and height effects, and classify poles according to assumed groundline strength. The proposed analysis supported a groundline strength equal to the historic DFS for all species, but required designers to recognize a change in strength with height.

Bodig and others (1986a,b) attempted to evaluate the strength of poles by assigning an ANSI class to each test pole and evaluating load capacity on the basis of ANSI minimum dimensions. This procedure accounted for effects of "over sizing" in that each test pole within a given class was assumed to have the dimensions dictated by minimum butt and tip circumferences and a linear taper. Pole load capacity, characterized as the groundline stress at failure, was assumed to provide a conservative estimate of strength at any location, as assumed dimensions were generally found to be conservative. This approach showed the ANSI DFS values to provide an appropriate estimate of mean strength for distribution poles and for Douglas-fir and southern pine transmission poles. It also indicated a slight size effect, varying by species, which made the DFS values unconservative when applied to western redcedar transmission poles.

In 2002, the premise of a size effect in poles prompted debate over possible changes to the DFS values. ASC O5 accepted the explanation that large taper in western redcedar transmission poles caused them to fail in weaker wood well above groundline. Projecting failure location strength to the groundline, then adjusting for drying in service, conditioning, and a class-oversize effect justified leaving the ANSI DFS value unchanged. Adopting a change in strength with height adjustment to explain the apparent lower groundline capacity for larger poles when tested in the standard cantilever configuration required acknowledging a reduction in strength with height in the standard. It was felt that this reduction in strength with height would have little effect on the strength of wood transmission poles as the centroid of transverse loads in service is generally more than 2 ft from the tip, placing maximum stress closer to the groundline. Gravity loads, however, have the opposite effect: they tend to move the location of maximum stress higher on the pole, closer to the expected location of failure in the standard test.

Table 1 compares values derived for poles over 50 ft in length using the traditional groundline stress-at-failure analysis and the approach adopted by the ANSI committee in 2002. The data for this comparison are primarily from the EPRI-sponsored research conducted in the 1980s (Goodman and others 1981, Phillips and others 1983, Bodig and others 1986a,b). All poles 50 ft and longer from these studies are included in this analysis. Table 1 basically compares the pole load capacity denoted as the groundline stress at failure under a standard test load to an estimate of groundline strength extrapolated from the failure location. The modulus of rupture at the failure location (MOR_{fl}) was determined from actual pole dimensions at the point of failure. This value was extrapolated to a groundline strength (MOR_{gl}) using a strength–height model presented in Annex A of the 1992 standard and included in the main standard in 2002.

Table 1 highlights the dilemma faced by ASC O5. Values shown in this table were all derived for poles ≥50 ft in length tested by the ASTM standard D 1036 cantilever test procedure. Groundline stress at failure is determined as the maximum groundline moment divided by the measured groundline section modulus. It represents a relative measure of pole capacity regardless of failure location. For long slender poles, the point of maximum stress (moment/section modulus) often occurs above the groundline. Assuming that wood pole strength decreases with height, it is possible that the point where stress exceeds strength will actually occur above the point of maximum stress. These effects make the longer pole appear weaker in terms of groundline stress at failure.

Rather than change the tabulated DFS values and pole dimensions in response to test results, the ASC O5 chose a new definition of DFS that required no change. The new definition identifies DFS as the groundline strength of an ANSI-classified, dry, conditioned pole. This new definition was deemed to have little effect on distribution size poles as they generally failed close to the groundline, making the previous definition of groundline stress at failure synonymous with groundline strength. For poles in lengths 50 ft and longer, the groundline strength was derived considering change in strength from the failure location using a model originally proposed by Bohannan (1971) to model strength reduction with height. Other changes included a maximum increase of 10% for drying in service, compared with 16% referenced in 1963, and an adjustment for class oversize effect (see Appendix D).

The change in definition of DFS solved one problem but created others. Drying and conditioning adjustments to the DFS on their own are not new; class oversize was not previously used to adjust the DFS values, but it was considered to present inherent conservatism in the sizing of poles. These factors took on new meaning, however, when the definition of DFS changed. The ANSI standard declares tabulated pole dimensions to be measured in the “green” condition. For a material specification, the 6% change from green to dry section modulus has little impact as these are treated as relative strength values. The dry/green section modulus ratio varies slightly between species, having little impact on relative pole dimensions when the largest pole in a class has a 25% greater section modulus than the smallest. In 2002, however, the change in definition of DFS from a groundline stress at failure of the pole to groundline strength gave these adjustments new meaning. The fact that the dry strength, derived as 1.16 times green strength, was used to derive dimensions to be measured in the “green” condition means that the “green” section modulus is 16% below that required to carry the designated class load. This corresponds to a 63% probability of failure (see Appendix A). The class oversize adjustment added for poles 50 ft and longer in the 2002 standard increased that probability of failure at the ANSI-designated load. This means DFS applied to longer poles infers a lower reliability for transmission poles than for distribution poles

The ANSI standard suggests that dry circumference will be 2% less than that measured in the green condition. This corresponds to a 6% lower section modulus. To carry its class load using dry strength, a pole of average strength sized in the green condition will therefore have a load capacity 6% below the implied capacity. Assuming a 14% strength coefficient of variation (Wood and Markwardt 1965), this translates to an implied capacity that is 0.42 standard deviations above the average strength, corresponding to a 66% probability of failure. As the moment capacity decreases faster than the moment with pole height, this probability of failure is likely to increase slightly toward mid-height.

Table 1—2002 ANSI derivation for DFS for poles ≥50 ft long from EPRI database[a]

		Fiber stress property (lb/in^2)			
Species	*n*	Groundline stress at failure	MOR$_{\text{fl}}$	MOR$_{\text{gl}}$	DFS
Douglas-fir	172	6,730	6,480	7,020	7,640
Southern pine	122	7,580	6,825	7,800	8,530
Western redcedar	105	4,200	4,310	5,030	6,040

[a] *n* is number of full-size poles tested in each length category. Groundline stress is based on measured dimensions. MOR$_{\text{fl}}$ is modulus of rupture at failure location; MOR$_{\text{gl}}$ is estimated groundline strength based on MOR$_{\text{fl}}$; DFS is groundline strength adjusted for conditioning, drying in service, and class oversize. For southern pine and Douglas-fir, a 10% increase for drying in service is countered by a 10% reduction due to high temperature pretreatment conditioning.

Pole Strength

Early evaluations of pole strength focused more on the comparison of wood species than on the selection of wood over other materials. It was not until the mid- to late 1960s, when the demand for power required larger, more reliable structures to support high voltage lines over long distances and through major metropolitan areas, that structural engineers became concerned about rigorous design standards for wood poles.

One of the earliest public references to pole strength was the NESC. In 1915, the second edition of the NESC listed three species: creosote treated southern pine, American chestnut (no longer available), and western redcedar. All three species were denoted as having the same bending strength (5,000 lb/in^2). As species options increased, pole users realized that not all species have the same strength and that they could not simply replace a pole of one species with the same size pole of another species. So, between 1920 and 1930, awareness of the need to classify poles by bending load capacity increased. In 1920, the NESC (3rd edition) adopted the ASTM definition of “Dense” (6 rings/inch and 33% or 50% summerwood in annual rings more than 2 in. from pith) and assigned dense southern pine a bending strength of 6,500 lb/in^2. The 4th edition of the NESC, published in 1927, included northern white cedar with an ultimate bending strength of 3,600 lb/in^2.

The first minimum quality specifications for wood utility poles were published in 1931. They included species-dependent minimum dimensions as well as physical appearance properties and were presented as a consensus standard agreed to by a group comprising wood pole producers, users, and general interest members. The specifications were published as a nationally recognized consensus standard under ASA guidelines.

In the first half of the 20th century, wood pole DFS values were changed a number of times in response to the introduction of new data. In the second half of that century, the introduction of new data resulted in a change to the definition of DFS rather than the values. When the ASA O5 standard committee was initiated, scientists from the Forest Products Laboratory of the USDA Forest Service advocated the use of standard, small–clear test values as a more consistent basis for assessment of pole load capacity. They cited inconsistencies in the full-size pole test data that compromised the credibility of these data as a design basis. The values ultimately selected represented a compromise heavily weighted in support of poorly controlled full-size pole test data. In 1955, the ASTM initiated a study to provide new, more credible test data (small–clear as well as full-size pole strength tests) that influenced the last changes to DFS values implemented in the ASA–O5–1963 standard in the 20th century.

Adoption of the larger transmission-size poles in 1972 sparked some question about the applicability of the existing DFS values to larger poles. Tests of larger poles, conducted in the 1980s, suggested a significant reduction in groundline moment capacity of larger poles when tested following the standard full-size pole test (ASTM D 1036). This initiated debate in both Canada (Bhuyan and Chetwynd 1994) and the United States over how this should be acknowledged in the load-capacity-based pole size classification tables (ANSI O5.1 and CSA–O15 (CSA 1990)).

First National Standard Specification for Wood Poles

By the time the first standard specification for wood poles was published in 1931, test data provided by the American Bell Telephone Company, American Telephone and Telegraph, Pacific Telephone and Telegraph Company, and the Engineering Experiment Station at State College Station in Raleigh, North Carolina, supported strength values of 5,600 lb/in^2 for western redcedar and 7,400 lb/in^2 for treated southern pine (Table 2). The values shown in Table 2 were derived using a cantilever bending test that had been developed by Bell Laboratories. This test was adopted as an ASTM standard in 1949. Descriptions of the individual poles tested to support this analysis showed they represented a range of conditions with respect to pole size, degree of conditioning, and quality. Colly (1931) points out that the sectional committee responsible for wood pole values chose to set strength values at half the standard deviation or 8% below the determined averages (Table 2, column 3) to address concerns over technical problems apparent in the data. The committee called this adjustment ($\gamma = 0.92$) a "variability factor." This resulted in the values of 7,400 lb/in^2 for creosote treated southern pine, 5,600 lb/in^2 for western redcedar, and 6,000 lb/in^2 for chestnut. The value of 3,600 lb/in^2 for northern white cedar was not adjusted. The committee decided that because this value had been used for at least 20 years with acceptable performance, it did not warrant a reduction to 3,300 lb/in^2.

The 1931 standard included a total of 10 pole classes. Pole classes 1 through 7 were designated as poles having tip load capacities defined by the function

$$P = 5400 - 950C + 50C^2 \tag{1}$$

where C is class (1 through 7) and P transverse tip load capacity of class C pole.

The groundline moment resulting from this load divided by the DFS for the species gives the groundline section modulus required for each pole size class. This relationship has remained unchanged for load requirements for pole classes 1 through 7. Classes 8, 9, and 10 were defined simply by tip circumference and were intended to cover lightly loaded lines.

Subsequent Standards for Wood Poles

Few events influenced growth in the utility pole industry more than the creation of the Rural Electrification Administration (REA). Formed in 1936 to expand the availability of electric utilities, the REA so accelerated the demand for poles that by 1947, yearly production was 8.1 million poles (Wilson and Drow 1953). Soon after the ASA O5 committee completed the 1931 standard, there was a push from REA to develop improved wood pole specifications and to expand tables to recognize the use of many western softwoods.

Because of discrepancies in species values resulting from inadequate control of moisture content, scientists from the Forest Products Laboratory in Madison, Wisconsin, proposed derivations based on an available database of bending strength values derived using standard green small–clear bending tests (ASTM D 143). The industry, however, preferred full-size pole data. The committee developed a compromise solution that involved derivation of a relationship between full-size poles and small–clear wood that would allow the use of small–clear data in instances where full-size pole data were missing or inadequate for deriving DFS. This approach was used by the War Committee on Specifications and Dimensions for Wood Poles formed in 1945 to prepare war standard specifications for species not previously used as poles.

Table 2—Strength values available to 1930 ASA committee[a]

		Full-size poles		Small clear green wood		
Species	*n*	Average MOR (lb/in^2)	Relative strength (lb/in^2)	Approx. relative strength (%)	Standard ultimate fiber stress (lb/in^2)	Relative strength of standard value (%)
Northern white cedar	56	3,621	41.2	45	3,600	45
Western redcedar	151	6,065	69	67	5,600	70
Chestnut	98	6,480	73.8	75	6,000	75
Southern pine (creosoted)	121	8,026	91.4	—	7,400	92.5
Southern pine (untreated)	55	8,784	100	100	8,000	100

[a] Colley (1932).

ASA O5.1–1948

After World War II, the ASA standards were revised to produce the new ASA O5.1–1948 standard, which attempted to resolve the discrepancies between values derived from standard tests of small–clear samples and those derived from full-scale pole tests (Wilson and Drow 1953). The committee dealt with this problem by first plotting the small–clear values (ordinate) versus pole-test values (abscissa) for average modulus of rupture (MOR) (Fig. 1). Drawing one line at 45° through the origin indicated that for most species, the pole values exceeded the small–clear derivations. Lodgepole pine, which was considered to have given satisfactory performance for years when designed at 6,600 lb/in^2, showed the greatest difference between small–clear and full-scale pole test values for the species shown in Figure 1, except for western hemlock. Data from the Canadian Forest Products Laboratory at the time showed higher values for western hemlock, which would have pushed the value above the designated line.

The ASA committee decided that a second line through the origin and the lodgepole pine coordinates should be used to define the fiber stress values for all species. This line defined values 20% greater for poles than for small–clear wood. A horizontal projection to this line from the (small–clear, pole) strength coordinates for each species defined the average value to be used in defining standard ultimate strength in the ANSI standard. Ratings assigned to new species were taken as less than or equal to 1.2 times the measured small–clear average MOR.

To minimize the number of dimension tables required, stress categories were developed by dividing the range of projected pole strength values into nine segments. The general philosophy was to provide 10% increments, but there was some deviation based on historic precedence. With this grouping, the "design" values ranged from 6% to 20% above the MOR determined from small–clear tests. A similar procedure to that described in the previous text was employed using western redcedar as the basis. This procedure also gave values for poles that were greater than values from the average small–clear MOR values, but to a lesser degree. Table 3 compares values derived using lodgepole pine and western redcedar bases to what was ultimately published in the 1948 ASA O5.1 standards.

Note that in 1948, the committee decided to retain the values for northern white cedar, western redcedar, and southern pine that were published in the 1931 standard. Species introduced by the war committee in 1945 were given the higher values derived using the lodgepole pine basis.

ASA O5.1–1963

The last changes to wood pole DFS values (Table 4) occurred in response to new information obtained from the ASTM wood pole research program (Wood and others 1960). The 1963 ASA O5.1 standard added four species (Alaska cedar, Engelmann spruce, white spruce, and Sitka spruce), removed one species (pond pine), and changed fiber stress values for four species (Douglas-fir, southern pine, northern white cedar, and western redcedar).

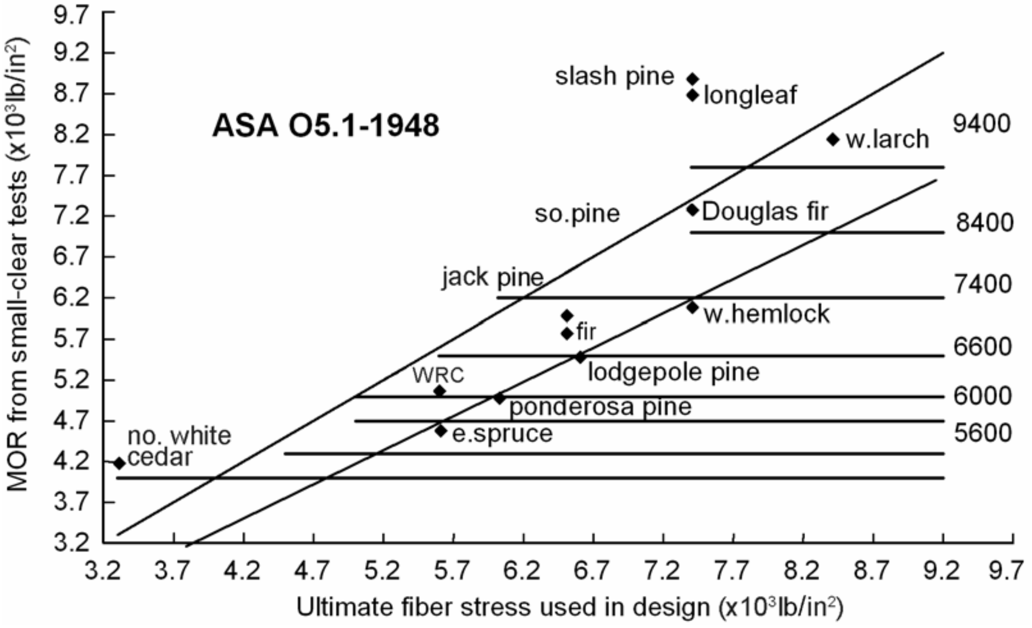

Figure 1—Comparison of tests of small–clear specimens and full-scale tests used as basis for DFS values on right (lb/in^2).

Table 3—Designated fiber stress values adopted in 1948 on basis of small–clear tests[a]

		ASA fiber stress (lb/in^2)		
Species	Clear wood MOR[a] (lb/in^2)	Lodgepole pine basis	Western redcedar basis	ASA 1948 basis
Northern white cedar	4,200	5,200	4,800	3,600
Englemann spruce	4,200	5,200	4,800	—
Western redcedar	5,100	6,000	5,600	5,600
Ponderosa pine	5,000	6,000	5,200	6,000
Jack pine	6,000	6,600	6,000	6,600
Lodgepole pine	5,500	6,600	6,000	6,600
Red pine	—	6,600	6,000	6,600
Western fir	5,800	6,600	6,000	6,600
Western hemlock	6,100	7,400	6,600	7,400
Douglas-fir	7,600	8,400	7,400	7,400
Southern pine	7,800	8,400	7,400	7,400
Western larch	7,500	9,400	8,400	8,400

[a] *Wood Handbook* (USDA 1940).

Table 4—ASA O5.1 1963 derivation of ASA fiber stress values based on standard small–clear specimen tests

					Adjusted strength (lb/in^2)			
		FPL–39/ASTM				Drying [b,d,f]		
Species	ASA O5-48	*n*	Pole MOR (lb/in^2)	Clear wood[a] (lb/in^2)	Green clear wood [b,c,d]	Small clear wood[e]	Pole MOR	ASA O5.1–63 (lb/in^2)
Northern white cedar	3,600	—	—	4,250	4,560	4,950	—	4,000
Western redcedar	5,600	46	5,370	5,120	5,500	5,960	5,790	6,000
Western larch[g]	8,400	54	8,500	8,180	7,900	8,577	8,250	8,400
Lodgepole pine	6,600	50	4,810	5,490	5,900	6,400	5,190	6,600
Douglas-fir	7,400	45	8,180	7,590	7,340	7,960	7,940	8,000
Yellow pine[g]	7,400	106	8,990[d]	7,970[g]	7,275	7,900	8,240	8,000
Loblolly	—	9	8,990	7,340	6,700	7,272	—	—
Shortleaf	—	40	8,450[e]	7,300	6,600	7,220	—	—
Longleaf	—	41	8,920	8,670	7,920	8,580	—	—
Slash	—	11	9,600	8,570	7,820	8,480	—	—
Western fir	6,600	—	—	—	—	—		6,600
Red	—	—	—	5,950	6,390	6,930	—	—
Grand[g]	—	—	—	6,060	6,510	7,060	—	—
Noble	—	—	—	5,790	6,220	6,750	—	—
Silver[g]	—	—	—	5,890	6,300	6,860	—	—
White	—	—	—	5,700	6,120	6,640	—	
Western hemlock	7,400	—	—	6,140	6,590	7,150	—	7,400
Alaska cedar	—	—	—	6,450	6,930	7,510	—	7,400
Sitka spruce	—	—	—	5,660	6,080	6,590	—	6,600
White spruce[g]	—	—	—	5,580	5,990	6,500	—	6,600
Engelmann spruce	—	—	—	4,540	4,880	5,290	—	5,600

[a] Values from *Wood Handbook* (USDA 1955).

[b] Reduced for variability effects.

[c] Increased 5% for form effect.

[d] Reduced for conditioning: 0.9 for boultonizing (Douglas-fir, larch) and 0.85 for steaming at <245°F (southern pine).

[e] Increased 8% for form effect.

[f] Increased 16% for drying in service.

[g] Clear wood strength values for these species changed enough in 1966 to have a significant impact on the ASA derivation, but since these are minor species the changes were never recognized by the ASA standard.

The 1963 standard also made an explicit declaration that the specification was intended only for poles loaded as cantilever members subject to transverse load resulting in maximum moment at the groundline.

More than 30 years of experience in observing the performance of wood poles and relating that performance to pole specifications weighed heavily in the selection of the DFS values used to classify poles using a standard definition of cantilever load capacity. Factors considered in the selection of DFS values included traditional adjustments adopted in the 1930s—variability, form factor, conditioning effects, and moisture content. The effects of high temperature conditioning on wood had been studied rather extensively. The other adjustments, however, which dated to discussions prior to 1940, were not well supported by theory or data but had been referenced in selection of values given in previous standards. These factors are discussed in more detail in the section titled Nominal Strength. For those species for which there were no or limited full-scale pole test data, the committee referenced published values for small–clear bending strength (Wood and Markwardt 1965, USDA 1955).

Table 4 compares the values given in the 1948 (column 2) and 1963 (column 9) versions of the ASA O5 standard. The intermediate columns show the scenarios considered by the committee that appeared to have the greatest influence on the assignment of DFS values. These include values derived from the ASTM wood pole report (Wood and others 1960) (Table 4, columns 3 and 4) and published clear wood values (column 5). Columns 6 through 8 show these values adjusted to take into account pretreatment conditioning, drying in service, and between-study variability. There was no standard set of parameters used to select a DFS value (column 9). This was, in the words of Wood and Markwardt (1965), “a system of rationalizations that afforded the technical basis supplemented by engineering judgment.”

Most of the ANSI DFS values appear to be based on clear wood strength with no adjustment for grade effect (Table 4, last column). Values such as those for western hemlock, western larch, lodgepole pine, and the western fir group, unchanged from the 1948 standard, appear to be more closely aligned to adjusted small–clear strength values than to full-scale pole test results. Northern white cedar was assigned a value close to the published clear wood value with no adjustment. For Alaska yellow cedar, western redcedar, and spruce, selected values were close to the clear wood strength adjusted for form (Newlin and Trayer 1924), variability (0.93), and in-service drying.

Changes to full-scale pole data and published small–clear strength values introduced subsequent to the 1963 standard had no effect on the ASA O5.1 values. In 1963, the Forest Products Laboratory played a dominant role in setting allowable stresses. The small–clear strength values referenced by ASA O5.1 were those published in the *Wood Handbook* (USDA 1955). Industry concern over the dominant role played by the Forest Products Laboratory led the ASTM Committee on Wood to introduce “Standard methods for establishing clear wood strength values” (ASTM D 2555) in 1966. In that standard, clear wood values for several species were changed from the values reported in the *Wood Handbook*. These changes, however, were never acknowledged by ANSI in the form of a revision to the 1963 DFS values. This supports the conclusion that these values were never tied to any algorithm that references published wood strength values.

Table 5 shows how the ASTM changes to clear wood strength values (D 2555–66) would have affected the ASA fiber stresses. Despite the fact that no significant changes were reported for the strength values for yellow pine poles, there was a change to the way “species group” clear wood

Table 5—Effect of changes in clear wood strength[a]

Species	1963 ASA–O5.1 fiber stress (lb/in^2)	D 2555–66 clear wood strength[b] (lb/in^2)	Adjusted clear wood strength[b] (lb/in^2) Green	Dry[c] 1.16 (Green)
Southern pine	8,000	7,590	6,861	7,923
Western larch	8,400	7,652	6,920	8,020
Grand fir	6,600	5,839	5,860	6,800
Noble fir	6,600	6,169	6,200	7,190
Silver fir	6,600	6,410	6,440	7,470
Western hemlock	7,400	6,637	6,670	7,730
White spruce	6,600	4,995	5,020	5,820
Englemann spruce	5,600	4,750	4,770	5,530

[a] ASA O5.1 DFS values not modified to recognize changes in clear wood strength presented in ASTM D 2555 standard.

[b] Reduced for variability effects (0.93), increased 8% for form effect, adjusted –0.9 for kiln drying for pine and larch (air-dry for other species, 1.0).

[c] Increased 16% for drying in service.

stresses are derived. The ASTM D 2555 standard requires that these values be assessed by weighting according to standing timber volume (7,590 lb/in^2) rather than using an average of the subspecies mean values. For the other species listed, average green clear wood values were changed by the ASTM D7 committee to reflect changes in the supporting data.

In 1963, the ASA O5 committee reduced the number of pole classes by one. Classes 8, 9, and 10 were not rated as utility poles. Rather they were used in construction of pole buildings. The suggestion to drop all three classes was countered by the agreement to drop only class 8, leaving the gap between utility and construction poles.

Addition of H-Class

In 1972, along with the change from the "ASA" designation to "ANSI" came the addition of an appendix that provided information on the assumptions made in developing the tabulated pole size classes. Load capacities assumed for each pole class were moved from the main body of the standard into the appendix, and these values were expanded to include the new H-class poles. Unlike the standard classification for distribution poles that is inversely related to pole size/capacity, the new size designations (H1 through H6) correlate directly to pole size and capacity. To simplify discussion related to the ANSI size classes, an index (*i*) is used, ranging from 0 to 12 and correlating to pole size from class 7 (smallest of utility poles) to H6 (largest transmission pole). The class loads for these poles fit a quadratic function (Eq. (2)):

$$\text{Class load} = 1200 + 250i + 50i^2 \qquad (2)$$

Tip circumference values for ANSI class poles follow a linear function of this class index.

$$\text{Tip circumference} = 15 + 2i \qquad (3)$$

The groundline circumferences are derived to provide the section moduli required to resist the groundline moments imposed by the class loads applied transversely at a location 2 ft from the top of the pole (ASTM D 1036–1999). The estimated groundline circumference is then extrapolated to a circumference 6 ft from the butt to give the ANSI tabulated values.

To estimate a 6-ft-from-the-butt circumference from the calculated groundline value, ANSI incorporates species-dependent circumference tapers, also given in the ANSI appendix. The origin of these taper values is uncertain, and they appear to have changed slightly over time. In 1974, Bohannan and others evaluated survey data collected for southern pine, Douglas-fir, and western redcedar pole producers. Their analysis showed mean circumference tapers for these three pole species (0.34, 0.31, and 0.46 in/ft, respectively) to be 20% to 50% greater than the values noted in the ANSI appendix and up to 50% lower than values derived from tabulated class tip and 6-ft-from-butt circumferences.

In addition to information on how size classes were determined, the ANSI annex provides some cautionary statements. Of particular importance is the note regarding the assumption that maximum stress occurs at groundline. This is theoretically correct only if the circumference at groundline is less than or equal to 1-1/2 times the circumference at the point of load. If this is not the case, maximum stress is likely to occur some distance above the groundline. Prior to 2002, the DFS was derived to represent the average stress at failure rather than groundline MOR and could be used with ANSI minimum dimensions to conservatively estimate pole capacity regardless of the location of maximum stress.

Wood Pole Test Data

In 1949, ASTM published the first standard test procedure for wood poles (ASTM D 1036). This standard actually includes two methods for testing poles as a structural element intended to be loaded as a cantilever having its maximum moment at a location close to the butt end. One method involves actually loading the pole as a cantilever beam, with one end rigidly anchored at and below "groundline" and load applied 2 ft from the other end. The second method, called the "machine," incorporates a universal testing machine. The pole is placed on simple supports, and a vertical load is applied at the "groundline" location. Test results differ slightly as the machine method includes stress resulting from the dead weight of the pole. Bending moment resulting from dead load is maximum at a location close to mid-span. When added to the moment resulting from the groundline concentrated load, this results in a slight shift in the point of maximum stress away from the groundline. The difference is generally inconsequential for short poles, but it becomes more significant as pole length increases.

ASTM Wood Pole Test Program

The ASTM wood pole research program initiated in 1955 strengthened the basis for characterizing the strength of domestic pole species. The study involved tests of full-size poles of five domestic species (southern pine, Douglas-fir, western redcedar, lodgepole pine, and western larch), along with matched small–clear specimens. Tests focused primarily on 30-ft poles but also included 25-ft and 55-ft poles. Yellow pine poles were tested using both the cantilever and machine tests described in ASTM D 1036. The other species were evaluated using only the machine test method. A total of 630 full-sized poles and 14,000 matching small–clear specimens were tested.

Results of this study (Wood and others 1960) provided a basis for the 1963 revision of the ASA O5.1–1948 standard DFS values. Changes included increases to designated strength values for Douglas-fir, yellow pine, and western redcedar. In addition, the data strengthened the committee's confidence in standard small–clear tests as a basis for evaluating pole strength, leading to an increase in the DFS for northern white cedar. Wood and Markwardt (1965) provide a synopsis of the committee decisions that led to these changes. Table 4 compares results published in the ASTM pole report, Wood and Marwardt's work (FPL–RP–39), and the ASA O5 standards.

Key points gleaned from the ASTM study that influenced subsequent decisions related to pole strength include the following:

- Average groundline strength of poles is closely correlated to strength of small–clear samples taken from the butt section.
- Change in strength with height is correlated to change in specific gravity.
- Strength of young fast-grown poles having fewer than 6 rings per inch in the outer 2 in. is significantly lower than that of poles having 6 or more rings in the outer 2 in.
- Poles in the 55-ft length category have lower groundline MOR than do 25-ft poles.

EPRI Wood Pole Test Program

The second major contribution to the advancement of wood pole engineering came in the 1980s when the Electric Power Research Institute (EPRI) sponsored a research program to promote development of probability-based design guidelines for wood utility structures. This effort attracted the attention of the wood industry as a major step in the development of probability-based load-resistance-factor design (LRFD) for engineered structures and for promoting the image of wood poles as engineered components. This research addressed many issues that were questioned after the adoption of the H-class sizes in 1972. The testing phase focused primarily on the larger poles, including 122 southern pine poles ranging from 50 to 70 ft long, 172 Douglas-fir poles ranging from 50 to 95 ft long, and 100 western redcedar poles ranging from 50 to 70 ft long. The analysis phase included a review and compilation of the state of the knowledge on wood poles. The published results have been widely referenced by academia as well as private industry and form the basis for design and maintenance of wood pole transmission line systems. Results are published by the EPRI under the designations EPRI EL–2040 project 1352–1 and EPRI EL–4109 project 1352–2.

As part of the EPRI-sponsored research from 1980 to 1985, a database was compiled from existing wood pole test data. This database comprised results of full-scale pole tests conducted in accordance with the ASTM D 1036 standard test procedure. The references for these studies are listed in Appendix B. These data were made available to the ANSI O5.1 committee for use in supporting current and future utility pole standards.

In 1994, Bhuyan and Chetwynd (1994) used a large portion of the data generated by the EPRI project along with privately sponsored testing of poles to provide a basis for evaluating the classification stress values published by the Canadian pole standard (CSA O15) as well as ANSI. They reviewed values for western redcedar, yellow cedar, red pine, jack pine, lodgepole pine, southern pine, and Douglas-fir. Their analysis of this data showed that test data for western redcedar, plantation-grown red pine, and some southern pine poles produced in the last 10 years and classified by the CSA O15 standard have load capacities that are one class lower than that implied by the standard. ANSI O5.1 classification stresses are greater than those of the Canadian Standards Association (CSA) for these three species.

The two most significant conclusions drawn from this work are (1) as pole size increases groundline moment capacity decreases and (2) western redcedar pole test data do not support the classification stress values, especially for poles greater than 50 ft in length.

Nominal Strength

Throughout the recorded archives that constitute the history of the ASA/ANSI O5 material specification, there are numerous discussions related to the meaning of the strength values selected as a basis for classifying wood poles. Design values for wood poles have traditionally been established by the NESC, as was discussed in the Introduction. For the past 60 years, however, the NESC has referenced the ASA/ANSI standard DFS values as a basis for wood pole design values. It is interesting to note that until the 2002 revision of ANSI O5.1, the DFS values were treated solely as a basis for relative size classification within the standard: the standard provided no assessment of these values in terms of the probability density function for bending strength. This changed in 2002 when the committee decided to redefine DFS as a groundline fiber strength that represents the mean value for a strength distribution having a coefficient of variation (COV) of 20%. The following text provides the background discussion that led the committee to make this change.

Initial Estimates

The natural variability of a material such as wood adds to the complexity of balancing design safety and economic feasibility. In the early 1930s, the ASA O5 committee adopted an analysis procedure that was assumed to give a 3% probability of failure for poles in a distribution system where each pole is assumed to carry its ASA O5 "class" load. This analysis relied on four basic assumptions (Colley 1931):

1. Pole strengths are normally distributed.
2. The average pole in any size class has an actual groundline section modulus that is 12.5% greater than that calculated on the basis of the minimum class circumference.
3. Creosoted southern pine poles have a mean strength of 8,026 lb/in^2 and standard deviation of 1,348 lb/in^2; western redcedar has an average strength of 6,065 lb/in^2 and standard deviation of 1,151 lb/in^2. These data provided the basis for assigning a 17% COV for bending strength for all pole species.
4. Poles are used in a redundant system in which three in-line poles share load, each carrying load in proportion to its relative stiffness.

The first two assumptions were reasonably conservative. The load increment between consecutive size classes was initially set at 25%. This meant that groundline section moduli for poles of a given length had a 25% increment between classes. If the average pole within a class is assumed to have a section modulus that falls midway between its class minimum and that of the next larger class, it will have an approximately 12% higher groundline bending moment capacity than that of a pole having the minimum class dimension. This means that the groundline stress on the average pole is 1/1.12 or 0.892 times[4] the DFS.

Colley (1931) points out that the sectional committee on wood poles chose to set strength values slightly below the determined averages to address concerns over technical problems apparent in the data. There was concern among committee members that moisture content variations in test poles had a significant impact on species strength values that were being used to establish size classification tables. Rather than reject the data, the committee chose to apply a confidence adjustment for bending strength, publishing values calculated to be one-half the standard deviation below the measured mean. Data available at the time showed a range of within-sample strength COVs of 17% to 23%. Rather than derive this for each species, the committee chose a "variability factor" ($\gamma = (1 - 0.5\Omega)$) based on the COV (Ω) of 17%. As a result, nominal resistance values were set as 92% of the mean test strength.

In 1963, the ASA committee adopted the same confidence adjustment on mean strength. At that time, the ASTM data showed COV values ranging from 10.8% to 16%. The committee assumed a COV of 14% and recalculated this variability factor to be 0.93. In 1963, this adjustment was warranted on the basis of small within-class sample size rather than variations in moisture content below fiber saturation at time of test.

In an attempt to assess the true meaning of the DFS value, Colley (1931) evaluated the product of "variability" and "oversize" adjustments in terms of standard normal deviations. For poles of classes 4 through 7, the product of class size adjustment (0.89) and variability (0.92) is 0.82. For a normal distribution having a COV of 17%, this factor times the mean gives a value equivalent to 1.055 standard deviations below the mean which equates to the 14th percentile of a normal distribution.

The committee then attempted to show that this value actually represented the 3rd percentile of in-service pole strength by proposing that a utility distribution line represents a load-sharing system. Theoretically, if the load on any one pole is distributed in a manner to cause the loaded pole and two adjacent poles to deflect together, each pole might be assumed to carry a share of the load proportional to its relative stiffness. Given a perfect correlation between stiffness and strength, one might also assume that each pole is then loaded to the same proportion of its load capacity. This means that failure will not occur until all three poles have been loaded to capacity. So, the capacity of the system is limited by the average strength of three poles in series rather than the weakest pole. The distribution of the average values of three poles will have a variance equal to one-third the variance of the individual values. Equating the ANSI DFS (mean pole capacity, adjusted by the variability and class-oversize adjustment factors) to a fractile of a normal distribution using Equation (4) provides a basis for estimating the probability that a pole in a load-sharing system will fail at a load that represents the average single pole capacity.

$$\gamma(0.892)\overline{X} = \overline{X}\left(1 - \frac{k}{\sqrt{3}}\Omega\right) \qquad (4)$$

where

Ω is coefficient of variation (assumed to be 17%),
γ variability of adjustment (assumed to be 0.92), and
k normal distribution variable.

Solving for k gives the value 1.827. Assuming that the adjusted strength representing the average stress on three adjacent poles is 1.821 standard deviations below the mean strength of three adjacent poles suggests that the DFS represents the 0.03 fractile of the distribution of strengths of three-pole load-sharing assemblies.

This analysis involves three flawed assumptions. First, there is no rigid load distributing element in a distribution line that will redistribute load laterally to adjacent poles in such a manner that all three poles will deflect equally. If such a mechanism did exist, the correlation between stiffness and

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[4] Today, the pole load increment varies from 27% for the smallest pole class to 14% for the largest, so the groundline stress on the average pole will vary from 0.88 to 0.93 times that estimated on the basis of minimum dimension.

strength would not be perfect, so the reduction in variability would not be as large as $1/3^{1/2}$. Second, the basis for the "variability factor" (0.92) was a lack of confidence in the data. The adjusted value was the committee's estimate of mean performance for green poles that met their minimum specifications, and γ should not have been included in the expression. Finally, load sharing is an engineering judgment, not a material property, and has little meaning in a material specification.

It does not appear that the NESC, which provided design recommendations on the basis of the ASA pole strength values, treated these values as though they were lower fractiles. It was standard practice at the time for timber engineering standards to recommend design values as 25% of the mean strength. The 5th edition of the NESC (1941) adjusted ultimate strength by a factor of 0.25 for grade B and 0.375 for grade C. This would have been overly conservative if applied to a 0.03 fractile of the pole strength distribution. By the early 1960s, design standards for wood began referencing a lower 5% exclusion for strength divided by 2.1 to account for duration of load effects. When applied to a population having a COV of 30%, this results in 0.25 times the mean. So, by the 1960s, the NESC approach was recognized as compatible with recognized timber engineering standards.

Test Data and In-Service Performance

A major issue for most standards development has been the relationship between laboratory evaluation under controlled conditions and in-service performance.

Early debate over development of the ANSI O5.1 standard focused on the efficacy of standard full-size pole tests compared with standard small–clear tests as a basis for setting pole size class requirements. The objective was to provide a classification system in which any pole of a designated "class" had the same tip load capacity, regardless of species. The small–clear test assessed the fiber strength that would control in the event of a groundline failure.

The full-size pole test provided a more direct assessment of pole capacity independent of failure location. The full-size pole tests attracted greater support as being more representative of the end product.

As engineered design gained importance for utility structures, the relationship between standard full-size pole tests and in-service pole loading began to be debated. The pole size classification system adopted by ASC O5, based on groundline stress at failure, was generally considered to provide an appropriately conservative estimate of load capacity for poles loaded as cantilever beams. When standard load conditions are applied to longer poles, tree shape and physiology have a significant influence on where and how the poles fail, leading to questions about a negative bias for longer poles that are likely to fail further away from groundline and to exhibit a lower groundline stress at failure.

While standard tests provide a means of assessing relative value between species and pole sizes, they rarely address all design issues. In the case of wood poles, DFS values used to set pole size class include some adjustment to account for effects of high temperature conditioning and drying in service.

Conditioning

Preservative treatments are what make wood a viable option for use as poles. Their effect on strength is therefore of vital concern to pole producers. Many studies conducted to assess treatment effects on strength have concluded that any strength reduction is due to high temperatures involved with pretreatment conditioning. This conditioning sterilizes the pole and removes excess water, making the pole more conducive to treatment.

The committee responsible for the 1963 version of the standard recognized potential strength reductions of 15% for steaming and 10% for boultonizing. The steaming reduction was supported by research studies conducted by Betts and Newlin (1915), Wilson and others (1930), Buckman and Rees (1938), Stamm (1956), and Thompson (1969).

The ASTM pole research program (Wood and others 1960) provided data that suggested that previous studies on steaming effects were non-conservative. The average strength of treated western larch and southern pine poles was less than 85% that of untreated poles. The difference in western larch was attributed to the lower specific gravity of the treated poles compared with that of untreated poles. In the case of southern pine, the committee attributed the large reduction to the fact that the poles were conditioned following the guidelines of the AWPA standards C1–55 and C4–55, which permitted the poles to be subjected to 259°F for 8 to 13 h. During the conditioning processes, temperatures reached as high as 267°F on four of the charges. These older standards were considered to be too destructive and not representative of state of the art in 1963. Erickson and Dohr (1959) compared bending strengths of longleaf pine poles at a range of conditioning times and temperatures and confirmed MacLean's findings: on average, poles that were steam conditioned at 245°F for 15 h showed a 16% loss in bending strength. The data of Erickson and Dohr also showed a reduction in variability, which led to the observation that a 13% loss in bending strength occurred at the 5th percentile.

Drying Effects

Wood drying in service has a mix of effects on load capacity. Wood strength and modulus of elasticity are known to increase with drying below the fiber saturation point, while section property is reduced as a result of shrinkage (Wolfe 2000). Drying checks that open as poles dry may expose poorly treated wood to trapped dirt and moisture, promoting local decay pockets. Excessive drying and weathering in arid environments may cause poles to lose strength over time. For most heavy timber engineered applications, these effects are considered to be offsetting.

Eggleston (1952) supported the premise that the green groundline strength basis was appropriate for the ASA pole size classification. He evaluated butt-soaked dry poles and concluded that except in soils so dry that pole butt moisture content would not exceed the fiber saturation point, the strength of the tops of southern pine poles in service is probably very close to the (saturated) groundline strength. Rhatigan and Morrell (2002) reported an average of 19% to 40% moisture content at groundline for creosoted Douglas-fir poles that had been in service for 35 years, suggesting that the assumption of dry in-service conditions is nonconservative.

Kulp (1957) reported finding decay in poorly treated wood above groundline. He found that 1.7% of 1,885 southern pine poles placed in test installations were removed after an average 15 years of service; 5 failed within 13 years of service as a result of decay. These poles had been treated with coal tar creosote. Kulp reported that 1.7% of the remaining poles were deteriorated as a result of decay or termite attack after 6 to 18 years of service. In semi-arid regions of Montana and the Dakotas, the tops of 6.1% of butt-treated lodgepole pine poles were partly decayed after 5 to 8 years of service, and the tops of 28% of butt-treated western redcedar poles had light to moderate shell-rot after 9 to 15 years of service. Light top decay around spur marks was present in 21% of untreated northern white cedar. Kulp did not report moisture content, but the incidence of decay is a reasonable indication that moisture content exceeded 20% in these poles.

Research in support of increasing pole strength for drying in service was reported by Wilson and others (1930) and by the ASTM pole study. These studies showed a 10% strength increase for shortleaf pine poles and 14% strength increase for lodgepole pine when dried to 20% moisture content. Further support for recognizing the dry strength of poles was gained from a survey conducted by the REA that showed that moisture content 4 ft above ground was lower than 20% in a high percentage of poles (Wood and Markwardt 1965). The REA data included more than 1,400 poles in a region north and east of 40˚N,100˚W (Kansas/Nebraska line, west of Missouri, north of Cincinnati, Ohio) and 700 poles to the south and east of that point. More than 1,200 poles were surveyed between mid-Kansas and the California/Nevada border; 33 were surveyed west of that point. The survey showed that moisture content at 4 ft above ground exceeded 20% in 13% of poles surveyed in the southeast region, 4% in the moderate northeast region, and fewer than 1% of poles west of the Colorado/Kansas border.

This information was available to the ASA committee in 1963 when it decided to consider drying in-service to account for a 10% to 16% increase in bending strength values determined in the green condition. Discussions leading to the acceptance of these increases included concern for the length of time from installation to the point a pole actually reaches 20% moisture content and the fact that for most poles, the highest stress occurs in a region where moisture remains at fiber saturation. In most cases, these increases offset reductions caused by pretreatment conditioning, resulting in values that appeared to be representative of the test data. It was only for species such as western redcedar that did not require high temperature conditioning that the drying effect noticeably increased strength above that determined in green pole tests. As the ANSI standard was considered a material specification and not a design standard at the time, no reference was made to these assumptions or to any required adjustments for poles to be used in wet environments. The DFS values were used by ANSI only to set relative dimensions in the pole size classification tables. They were not considered by ANSI O5.1 to be a design basis.

In 1986, ASC O5 made an explicit declaration that the tabulated pole circumference values were to be measured in the "green" condition. This was done to address questions about undersized dry poles. The fact that the tabulated "green" circumferences are derived using estimates of "dry" groundline strength is of little consequence when the only concern is size classification to ensure species interchangeability. If the standard is to promote the DFS values as a basis for design, however, it should be remembered that the ANSI class loads are not a good measure of the load capacity of the classified "green" pole.

As discussed earlier in this paper, a major problem with using dry strength to set a green dimension is that the failure probability for the target pole is greater than 50% at the designated class load. A green pole sized using the dry strength will have a strength that is over one standard deviation (or 16%) below the strength required to carry the designated class load. When this pole dries, it will have a section modulus 6% below that required of a dry pole.

The offsetting effects of shrinkage and fiber strength increase with drying, combined with the results of the EPRI-funded research on poles taken out of service, suggest that taking an increase in strength for drying in service is nonconservative. The EPRI research included a limited set of 74 western redcedar, 49 southern pine, and 11 Douglas-fir poles taken out of service after 30 years that had average strengths 20% to 30% less than that of new counterparts. The effect of drying in service most likely varies with exposure conditions as well as species. An across the board adjustment was not warranted, and ASC O5 provided neither recommendations nor warnings about adjusting load capacity for drying.

Pole Shape

The relationship between in-service capacity and standard test capacity is influenced by physical aspects of the pole and test specimen. These may be placed in three categories:

1. Form, relating to round as opposed to prismatic sections.
2. Taper, relating to change in section property over pole length.
3. Size, relating to weak-link theories, which suggest that average extreme fiber strength will decrease as size increases.

Pole form—Form effect refers to differences in the strength assessment of round as opposed to rectangular sections when using the standard engineering model for stress in a bending member. This model, which estimates stress as an inverse function of section modulus, assumes that stress is uniform across the width and varies linearly with distance from the neutral axis of any section. Newlin and Trayer (1924) concluded that this model gives roughly an 18% higher value for stress at failure for round than for square wood sections. Basically, the researchers found that square and round sections having the same cross-sectional areas had the same bending moment capacity. Because the square section has an 18% greater section modulus, its calculated stress at failure is 18% less than that of the round section. Therefore, when estimating the strength of a round section on the basis of published small–clear square section strength, some form effect should be recognized.

Pole taper—Wood pole taper has played a major but largely unrecognized role in the history of structural performance of ANSI poles. Table 6 summarizes measured and implied wood pole tapers that have had some influence on this standard and on the history of pole performance.

The ANSI pole size classification relies in part on measured pole taper values, but the size classification tables hold conservative implications in this regard for the pole user. In deriving the 6-ft-from-the-butt (6'FB) circumferences in the size classification tables, the values listed in Annex B of the ANSI O5.1 standard (see Table 6) are used to transfer required circumference dimension from groundline to the 6'FB location. In selecting an ANSI pole, however, a pole designer must assume that it will have the designated minimum tip and 6'FB circumferences given for its class. The corresponding taper is the difference in these dimensions divided by the length between them. As the tip circumferences vary in increments of 2 in. and the 6'FB circumferences for any length change in increments exceeding 2 in., the implied tapers increase with pole size. Table 6 provides a comparison of 55-ft and 80-ft poles; the low values in these cases are for class 7 poles and the high values for class H6 poles.

Table 6 also provides measured tapers reported by Bohannan and others (1974) and Wood and others (1960). The circumference values reported by Bohannan were determined from measurements of poles ranging in length from 55 to 80 ft. These data showed no consistent relationship between taper and pole length. In all cases, however, the taper appeared to increase with groundline circumference as implied by the ANSI tables. Wood's data, which was determined for 25- to 55-ft-long poles, showed a definite increase in taper with pole length for southern pine and western redcedar, but the evaluation did not provide a comparison across size classes.

Comparing ANSI Annex B values to those reported by Bohannan suggests possible undersizing for longer poles. While the ANSI butt taper values appear to be about average for the 25- to 55-ft poles reported by Wood, they fall below the median values reported in Bohannan's survey for each species (0.021 vs 0.028 for southern pine, 0.017 vs 0.027 for Douglas-fir, 0.032 vs 0.039 for western redcedar). The implication is that on average, for poles >50 ft in length where the groundline is above 6 ft from the butt, the 6'FB dimension required to give the necessary groundline section modulus will be underestimated using the ANSI Annex B value. Projecting back to groundline using the implied tapers in the ANSI tables results in an undersize groundline dimension for the classified pole.

Table 6—Pole taper values

		Taper (in./in.)[a]								
		Southern pine			Douglas-fir			Western redcedar		
Reference	Pole length (ft)	Low	Med.	High	Low	Med.	High	Low	Med.	High
Bohannan	55–80	0.014	0.028	0.045	0.011	0.027	0.050	0.018	0.039	0.077
Wood	25–55	0.015	—	0.029	0.012	—	0.017	0.024	—	0.042
ANSI Annex B	—	0.021	0.021	0.021	0.017	0.017	0.017	0.032	0.032	0.032
ANSI tables	55	0.033	—	0.043	0.033	—	0.043	0.047	—	0.055
ANSI tables	80	0.030	—	0.040	0.030	—	0.040	0.038	—	0.049

[a] Calculated as change in circumference per change in height.

Taper has a definite effect on how stresses are distributed in a pole. Stress is not distributed linearly in a tapered cantilever beam. The bending moment (*M*) for a cantilever increases at a constant rate from the point of load application to the reaction. However, when divided by the section modulus (*S*), which increases as a cubic function of distance from the load point, the quotient (*M*/*S*) or stress is an inverse quadratic function that increases at a decreasing rate. If the taper is steep enough or the cantilever long enough, the stress stops increasing with distance from the load point and may decrease. This means that the point of maximum stress in a tapered cantilever does not necessarily coincide with the point of maximum moment.

For a linearly tapered, round cantilever beam of uniform homogeneous material, the quotient *M*/*S* will reach its maximum value at a point where the pole circumference is 1.5 times the circumference at the point of load. Under these conditions, the distance from the point of load to the point of maximum stress (X_σ) can be estimated as

$$X_\sigma = 0.5C_{\mathrm{lp}}/\tau \qquad (5)$$

where

C_{lp} is circumference at load point and

τ circumference taper.

For most distribution poles, Equation (5) suggests that the maximum stress location will be at groundline. The estimated maximum stress location for a class 1 55-ft pole, however, would be 6 ft above groundline for southern pine and 18 ft above groundline for the average western redcedar using tapers taken from Wood and others (1960) or median values reported by Bohannan and others (1974).

Of course, this purely geometric derivation provides only a rough approximation of how stresses vary in an actual wood pole. There are other variables to consider when predicting where a pole will fail. If maximum stress occurs at 6 ft above ground in dry wood and the groundline is saturated, it is likely that the groundline stress will exceed the saturated strength before maximum stress will exceed dry strength. Physiological effects such as knots, juvenile wood, strength change with height, and diving grain exposed in the debarking–peeling process also present points of weakness that might initiate a failure at a location other than the point of maximum stress.

***Material variability*—**Because wood is a product of nature rather than a manufacturing process, its strength is much more variable than that of most commonly used engineering materials. Wood strength varies not only between trees but also within a tree. Between-tree variability in strength is attributed to growing conditions (e.g., nutrients, rainfall, length of growing season, competition) and within-tree variability to factors such as the transition from juvenile to mature wood, the occurrence of branches and reaction wood, and growth rate, which are partly related to species and partly influenced by the environment.

***Change in strength with height*—**Change in strength with height and size effect are two overlapping yet independent issues used to characterize the perceived reduction in strength with pole size. When ANSI O5.1 adopted the H-classes in 1972, the specification treated these classes the same as the distribution pole classes in that the poles were sized as single pole cantilever beams. Unlike distribution poles, however, many of these larger poles were used in multi-pole structures (such as “H-frames”), which required cross-bracing that imposed critical stresses at close to mid-height above groundline. The concern for critical stress occurring at or above mid-height prompted Bohannan (1971) to evaluate change in pole strength with height and to propose a strength–height adjustment that defined the lower boundary of strength change with height on the basis of the limited data available at the time. This model took the form

$$F_x = F_{gl}\left(1 - 0.5\frac{X}{L}\right) \qquad (6)$$

In developing this model, Bohannan reviewed the limited available data on change in pole strength with height. The ASTM pole data provided data on failure strength as a function of location. Other sources provided information on incremental tests of pole sections from the butt to the tip and on specific gravity changes with height (Okkonen and others 1972). After collecting this information, Bohannan derived his model to characterize the lower boundary of strength as a function of location with respect to pole length. The model, based on measured dimensions of test poles, was intended to provide a conservative estimate of the strength of the pole at height x above the groundline. Annex A, which was introduced in 1976 and added in 1979, incorporated Bohannan’s model as nonmandatory information. It stated that the basic premise of the model is that it would be used only when actual dimensions are known and acknowledged that a reduction in fiber stress with height is not required when analyzing a pole based on minimum size and assumed linear taper values given in ANSI tables.

Soon after appendix A was added, this adjustment became a major issue of concern among ASC members as it held implications beyond the concern for multi-pole structures. Members realized that strength reduction also had an impact on smaller poles. One member showed that a 60-ft class-4 southern pine or Douglas-fir pole would have a critical section 26 ft above the groundline and require 3.5-in. greater circumference at that point.

Loss of strength with height continued to be an issue of debate into the early 1980s despite the fact that it was published in an annex and was not an official part of the ANSI main standard.

As part of the EPRI-sponsored pole research conducted in the 1980s, undamaged sections of test poles were retested to assess change in strength with height. The retest sample included 58 southern pine, 87 Douglas-fir, and 32 western redcedar poles. Strength change with height based on measured dimensions was compared to strength change based on ANSI minimum dimensions. Results confirmed that the conservatism inherent in the use of ANSI minimum dimensions and the assumption of a linear taper countered any strength loss with height for Douglas-fir and western redcedar. Bodig and Goodman (1986) and Phillips and others (1985) confirmed the efficacy of Bohannan's model for southern pine when based on measured dimensions.

When the data are reevaluated using minimum class dimensions, however, the relationship becomes

$$F_x = 9400\left(1 - 0.176\frac{X}{L}\right) \qquad (7)$$

This means the strength reduction is roughly 9% by mid-height when based on minimum class dimensions compared to 25% with actual dimensions.

Size effect**—**Weibull (1984) developed his extreme value distribution on the basis of a weak-link theory that predicts reduction in mean strength as size increases. Wood (1956, 1958) first noted a size effect in poles while working on the ASTM pole study and reported the effect for poles less than 50 ft in length. Erickson and Dohr (1958) showed a decrease in tree strength with age: trees more than 200 years old have lower strength than younger trees. The effect of size on load capacity has been noted for most engineered applications of wood (Bohannan 1966).

Figure 2 shows relative strength values for various species and pole size classes. The relative value in this case is expressed as a fraction of the 1963 DFS for southern pine (8,000 lb/in^2). Figure 2 suggests that for the ASTM tests, relative strength varies with pole size. However, it must be noted that the class 1-55 and 9-25 plots represented only 5 or 6 tests in each species, whereas the class 6-30 samples ranged from 15 to 43 tests. This observation did not attract much debate in 1963 when the ASA standard dealt only with distribution poles and design capacity was a relatively minor issue. It did, however, attract attention in 1980 when test values from the EPRI-sponsored study were low by comparison with ASTM values.

The change in strength with pole size is attributed in part to geometry and tree physiology. As noted previously, maximum stress in a tapered round cantilever of uniform properties will occur at the location where the circumference is 1.5 times that of the point of load. For poles of a given groundline dimension, the longer the pole the greater the chances of failing at a location significantly above groundline. The greater the distance between groundline and the failure location, the greater the assumed difference between groundline strength and groundline stress at failure. Because the poles in the ASTM study were evaluated on the basis of groundline stress at failure and not actual groundline strength, there is some question as to the source of the apparent size effect seen in that data. However, the majority of these poles would be expected to fail close to the groundline.

At the time the EPRI study results were published, the evaluation of groundline stress at failure was still considered to be the easiest way to deal with the evaluation of pole capacity. A tendency for a decrease in groundline stress at failure with an increase in pole size presented in the EPRI

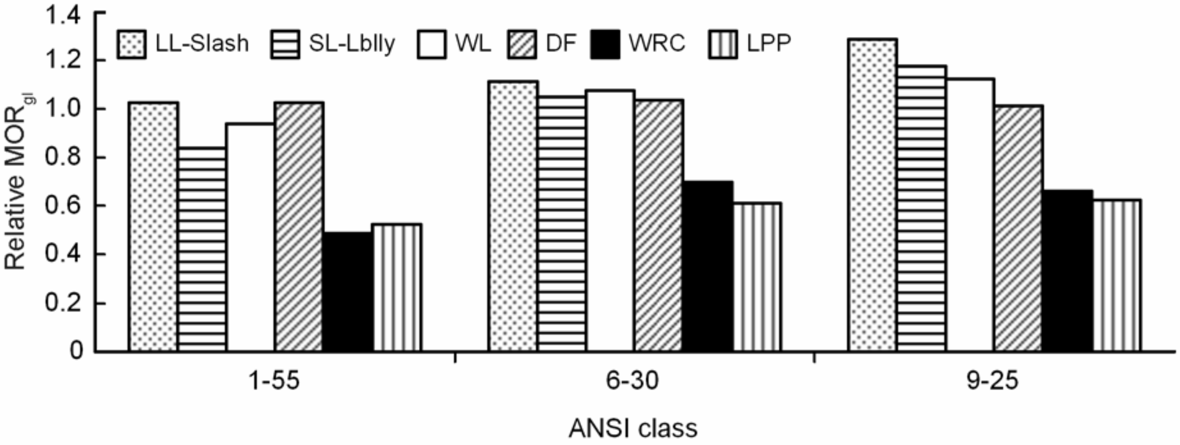

Figure 2—Relative strength (fraction of 8,000 lb/in^2) of five species in three size classes tested in ASTM wood pole research. LL is longleaf pine, SL shortleaf pine, Lblly loblolly pine, WL western larch, DF Douglas-fir, WRC western redcedar, and LPP lodgepole pine.

study raised some concern among committee members about acknowledging these results in the material specification. At that time, the ANSI O5.1 standard attributed no design significance to DFS values. These values were used only as a basis for establishing the pole circumference 6 ft from the butt for the ANSI pole size classification. The premise of species interchangeability inherent in this classification system was not violated if size effect was treated as a design consideration, rather than a material variable. The committee decided to acknowledge the EPRI results in Annex C as additional information for those interested in promoting the application of probability-based design to utility structures. In this way, the burden of using or ignoring these effects was left on the designer with relatively minor impact on the ANSI standard. Without code recognition by the NESC, probability-based design was not used for design. Nevertheless, interest in the Annex C values continued and this information provided the basis for calibration studies of competing design methods.

Annex C

Results of the EPRI-sponsored research to promote the use of probability-based design for wood transmission structures were summarized in the form of a condensed introductory guideline and published as Annex C of the ANSI O5.1 standard specification. This guideline was written to be compatible with the established groundline capacity classification system presented in the ANSI O5.1 standard. To address the size effect made apparent by this analysis, the technical advisory committee to that project recommended separating the data into two distinct length categories (<50 ft, ≥50 ft) with a discrete reduction in strength at the 50-ft length rather than adopting a continuous function to account for size effect.

In the evaluation of these strength values, all data were adjusted to the minimum groundline dimensions corresponding to the ANSI class of each pole rather than using the measured dimension. The groundline circumferences used to determine groundline bending strength were the ANSI-class minimum values. These are the values an engineer assumes for an "ANSI pole" in calculating pole moment capacity. The values presented in Annex C are therefore appropriate only for use with ANSI-class pole minimum dimensions.

In addition to differentiating by length and adjusting for the ANSI-class minimum dimensions, the larger pole values were adjusted to remove any pole size bias. As the EPRI study indicated, the effect of groundline circumference on pole strength and test poles was greater for western redcedar (largest pole H4-70) and Douglas-fir (largest pole H2-75) than for southern pine (largest pole 1-75), leading to the conclusion that the southern pine data had an inherent size bias. As a result, ASC O5 felt that all test values should be adjusted to the equivalent class 2-65' circumference value. This was done using a linear regression between groundline stress at failure adjusted for ANSI minimum dimension ($AMOR_{gl}$) and the ANSI 6-ft-from-butt circumference (c)

$$AMOR_{gl} = a + bc \tag{8}$$

Regression parameters (a, b) are provided in Table 7 along with the ANSI minimum circumference at groundline for the class 2 65-ft poles (46.5 for Douglas-fir and southern pine, 51.5 for western redcedar). Groundline stress values derived for poles greater than or equal to 50 ft in length (Bodig and others 1986a,b) are shown in the last column. The difference between mean distribution pole values (test poles ranging from 25 to 50 ft) (ANSI O5.1; App. C, Table C1) and the values for an ANSI class 2 65-ft pole (ANSI O5.1; App. C, Table C2) represents the effect of pole size on groundline stress at failure.

Despite the effort to keep design information separate from the material specification, the ASC O5 was forced to address disparities highlighted by the Annex C guideline. An initiative by the Institute of Electrical and Electronic Engineers (IEEE) to revise utility structure design procedures of the NESC raised questions about the efficacy of the ANSI DFS values for design of transmission structures. The primary concerns included the effect of pole size and the bending strength values found for western redcedar poles. The IEEE requested a recommendation from the ASC O5 on values to use for design to retain a uniform reliability across species and class sizes. This prompted the ASC O5 to consider options for recognizing disparities between the ANSI fiber stress values adopted in 1963 and those determined on the basis of larger pole tests summarized in Annex C. An initial recommendation to recognize the effect of pole size was to use a stepped allowable fiber stress. With this option, large poles would have a single, reduced DFS that would be less than the published value for small poles. This would result in a sudden jump in the required circumference when that transitional pole size was reached. It seemed awkward and unrealistic that a slightly larger pole would suddenly have less strength, and this too was rejected.

Table 7—Regression parameters to account for size effect on poles (Bodig and others 1986)

Species	Sample size	a	b	ANSI 2-65 circumference (in.)	$AMOR_{gl}$ (lb/in^2)
Douglas-fir	248	12,332	–96.1	46.5	8,890
Southern pine	263	12,589	–89.4	46.5	10,190
Western redcedar	487	7,952	–53.5	51.5	6,310

Wolfe and others (2001) proposed the use of a continuous function that would maintain the traditional approach to pole capacity evaluation, but it was rejected as too complicated and too costly for pole producers. This approach retained a relationship between the groundline moment capacity (M_{gl}) and groundline circumference (C_{gl}) similar to that used previously (Eq. (9)) but with parameters that were derived empirically from a statistical evaluation of the 5% lower exclusion bending strength versus groundline circumference (Eq. (10)).

$$M_{\mathrm{gl}} = \sigma k C_{\mathrm{gl}}^{\ 3} \tag{9}$$

$$M_{\mathrm{gl}} = A k \gamma C_{\mathrm{gl}}^{\ B+3} \tag{10}$$

For poles used only in distribution structures, the value of "*B*" was set to zero giving an equation identical to Equation (9) except that the *A* parameter represented a L5%EL value for bending strength. A calibration factor γ is also included to adjust for the lower values found for western redcedar.

For the three species used for transmission structures (southern pine, Douglas-fir, and western redcedar), the *B* parameter had a negative value. This means that the groundline moment capacity did not increase as rapidly with an increase in groundline circumference as it did using Equation (9), thus accounting for the observed size effect.

This approach retained the simplicity of the ANSI approach to pole classification as well as species interchangeability, but it required too many changes to the established ANSI values. The perception that this would cause significant changes in class sizes prompted many producers to conclude that transition costs would be prohibitive. Targeting a lower 5% exclusion value for bending rather than a mean value meant that the class loads would have to be reduced. Despite the fact that this could be done with little impact on class circumference values and would be compensated by warranting higher resistance factors in design, it gave the appearance of reducing the capacity of wood poles. ANSI DFS values had traditionally represented a value close to the mean strength at groundline for distribution poles: 5% lower exclusion values are roughly 30% less than the traditional DFS values. Finally, this approach acknowledges a larger gap between the ANSI DFS and measured mean strength of western redcedar than exists for the other species. Members of ASC felt this was not adequately substantiated. They explained the strength gap as the result of deficiencies in the test and analysis procedures when evaluating a species with large taper in the lower half, which caused failures to occur higher in the pole giving a groundline moment at failure that was not representative of groundline moment capacity.

ANSI O5–02

Rather than change the method of deriving DFS values and the tabulated circumferences, ASC O5 opted to change the definition of DFS from a relative measure of pole groundline moment capacity to an estimate of groundline strength (see Appendix A). Prior to 2002, the DFS was evaluated as the groundline stress at failure for a dry, treated pole loaded as a cantilever beam with a transverse load applied 2 ft from the top. The sole ANSI O5.1 use of this value was to represent a relative measure of pole capacity as a basis for pole size classification. A number of attempts were made by committee members (Colley 1932, Wilson and others 1953) to attach a level of statistical significance to the derivation, but their assessments were all based on assumptions that were never supported by data or any rigorous analysis of field performance. The ASA and later the ANSI standards were strictly material specifications that presented the DFS values as only a relative measure of pole strength.

In 2002, the DFS was redefined as the fiber strength of dry, treated wood. The ANSI standard added statements of significance that these values represent the means of distribution of groundline fiber strength and that these distributions are known to have a COV of 20%. The standard states that these evaluations were made with sufficient precision to be used for probability-based design. The new definition introduced an acceptance of design liability by ASC O5 as the DFS value is now defined to represent wood strength at a specific location as opposed to relative pole capacity used simply to set pole size classes.

Using data provided by the EPRI project (Phillips and others 1985, Bodig and others 1986) (Appendix C) the committee derived groundline strengths ($\mathrm{MOR_{gl}}$) for poles in lengths of 50 ft and longer on the basis of measured modulus of rupture (strength) at the failure location ($\mathrm{MOR_{fl}}$) and the strength–height relationship proposed by Bohannnan (1971). Prior to 2002, DFS values included adjustments for variability in test data attributable to handling and test procedures (0.93), high-temperature conditioning, and drying in service. This approach was retained for poles shorter than 50 ft based on the assumption that they normally fail at the groundline. For the 2002 revision of the ANSI O5.1 standard, however, the ASC O5 supported dropping the adjustment for variability and accepting a maximum 10% increase for drying in service, originally recommended by Wood and Markwardt (1965) for longer poles.

A new adjustment, also adopted for long poles, was a class-oversize adjustment that varies from 7% to 13%, depending on the pole class size. Due to its variation with class size, this adjustment requires an iterative procedure to match the product of adjusted DFS and average class section modulus to a designated class load moment (discussed in Appendix C). This adjustment decreases the probability that poles within a size class will have a groundline bending moment capacity that exceeds that imposed by the designated class load (see Appendix A). The resulting values are shown in the column labeled "DFS" in Table 1. Comparing these DFS values to the DFS values that would have been derived using the previous definition shows a 14% advantage for Douglas-fir and southern pine and a 44% advantage for western redcedar.

In 2002, groundline strength of poles in lengths 50 ft and longer were extrapolated from the failure location using the strength–height model previously presented in Annex A of the ANSI O5.1 standard. As change in strength with height is now an inherent part of the evaluation of groundline strength, it is required that this model be used when assessing the strength profile for ANSI poles. Adoption of this model to estimate groundline strength, however, detracts from species interchangeability. Groundline circumference values are still calculated on the basis of section modulus required to resist the class-load-induced groundline moment. Using the projected groundline fiber strength rather than the groundline stress at failure means that the groundline dimension does not account for the occurrence of a failure above groundline. The larger the taper of a pole, the greater the distance from groundline to the point of maximum stress and the greater the difference between groundline stress at failure and groundline strength. Unless the groundline circumferences are adjusted to account for this, poles of the same class but different taper will have different load capacity

While the decision to change the DFS value from a relative measure of pole capacity to a measure of bending strength applicable only at groundline constituted a significant change to the standard, on its own this decision had a minor impact on solving the problem of size effect that was apparent in the test data. The change that had the greatest impact on maintaining the traditional DFS values was the adoption of a classification oversize adjustment.

The analysis presented in Appendix C shows that the projected MOR at groundline, when adjusted for only conditioning and drying in service (5,530, 6,950, and 7,720 lb/in^2 for western redcedar, Douglas-fir, and southern pine, respectively) is still significantly less than the ANSI DFS values for these species (6,000, 8,000, and 8,000 lb/in^2, respectively). Adjusting DFS values so that the average, rather than minimum-sized, pole in a class will be sized to carry the ANSI-designated class load gives DFS values of 6,040, 7,640, and 8,530 lb/in^2, respectively.

The effects of these changes on ANSI values are discussed in Appendix A. Small poles are still sized using DFS values that were derived assuming a 16% increase for drying in service but with no increase for class oversize. Long poles (>50 ft) are assumed to have a dry strength 10% greater than the measured green value. But they are also given a 7% to 13% increase for class oversize. Despite the fact that poles are sized using values adjusted for drying in service, ANSI notes that the tabulated circumferences are intended to be measured in the green condition and that a reduction in section modulus of up to 6% due to shrinkage should be expected. This means that a minimum-sized pole of average strength and more than 50 ft in length could be overstressed by as much as 24% (1.1 × 1.13), whereas the average-strength minimum-sized distribution pole would be overstressed by 16% if required to carry its class load in the green condition. If loaded in the dry condition, an average strength minimum-sized long pole would be overstressed by 13% to 20% at the class load (6% due to shrinkage and 7% to 13% for the class oversize adjustment), whereas the smaller, minimum-sized distribution pole would be only 6% under capacity at the designated class load.

The fact that the probability of failure of the average ANSI pole loaded to its class load in a standard cantilever test went from 50% or more (depending on strength variability assumed) prior to 2002 to 70% or more after 2002 (see Appendix A) is really not a critical issue as long as the pole user realizes that class loads are used simply to classify poles. There should be some concern, however, that the new definition of DFS for long poles has been inflated by 7% to 13% above dry strength and a dry pole is likely to have a section modulus 6% below that of a green pole. This suggests that the DFS for long poles should be reduced by 20% when calculating the groundline strength for design purposes.

The most obvious drawback to this change is the confusion it creates over the meaning of the ANSI fiber stress value. The engineer will still use the minimum dimensions permitted for the pole class selected to assess capacity. The pole selection must be made by first assessing the load profile. Using the ANSI dry DFS and the change in strength and circumference along pole length, the engineer must determine the required minimum section property profile for portions of the pole expected to dry in service and then decrease the DFS by 10% (16% for short poles) to check section property required in any wet regions. Once a required profile has been established, it can be compared to ANSI-tabulated dimensions to select a class size. The difference in average failure probability at the designated class load for long and short poles will most likely be of little importance provided the design engineer references the tables simply as a catalog of ANSI size codes.

Conclusions

- The practice of classifying green poles by groundline circumference determined on the basis of dry strength is inappropriate for design applications. The ANSI standard should derive class dimensions on the basis of green strength and suggest an adjustment for drying above groundline.

- The practice of sizing long poles so that the average size pole is defined by the minimum required section modulus and of sizing short poles so that the smallest permissible size pole is defined by the minimum required section modulus leads to inconsistency in pole reliability. ANSI should apply the same bias to all pole classes.

- For species having ANSI DFS values derived from published small–clear tests, values should be modified to reflect the changes implemented in ASTM D 2555.

- It is more conservative for taper values used to project 6-ft-from-butt dimensions to err on the high side. Comparison of taper values reported in Annex B of the ANSI O5.1 standard appear low by comparison to those determined by Bohannan. This could result in undersizing for poles more than 40 ft in length.

- The relationship between strength and pole size is not adequately characterized by the strength-reduction-with-height function adopted by ANSI. The relationship between pole size and strength as evaluated over the past 50 years suggests that it would be technically more correct to reduce the allowable fiber stress with increasing pole size (volume) rather adopting an untested model to project groundline strength from strength at the failure location.

- Pole design should focus on differences between stress and strength profiles, setting design as a fraction of the load at which the maximum difference is zero. The ANSI O5.1 standard currently instructs users to consider change in strength with height only when the point of maximum stress occurs above groundline. The DFS derivation adopted in 2002, however, extrapolates strength from the point where the difference between stress and strength are the greatest (failure location); not the point of maximum stress. Suggesting a design approach to prevent stress from exceeding strength would add credibility to the design method presented and would make wood pole strengths comparable to that of other materials.

- The wood pole industry should continue to work on refining its knowledge of pole physiology and shape to provide more reliable wood pole structures and a more credible standard. Also, if the producers can be encouraged to revise the class circumference tables, then interspecies exchangeability can be restored.

Observations

The ANSI O5.1–02 standard appears to magnify the disparities between the main body of the standard and Annex C. Annex C presents what has been viewed as technically correct and compatible with established timber engineering methods compromised by what is politically and economically acceptable to users and producers. By keeping design information in an annex, separate from the material specification, the committee was able to maintain the standard size classifications established 40 years ago, despite the realization that they did not represent uniform reliability across species or across classes within a given species.

The methods outlined in Annex C brought ANSI up-to-date with utility-structures engineering, but they suggest a need to recognize a size effect when designing wood pole transmission structures. The committee responsible for the ANSI O5 standards (ASC O5) has attributed the perceived problem of a size effect to change in strength with height. The committee must decide if it wants ANSI O5.1 to remain a material specification or to assume the role of a wood pole design standard. The standard needs a stronger technical basis to bring it into alignment with Annex C.

Before ANSI O5.1 moves too far into the realm of a design standard, it would be beneficial to develop technical support for assumptions made and to more carefully consider their impact on the market for utility poles. The fact that poles are generally classified in the green condition means that groundline section modulii derived using the new “dry” definition of DFS values for large poles may result in undersized poles. A size class adjustment is appropriate only when converting the DFS, used to set size class dimensions, to a design value. It is not appropriate to adjust test data for size class effect to derive a size class dimension as it leaves the false impression that all poles having a strength greater than or equal to the DFS in a given size class have the capacity to carry the designated class load.

If loads on a large pole are located to induce maximum stress at groundline, where moisture could be at or above fiber saturation point, a pole whose required groundline dimension is derived on the basis of dry strength will be undersize. Thus, in addition to including a reduction in strength with height, the standard should include a moisture adjustment for critical sections being within 4 ft of the groundline.

The fact that this new derivation requires recognition of a change in strength with height means that this becomes mandatory for all poles. The committee is basically assuming that small poles fail close to the ground where change in strength with height is minimal. The strength change profile (Eq. (7)) is undoubtedly conservative in the lower half of trees; even for small poles it indicates that the critical stress location is above groundline. The result is that the strength change with height evaluation is required for any poles designed using the new ANSI standard. If the committee wishes to design poles with this degree of sophistication, the strength profile must be derived to present a more accurate picture, not only of fiber strength change but of section property change.

Literature Cited

ANSI. 1997. Specifications and dimensions for wood poles. New York, NY: American National Standards Institute.

ASTM. 1966. Standard methods for establishing clear wood strength values. Standard D 2555–66. West Conshohocken, PA: American Society for Testing and Materials.

ASTM. 1999. Standard test methods of static tests of wood poles. Standard D 1036–99. West Conshohocken, PA: American Society for Testing and Materials.

ASTM. 2000. Standard test methods for small clear specimens of timber. Standard D 143–94. West Conshohocken, PA: American Society for Testing and Materials.

Betts, H.S.; Newlin, J.A. 1915. Strength tests of structural timber treated by commercial wood preserving processes. Bull. 286. Washington, D.C.: U.S. Department of Agriculture.

Bhuyan, G.; Chetwynd, D. 1994. Strength and stiffness of new wood utility poles. Rep. 119D 1010. Montreal, Canada: Canadian Electricity Association.

Bodig, J.; Goodman, J.R. 1986. Western redcedar data and size effect. Wood pole properties. Review and recommendations for design resistance data. EPRI EL–4109, Vol. 3. Palo Alto, CA: Electric Power Research Institute.

Bodig, J.; Goodman, J.R.; Brooks, R.T. 1986a. Western redcedar data and size effect. Wood pole properties. Review and recommendations for design resistance data. EPRI EL–4109, Vol. 3. Palo Alto, CA: Electric Power Research Institute.

Bodig, J; Goodman, J.R.; Phillips, G.E.; Fagan, G.B. 1986b. Douglas-fir data—Wood pole properties. Review and recommendations for design resistance data. EPRI EL–4109, Vol. 2. Palo Alto, CA: Electric Power Research Institute.

Bohannan, B. 1966. Effect of size on bending strength of wood members. Res. Pap. FPL–RP–56. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Bohannan, B. 1971. Height–strength relationship for round timbers. Working document for ANSI O5.1 committee use.

Bohannan, B.; Habermann, H.; Lengel, J.E. 1974. Taper of wood poles. Gen. Tech. Rep. FPL–GTR–2. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Buckman, S.J.; Rees, L.W. 1938. Effect of steaming on strength of southern yellow pine. In: Proceedings, American Wood Preservers' Association. 34: 264–296.

CSA 1990. Wood utility poles and reinforcing stubs. CAN/CSA O15–90. Etobicoke, Ontario: Canadian Standards Association.

Colley, R.H. 1932. Ultimate fiber stresses for wood poles. Bell Telephone System, Tech. Pub. Monograph B615. New York, NY: Bell Telephone Laboratories.

Eggleston, R.C. 1952. Pole strength tests. Forest Products Journal. April, pp. 3–24.

Erickson, E.C.O.; Dohr, A.W. 1959. Effect of steam conditioning on the strength of treated longleaf pine poles. Interim rep. Madison, WI: ASTM Wood Pole Program P&E.

Goodman, J.R.; Vanderbilt, M.D.; Criswell, M.E.; Bodig, J. 1981. Probability based design of wood transmission structures—strength and stiffness of wood poles. EPRI EL–2040, Vol. 1. Palo Alto, CA: Electric Power Research Institute.

Jones, R.L. 1931. New standard specifications for wood poles. Bell System Technical Journal, September, pp. 514–524.

Kulp J.W. 1957. Second report on tests of service life of poles in REA-financed electric systems. REA Bull. 169–32. Washington, D.C.: U.S. Department of Agriculture, Rural Electrification Administration.

MacLean, J.D. 1951. Rate of disintegration of wood under different heating conditions. In: Proceedings, American Wood Preservers' Association. 47: 155–168.

MacLean, J.D. 1953. Effective of steaming on the strength of wood. In: Proceedings, American Wood Preservers Association. 49: 88–112.

National Bureau of Standards. 1927. National Electrical Safety code, initiated 1913.

Newlin. J.A.; Trayer, G.W. 1924. Form factors of beams subjected to transverse loading. Rep. 181. National Advisory Committee for Aeronautics. Reprinted October 1941 as Rep. 1310. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Okkonen, A.; Wahlgren, G.; Maeglin, R. 1972. Western wood density survey. Forest Products Journal. 22(7): 37–41.

Phillips, G.E.; Bodig, J.; Goodman, J.R. 1985. Background and southern pine data. Wood pole properties. Review and recommendations for design resistance data. EPRI EL–4109, Res. Proj. 1352–2, Vol. 1. Palo Alto, CA: Electric Power Research Institute.

Rhatigan, R.G.; Morrell, J.J. 2002. Seasonal variations in moisture content of in-service poles in the Willamette Valley. In Proceedings, International conference on utility line structures. Ft. Collins, CO: Engineering Data Management Inc. and Colorado State University.

Stamm, A.J. 1956. Thermal degradation of wood and cellulose. Industrial Engineering Chemistry. 48: 413–417.

Thompson, W.S. 1969. Factors affecting variation in compressive strength of southern pine piling. In: Proceedings. Washington, DC: American Wood Preservers' Association.

USDA. 1940. Wood handbook. Washington, D.C.: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 326 p.

USDA. 1955. Wood handbook. Washington, D.C.: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 528 p.

Wilkinson, T.L. 1968. Strength evaluation of round timber piles. Res. Pap. FPL–RP–101. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Wilson, T.R.C. 1923. Results of some strength tests on wooden poles. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Wilson, T.R.C.; Drow, J.T. 1953. Fiber stresses for wood poles. No. D1619. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Wilson, T.R.C.; Carlson, T.A.; Luxford, R.F. 1930. The effect of partial seasoning on the strength of wood. In Proceedings. Granbury, TX: American Wood Preservers' Association.

Wolf, J. 1979. Bending strength of roundwood timber poles: a summary and evaluation of literature. Unpublished. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Wolfe, R.W. 2000. Design stress derivation for ANSI poles. In: Proceedings, International conference on utility line structures; 2000 March. Ft. Collins, CO: Colorado State University and Engineering Data Management Inc.

Wolfe, R.W.; Bodig, J.; Lebow, P.K. 2001. Derivation of nominal strength for wood utility poles. Gen. Tech. Rep. FPL–GTR–128. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Wood, L.W. 1956. Testing poles for the American Society for Testing and Materials. Interim rep. P&E 134. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Wood, L.W. 1958. Strength tests of wood poles. Interim rep. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Wood, L.W.; Markwardt, L.J. 1965. Derivation of fiber stresses from strength values of wood poles. Res. Pap. FPL–RP–39. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Wood, L.W.; Erickson, E.C.O.; Dohr, A.W. 1960. Strength and related properties of wood poles. ASTM final rep. Conshohocken, PA: American Society for Testing and Materials.

Appendix A—Effect of 2002 Revision on Pole-Class Strength Distribution

In 2002, six basic changes were made to the ANSI standard that influence the definition of the designated fiber stress (DFS) for poles in lengths greater than or equal to 50 ft in length (referred to as "long" poles in this appendix). These changes were as follows:

1. Redefining DFS to be groundline strength rather than groundline stress at failure
2. Adopting a change in strength with height function as part of the evaluation of pole properties
3. Dropping recognition of the influence of test procedure variability from derivation of strength of long poles
4. Incorporation of a class oversize adjustment for long poles
5. A reduction in the long pole strength adjustment for drying in service
6. Declaration of a 20% coefficient of variation for pole bending strength

The change in definition of DFS allowed ANSI to acknowledge the observed "size effect" (Bodig and Goodman 1986) discussed in Annex C of the O5.1 standard as being a result of a change in strength with height rather than a groundline dimension. The advantage of this change is that it provides a basis for retaining established DFS and therefore the ANSI class groundline dimensions for all pole classes.

To justify maintaining the same DFS for long poles, ANSI adopted as mandatory the change-in-strength-with-height function previously presented in the "non mandatory" Annex A. The inverse of this function was used to estimate groundline strength for long test poles that failed above groundline. As the inverse of this function was used to evaluate groundline strength, the function becomes a mandatory adjustment for anyone concerned about the strength of poles in locations other than the groundline.

The 2002 derivation of DFS for long poles did not include the adjustment for variability inherent in the established DFS values. The "variability" adjustment, originally introduced in 1931 and re-approved in 1963, was proposed to counter the effect of testing poles that had unknown moisture gradients at the time of testing. This was basically a confidence adjustment that reduced the measured mean by a half standard deviation. In 1963, ASA O5 elected to use a 14% coefficient of variation for bending strength of poles on the basis of results published in the ASTM wood pole report (Wood and others 1960). As there were no changes to the derivation of DFS for poles under 50 ft in length in 2002, the variability adjustment (0.93) is still considered to be an inherent part of those values. This adjustment was not used, however, in the 2002 derivation of DFS for long poles (≥50 ft).

In 2002, ANSI adopted an adjustment for class oversize effect for long poles. This adjustment was previously acknowledged as a design consideration (Annex C of the ANSI O5.1 standard), but it was not included in the previous derivation of DFS values, which are used to set the pole class dimensions. This adjustment basically realigns the designated class load with the median-size rather than the minimum-size pole in a given size class. Multiplying the minimum allowable section modulus for a pole class–length combination by the DFS thus gives the moment capacity of the median "long" pole and the minimum "short" pole. This suggests that for a given pole class, an average strength median sized "long" pole and an average strength minimum sized "short" pole have the groundline moment capacity implied by their designated class load.

Other changes adopted in 2002 that affect the conclusions about the ability of a pole to resist the implied groundline moment capacity include the adjustment for drying in service and the assumed strength variability. In 1963, two drying-in-service adjustments were considered, 1.1 and 1.16. The latter value was preferred for derivation of DFS for the primary pole species. In 2002, an adjustment of 1.1 was adopted for long poles. The implication of this change is that short poles that are more likely to be critically stressed at groundline have a larger increase for drying in-service than are poles more likely to fail above ground.

As for the variability of pole bending strength, the 2002 version of ANSI O5.1 declares that wood poles have a strength coefficient of variation of 20%. This value is higher than previously referenced by ANSI and higher than that obtained for data presented in the ASTM study (Wood and others 1960) or the EPRI-sponsored study (Bodig and others 1986).

The following analysis provides a comparison of pole strength as defined by ANSI standards before and after the 2002 revision. All assumptions are based on values either provided in ANSI O5.1 standards or taken from discussions that support those values. This analysis includes only those parameters that were modified in 2002. It assumes that removing adjustments for variability, drying in service, and class oversize effects from ANSI DFS values truly represents mean groundline strength of green poles adjusted for the effects of pretreatment conditioning.

This analysis also focuses only on the strength at the groundline. There is little support for the premise that

Table A1—Normalized distribution parameters for ANSI poles before and after 2002

	Distribution parameter			
	Long poles[b]		Short poles[c]	Pre-2002
Variable[a]	H6	2	All	All
Drying in service	1.10	1.10	1.16	1.16
Variability	1.0	1.0	0.93	0.93
Class oversize	1.06	1.11	1.0	1.0
Strength COV	0.2	0.2	0.2	0.14
Normalized mean (λ)	0.858	0.819	0.927	0.927
Normalized SD	0.172	0.164	0.216	0.151

[a] SD is standard deviation.
[b] Poles ≥50 ft.
[c] Poles <50 ft.

strength change with height is the same for all species and pole classes, and we know that tree geometry varies with species as well as growing conditions. For this comparison of effects of the 2002 change, it is not necessary to address that issue except to acknowledge that the adjustment for change in strength with height function adopted by ANSI adds a degree of conservatism to the analysis of pole capacity.

Table A1 lists normalized distribution parameters for pole strength implied by the ANSI derivation of DFS. Three variables listed (drying in service, variability, and class oversize adjustment) are those that vary between “long” and “short” poles in the 2002 standard and between long poles before and after 2002. The “normalized mean” value is the ratio of the mean strength in the green treated condition to DFS. The normalized strength distributions thus have a mean value (λ) with the same coefficient of variation (COV = Ω) as the actual pole strengths. For example, on average, a long, class 2, “green” pole will have a mean strength that is 82% of the DFS value and a standard deviation (λΩ)of 0.16. Mean strength is less than DFS because the DFS includes increases for drying and class oversize. Short poles have a mean strength of 93% of the DFS because of the 7% reduction for variability in combination with the 16% increase for drying (1/[(1.16)(0.93)]).

A distribution of pole moment capacities can be generated using a method known as random-products generation. This is done by randomly choosing a normalized strength from a normal distribution having a mean value λ and a coefficient of variation Ω and multiplying by a randomly chosen normalized section modulus (random class value divided by the minimum required for the class). The section modulus is selected assuming a uniform distribution with a range from 1.0 to the maximum for the pole class (see Table A2). Repeating this process 6,000 times for each of the four scenarios provides a basis for evaluating the effect of the 2002 changes on implied pole capacity.

Table A2—Maximum value for uniform distribution of normalized section moduli

Class	2	1	H1	H2	H3	H4	H5	H6
Max	1.22	1.20	1.19	1.17	1.16	1.15	1.14	1.13

As the revisions for long poles incorporated in 2002 shift average capacity from the minimum-size to the median-size pole in the group, it is not surprising that fewer poles within a pole class have the groundline moment capacity implied by the designated class load. The comparison of load capacity distributions shows that at the 5th percentile, the load capacities for the H6 and the class 2 long poles were 57% and 55% of the implied capacity, respectively. For the short poles, the 5th percentile value was 62% of implied capacity; for class 2 poles evaluated prior to 2002 ANSI parameters, poles at the 5th percentile had 73% of the implied capacity. The mean strength of the H6 and class 2 long poles was 91% of the implied capacity. As stated earlier, this is due to the use of a dry strength being used to size a green pole. Values for short poles and the pre-2002 analyses showed the mean strength to be 102% of implied capacity.

Overall, only 35% the distribution of long poles was capable of carrying the implied groundline moment. The 2002 analysis of short poles and the pre-2002 analysis resulted in more than 54% of poles in a given class having capacity that exceeded that implied by the designated class load.

This analysis suggests that the ANSI 2002 change created a gap in the inherent capacity of short and long poles. This is of minor consequence for ANSI O5.1. Pole sizes corresponding to any ANSI class are the same as they were prior to 2002. For the utility engineer, however, it is noteworthy that only slightly more than a third of the long poles selected on the basis of groundline moment divided by the ANSI DFS have the groundline moment capacity to resist that DFS in the green condition.

For those concerned about the future use of round poles as engineered components, there is a definite advantage to cooperating with those committees responsible for design of utility structures (ANSI C2/NESC and ASCE committee on wood) to assure that they understand the basis for the difference between the ANSI DFS as applied to “long” and “short” poles.

Appendix B—Wood Pole Test Data Sources

Alexander, J.B. 1934. Strength of lodgepole pine telephone poles. Circular 42. Vancouver, B.C.: Western Forest Products Laboratory.

Alpin, E.N. 1967. Strength of red pine poles. Rep. No. OP–X–10. Ottawa, Ontario: Eastern Forest Products Laboratory.

Anon. 1947. Strength of telephone poles of eastern cedar, red pine, and jack pine. Circ. 31. Ottawa, Ontario: Eastern Forest Products Laboratory.

Betts, N.W.; Hein, A.L. 1914. Tests of Rocky Mountain woods for telephone poles. Bull. 67. Washington, D.C.: U.S. Department of Agriculture, Forest Service.

Bodig, J.; Goodman, J.R. 1986. Wood pole properties. Vol. 3, Western redcedar data and size effect. EPRI EL–4109, Res. Project. 1352–2. Palo Alto, CA: Electric Power Research Institute.

Bodig, J; Goodman, J.R.; Phillips, G.E.; Fagan, G.B. 1986. Wood pole properties. Vol. 2, Douglas-fir data, EPRI EL–4109, Res. Proj. 1352–2. Palo Alto, CA: Electric Power Research Institute.

East Kentucky Power Cooperative Inc. 1980. Eleven new poles. Tested at Colorado State University, Fort Collins.

Fox, S.P. 1977. Strength of ACA and PCP treated white spruce, western hemlock, and amabilis fir. Unpublished report. Vancouver, B.C.: Western Forest Products Laboratory.

Goodman, J.R.; Vanderbilt, M.D.; Criswell, M.E.; Bodig, J. 1981 Probability-based design of wood transmission structures—strength and stiffness of wood poles. EPRI EL–2040, Vol. 1. Palo Alto, CA: Electric Power Research Institute.

Idaho Power Co. 1981 Forty-nine western redcedar poles removed from service and tested at Colorado State University, Fort Collins, Colorado.

Jessome, A.P. 1955. An investigation of the bending strength of 25-foot red pine poles by the machine test method. Progress Rep. No. 9, Proj. No. 0–74–1. Ottawa, Ontario: Department of Northern Affairs and National Resources, Forest Products Laboratory.

Jessome, A.P. 1960. An investigation of the bending strength of 35-foot untreated lodgepole pine poles by the machine test method. Department of Northern Affairs and National Resources Progress. Progress Rep. No. 5, Proj. No. 0–74–1, Ottawa, Ontario: Forest Products Laboratory.

Kennedy, D.E.; Wakefield, W.E. 1948. The strength of jack pine poles infected with pocket rot. Circ. 65. Ottawa, Ontario: Canadian Dept. Mines and Resources, Domestic Forest Service, Forest Products Laboratory.

McElhannay, T.A.; Perry, R.S. **1928.** Tests of relative strength of green-cut and fire-killed western cedar pole timber. Circ. 22. Forest Service, Department of Interior. Canada.

McGowan, W.M. 1960. Strength of Douglas-fir telephone poles. Tech. Note 15. Vancouver, B.C.: Western Forest Products Laboratory.

McGowan, W.M. 1962. Strength of western hemlock power and communication poles. Tech. Note 27. Vancouver, B.C.: Western Forest Products Laboratory.

McGowan, W.M.; Smith, W.J. 1965. Strength and related properties of western redcedar poles. Dept. Pub. 1108, Vancouver, B.C.: Western Forest Products Laboratory.

Platt, J.C. 1955. Breaking strength of red pine poles. Rep. 55–587. Ontario, Canada: Hydro-Electric Power Commission.

Platt, J.C. 1958. Strength tests of jack pine poles. Rep. 58–149. Ontario, Canada: Hydro-Electric Power Commission of Ontario.

Platt, J.C. 1958. The strength tests of wood poles: western redcedar and red pine. Rep. 58–206. Ontario, Canada: Hydro-Electric Power Commission.

Phillips, G.E.; Goodman, J.R.; Bodig, J. 1986. Wood pole properties. Vol. 1, Background and southern pine. EPRI EL–4109, Res. Proj. 1352–2. Palo Alto, CA: Electric Power Research Institute.

Public Service Company of Colorado. 1980. Twenty-five new mechanically pretreated poles. Tested at Colorado State University, Fort Collins.

Thompson, W.S. 1969. Effect of steaming and kiln drying on the properties of southern pine poles. Part i. Mechanical properties. Forest Products Journal. 19(1): 21–28.

Wilson, T.R.C. 1917. Results of some strength tests on wooden poles. Madison, WI: U.S. Dept. of Agriculture, Forest Service, Forest Products Laboratory.

Wood, L.W.; Erickson, E.C.O.; Dohr, A.W. 1960. Strength and related properties of wood poles. West Conshohocken, PA: American Society for Testing and Materials.

Appendix C—Data Used to Derive DFS for Long Poles

In 2001, Martin Rollins of H.M. Rollins Company proposed an alternative approach to accounting for the observed "size" effect in wood poles on behalf of the North American Wood Pole Coalition. His proposal was unique in that it recognized a size effect but did not require any modification to the ANSI classification tables. He presented this proposal to the ANSI O5.1 committee in the form of a committee report titled *Evaluation of Fiber Stress Values for Wood Utility Poles in ANSI Standard O5.1.* Although it never received critical review outside the fiber stress subcommittee, the report played a significant role in the committee's decision to adopt proposed changes to the ANSI standard.

Essentially, Rollins attributed "size" effect to a "height" effect. His premise was that the groundline strength does not change with increased circumference, but the load capacity as characterized by groundline stress at failure goes down because longer poles fail further from the ground in weaker wood. His solution was to change the definition of "designated fiber stress" (DFS) from groundline stress at failure to groundline strength. Rather than use a groundline strength derived from published small–clear strength test data, he decided to estimate groundline strength by adjusting the modulus of rupture (MOR) of full-size poles tested in the green condition. The primary adjustment accounted for the change in strength with height. This Rollins borrowed from annex A of the existing ANSI standard. Other adjustments included a 10% increase for drying in service, a pretreatment conditioning adjustment (0.9 for kiln drying and boultonizing, 1.0 for air drying), and a classification oversize adjustment that varied from 1.07 for a class H4–70-ft pole to 1.117 for a class 3–60-ft pole (Appendix D).

The height adjustment was originally derived as a conservative estimate of strength change with height when the actual dimensions are known. This adjustment varies linearly from 1.0 at groundline to 0.75 at midheight. The inverse varies from zero for a failure occurring at groundline to 1.33 if the failure occurs at midheight.

The class oversize adjustment is intended to recognize the fact that the majority of poles will have a groundline circumference larger than the tabulated minimum value for their size class. Any increase to groundline stress results in a proportional decrease in the required goundline section modulus. The ASC O5 committee voted to apply an increase to the projected groundline strength so that an average strength pole will be required to have the average, rather than the minimum, section modulus for its pole class to carry its class load. Determination of this increase involves an iterative process. The average groundline strength must first be determined on the basis of all pole tests. For each pole class–length combination, the standard-test moment divided by the average groundline strength gives the initial estimate of groundline section modulus required. Poles are initially classified by these minimum section modulii.

The class load is calculated as

$$P_i = 1200 + 250i + 50i^2 \qquad \text{(C1)}$$

where i is an index corresponding to the size class (0 for class 7, 12 for class H6), and P_i is the ANSI-designated load for class index.

The class oversize adjustment is defined as

$$A_{\cos} = (P_{i+1} + P_i)/(2P_i)$$

This simplifies to

$$A_{\cos} = 1 + 50(3 + i)/P_i \qquad \text{(C2)}$$

Application of this adjustment requires an iterative evaluation of minimum class size. Test poles are first classified using the estimated groundline strength (i.e., 8,000 lb/in^2 for southern pine and Douglas-fir) to match the pole to an ANSI load class (P_i) (Eq. (C1)) and required minimum groundline dimension. The pole's projected groundline strength is then adjusted by $A_{\cos}$ (Eq. (C2)). The average of adjusted values is used to recalculate minimum required section modulii for each pole class, and the test poles must be reclassified to recognize possible pole class change. This is repeated to convergence, resulting in a groundline strength that gives an average within-class section modulus the average capacity to carry the designated class load.

Table C1 shows the values derived at three stages in the derivation of the DFS for long poles. These values were derived using test data provided by EPRI-funded research on transmission-sized poles (Bodig and Goodman 1986, Bodig and others 1986b, Phillips and others 1985). MOR$_{\text{gl}}$ is an estimate of the average groundline strength as projected from the measured stress at failure. MOR$_{\text{gl}}$' is MOR$_{\text{gl}}$ adjusted for drying in service and conditioning. DFS is the designated fiber stress which, when multiplied by the minimum section modulus for a designated pole class-length, gives the average pole moment capacity for that class length.

Table C2 summarizes the resulting minimum groundline circumferences and oversize adjustment factors corresponding to the new DFS values. The oversize adjustment is ratio of average to minimum section modulus for each pole class. Test-pole projected groundline strengths were multiplied by these factors in the derivation of the average species DFS values. Values used for each pole are provided in column 8 of Tables C3, C4, and C5.

The "variability" adjustment originally incorporated in the first ASA O5 standard and continued in 1963 to counter effects of variability introduced by sampling, handling, and test methods was not used in this derivation. This was discussed in Rollins' report but not included in his final tally of strength values. If we assume a coefficient of variation of 14%, the adjustment of a half standard deviation would be 0.93 and the values reported in Table C1 would be changedto 5,630 lb/in^2 for western redcedar, 7,130 lb/in^2 for Douglas-fir, and 7,960 lb/in^2 for southern pine. Measured-COV values ranged from 15% to 19% for the test poles. This would give variability adjustments ranging from 0.90 to 0.92.

Table C1—Derivation of DFS for long poles

Species	MOR_{gl} (lb/in^2)	MOR_{gl}'[a] (lb/in^2)	DFS[b] (lb/in^2)
West redcedar	5,030	5,530	6,040
Douglas-fir	7,020	6,950	7,640
Southern pine	7,800	7,720	8,530

[a] Adjusted for class oversize.
[b] Adjusted for conditioning and drying-in-service.

Table C2—Minimum class groundline circumference and corresponding class oversize adjustment used to derive DFS for long poles

			Groundline circumference (in.) for different species and various pole lengths (ft)										
			Western redcedar				Douglas-fir			Southern pine			
Class	*i*	Class load (lb)	50	55	60	70	50	65	75	50	65	75	Oversize adjust
7	0	1,200	33	34	35	37	31	34	36	30	33	34	1.13
6	1	1,500	36	37	38	40	33	36	38	32	35	37	1.13
5	2	1,900	39	40	41	44	36	39	41	35	38	40	1.13
4	3	2,400	42	43	45	47	39	43	45	37	41	43	1.13
3	4	3,000	45	47	48	51	42	46	48	40	44	47	1.12
2	5	3,700	48	50	52	54	45	49	52	43	47	50	1.11
1	6	4,500	51	53	55	58	48	52	55	46	51	53	1.10
H1	7	5,400	55	57	58	62	51	56	59	49	54	57	1.09
H2	8	6,400	58	60	62	65	54	59	62	52	57	60	1.09
H3	9	7,500	61	63	65	69	57	62	65	55	60	63	1.08
H4	10	8,700	64	66	68	72	59	65	69	57	63	66	1.07
H5	11	10,000	67	70	72	76	62	68	72	60	66	70	1.07
H6	12	11,400	70	73	75	79	65	71	75	63	69	73	1.07

Table C3—Groundline data for southern pine

ID	Length (ft)	MOR$_{fl}$ (lb/in^2)	Failure location (in.)	Groundline stress at failure (lb/in^2)	MOR$_{gl}$ (lb/in^2)	Groundline circumference (in.)	Oversize adjust	AMOR$_{gl}$ (lb/in^2)
P010	50	9,371	12	9,476	9,481	46.0	1.100	10,429
P014	50	6,955	42	7,113	7,250	44.8	1.100	7,975
S004	50	9,632	60	9,673	10,227	43.1	1.108	11,332
S005	50	5,665	96	6,830	6,246	42.0	1.108	6,921
S006	50	6,871	168	8,603	8,207	41.5	1.117	9,165
S007	50	7,717	90	7,967	8,454	43.1	1.108	9,368
S008	50	7,748	51	8,734	8,151	43.2	1.108	9,032
S012	50	6,935	114	7,277	7,796	42.0	1.108	8,639
S034	50	6,448	0	6,448	6,448	45.1	1.100	7,093
S039	50	5,509	159	6,149	6,512	41.5	1.117	7,272
S040	50	6,699	168	7,114	8,002	43.6	1.108	8,867
S041	50	6,171	177	7,175	7,449	43.7	1.108	8,254
S042	50	7,190	15	7,195	7,296	44.5	1.100	8,026
S055	50	9,169	30	9,447	9,444	41.5	1.117	10,545
S062	50	4,506	225	5,621	5,762	44.0	1.108	6,385
S074	50	6,080	36	6,018	6,300	42.8	1.108	6,981
S081	50	8,475	0	8,475	8,475	43.6	1.108	9,391
S082	50	8,953	213	9,993	11,281	41.3	1.117	12,598
S098	50	5,196	180	5,729	6,294	42.4	1.108	6,974
S105	50	7,340	93	7,446	8,067	44.0	1.108	8,939
S108	50	7,386	48	7,669	7,746	44.6	1.100	8,521
S109	50	9,905	0	9,905	9,905	41.9	1.108	10,976
S111	50	5,564	96	5,506	6,135	43.4	1.108	6,798
S112	50	5,792	120	6,451	6,554	41.6	1.117	7,319
S113	50	5,915	54	6,059	6,242	42.1	1.108	6,916
S114	50	7,711	96	8,985	8,502	43.0	1.108	9,421
S116	50	9,047	9	9,024	9,127	43.0	1.108	10,113
S120	50	5,666	240	7,062	7,383	42.8	1.108	8,181
S001	65	9,539	84	10,005	10,169	49.1	1.100	11,186
S009	65	6,074	87	5,966	6,490	51.4	1.100	7,139
S013	65	6,735	264	7,071	8,363	48.7	1.108	9,267
S016	65	8,523	306	10,191	11,007	50.4	1.100	12,108
S022	65	7,662	12	7,735	7,730	48.0	1.108	8,566
S025	65	7,252	186	8,568	8,405	49.1	1.100	9,245
S031	65	6,001	384	6,667	8,001	51.2	1.100	8,801
S035	65	6,402	88	6,566	6,846	49.0	1.100	7,531
S037	65	5,316	440	7,233	7,088	50.3	1.100	7,797
S049	65	4,649	387	5,991	6,199	52.0	1.093	6,773
S053	65	6,733	380	8,619	8,977	50.3	1.100	9,875
S063	65	5,098	324	6,818	6,699	52.7	1.093	7,319
S064	65	4,095	258	5,058	5,057	50.6	1.100	5,563
S066	65	5,601	150	5,793	6,298	50.5	1.100	6,927
S073	65	7,105	6	7,066	7,137	53.5	1,093	7,797
S100	65	6,462	312	7,774	8,393	49.0	1.100	9,232
S101	65	6,306	297	6,884	8,075	50.8	1.100	8,882
S102	65	4,918	360	6,632	6,557	49.0	1.100	7,213
S107	65	7,980	0	7,980	7,980	51.4	1.100	8,778
S110	65	3,783	330	4,345	5,000	48.6	1.108	5,540

Table C3—Groundline data for southern pine (continued)

ID	Length (ft)	MOR$_{fl}$ (lb/in^2)	Failure location (in.)	Groundline stress at failure (lb/in^2)	MOR$_{gl}$ (lb/in^2)	Groundline circum-ference (in.)	Oversize adjust	AMOR$_{gl}$ (lb/in^2)
S115	65	7,031	264	8,404	8,731	51.0	1.100	9,604
S002	65	6,321	234	7,714	7,639	47.1	1.108	8,465
S003	65	6,522	210	7,876	7,717	49.0	1.100	8,489
S010	65	10,534	0	10,534	10,534	47.4	1.108	11,673
S011	65	5,784	276	6,899	7,262	47.6	1.108	8,047
S014	65	6,548	21	6,556	6,651	48.1	1.108	7,370
S015	65	3,380	396	4,233	4,507	46.3	1.108	4,994
S030	65	6,714	57	7,097	7,009	46.1	1.108	7,766
S032	65	5,926	306	6,578	7,653	49.2	1.100	8,418
S033	65	4,047	441	6,267	5,396	48.0	1.108	5,979
S046	65	8,925	39	9,321	9,189	47.6	1.108	10,183
S047	65	6,508	252	8,045	7,994	48.1	1.108	8,858
S054	65	8,467	21	8,783	8,600	46.8	1.108	9,530
S059	65	8,785	0	8,785	8,785	46.9	1.108	9,735
S069	65	4,492	369	5,981	5,989	48.8	1.108	6,637
S075	65	7,865	240	8,557	9,556	47.0	1.108	10,590
S078	65	7,366	0	7,366	7,366	48.6	1.108	8,162
S079	65	3,966	456	6,539	5,288	47.4	1.108	5,860
S080	65	7,280	180	7,854	8,394	46.6	1.108	9,302
S083	65	6,747	342	7,722	8,996	48.2	1.108	9,969
S088	65	4,612	360	5,730	6,149	46.4	1.108	6,814
S092	65	7,755	30	7,751	7,930	48.8	1.108	8,788
S094	65	5,337	375	6,683	7,116	46.0	1.108	7,885
S095	65	5,157	396	7,030	6,876	48.4	1.108	7,619
S106	65	8,013	54	8,410	8,345	45.4	1.117	9,319
S117	65	6,498	102	7,288	7,027	46.1	1.108	7,786
S118	65	7,955	198	8,675	9,315	45.8	1.108	10,322
S119	65	6,151	300	7,137	7,898	48.6	1.108	8,752
S018	65	6,483	264	8,108	8,050	49.2	1.100	8,855
S019	65	5,398	396	8,731	7,197	45.5	1.117	8,037
S021	65	8,391	154	9,499	9,466	43.9	1.117	10,570
S023	65	10,338	0	10,338	10,338	42.8	1.117	11,544
S027	65	8,371	78	8,412	8,882	43.9	1.117	9,918
S028	65	7,189	45	7,618	7,436	46.4	1.108	8,240
S036	65	4,960	210	3,588	5,869	48.5	1.108	6,503
S038	65	7,776	168	8,980	8,876	42.4	1.125	9,985
S045	65	5,777	189	7,062	6,713	43.2	1.117	7,496
S051	65	8,012	42	8,224	8,268	43.2	1.117	9,233
S052	65	7,353	228	8,280	8,839	43.3	1.117	9,870
S058	65	6,812	312	7,913	8,848	45.6	1.117	9,880
S060	65	4,319	309	4,965	5,594	44.9	1.117	6,246
S070	65	5,812	261	6,514	7,197	44.0	1.117	8,037
S071	65	7,427	210	7,709	8,788	43.3	1.117	9,813
S077	65	6,296	99	6,376	6,792	42.8	1.117	7,584
S085	65	8,208	39	8,173	8,451	46.0	1.108	9,365
S091	65	5,022	360	6,478	6,696	43.7	1.117	7,477
S097	65	7,894	72	7,555	8,337	45.1	1.117	9,309
S099	65	7,705	171	7,994	8,817	43.4	1.117	9,845

Table C3—Groundline data for southern pine (continued)

ID	Length (ft)	MOR_{fl} (lb/in^2)	Failure location (in.)	Groundline stress at failure (lb/in^2)	MOR_{gl} (lb/in^2)	Groundline circumference (in.)	Oversize adjust	$AMOR_{gl}$ (lb/in^2)
S103	65	5,207	312	7,097	6,763	42.8	1.117	7,552
S017	75	6,616	132	6,989	7,222	52.7	1.100	7,945
S020	75	4,947	444	6,523	6,596	50.8	1.108	7,309
S024	75	5,447	236	6,285	6,409	48.5	1.108	7,102
S026	75	7,166	258	8,712	8,573	47.0	1.117	9,573
S029	75	8,023	36	8,172	8,211	49.1	1.108	9,099
S043	75	6,238	504	8,698	8,317	49.3	1.108	9,217
S044	75	8,419	189	9,450	9,570	49.7	1.108	10,604
S048	75	7,989	204	9,410	9,180	48.8	1.108	10,173
S050	75	7,390	75	7,903	7,760	47.8	1.117	8,666
S056	75	5,475	405	7,231	7,300	49.3	1.108	8,089
S057	75	6,768	105	7,016	7,252	55.7	1.093	7,924
S072	75	9,302	0	9,302	9,302	49.4	1.108	10,308
S086	75	7,872	225	8,711	9,187	46.8	1.117	10,259
S087	75	9,740	42	10,139	10,007	50.9	1.108	11,089
S089	75	5,629	396	8,678	7,505	49.3	1.108	8,317
S093	75	11,204	0	11,204	11,204	49.9	1.108	12,415
S104	75	6,624	282	7,246	8,072	55.1	1.093	8,819
Average		6,826		7,579	7,798		1.10	8,618
Adjusted for drying in service (1.1) and conditioning (0.90)					0.99			
Adjusted value					7,720			8,532

Table C4—Groundline data for Douglas-fir

ID	Length (ft)	MOR_{fl} (lb/in^2)	Failure location (in.)	Groundline stress at failure (lb/in^2)	MOR_{gl} (lb/in^2)	Groundline circum-ference (in.)	Oversize adjust	$AMOR_{gl}$ (lb/in^2)
D010	50	7,561	12	7,715	7,650	46.9	1.10	8,415
D013	50	7,197	72	7,492	7,737	45.6	1.10	8,510
D031	50	7,338	24	7,482	7,513	47.3	1.10	8,264
D039	50	7,483	0	7,483	7,483	47.3	1.10	8,231
D057	50	6,656	0	6,656	6,656	45.1	1.10	7,322
D058	50	5,785	42	5,765	6,030	48.7	1.09	6,589
D091	50	7,094	96	7,156	7,822	46.2	1.10	8,604
D092	50	6,992	24	7,046	7,158	46.2	1.10	7,874
D093	50	5,363	90	6,315	5,875	45.7	1.10	6,463
D094	50	4,769	150	5,411	5,580	47.2	1.10	6,138
D095	50	6,874	33	7,016	7,101	45.4	1.10	7,811
D097	50	5,773	24	5,738	5,910	47.5	1.10	6,501
D100	50	6,917	18	7,067	7,040	47.2	1.10	7,744
D101	50	5,302	0	5,302	5,302	49.7	1.09	5,793
D108	50	5,866	75	6,640	6,326	45.0	1.11	7,010
D110	50	7,792	9	7,875	7,861	47.2	1.10	8,647
D118	50	5,461	48	5,512	5,727	46.6	1.10	6,300
D120	50	6,370	66	6,750	6,805	47.0	1.10	7,486
D128	50	5,741	36	5,828	5,949	45.8	1.10	6,543
D137	50	5,727	0	5,727	5,727	46.2	1.10	6,300
D148	50	7,613	54	7,783	8,033	45.4	1.10	8,837
D152	50	7,801	0	7,801	7,801	45.6	1.10	8,581
D156	50	8,594	24	8,846	8,799	46.3	1.10	9,678
D164	50	7,446	30	7,625	7,669	45.2	1.10	8,436
D168	50	8,768	0	8,768	8,768	44.2	1.11	9,716
D009	50	6,692	18	6,698	6,811	47.6	1.10	7,492
D012	50	5,326	6	5,380	5,357	50.2	1.09	5,853
D034	50	7,043	36	7,340	7,298	48.8	1.09	7,973
D038	50	5,829	151	5,853	6,828	48.5	1.09	7,460
D096	50	5,541	18	5,636	5,639	46.8	1.10	6,203
D099	50	9,137	48	9,423	9,583	48.2	1.09	10,470
D103	50	4,708	174	5,295	5,663	50.3	1.09	6,187
D105	50	6,175	138	6,667	7,128	48.5	1.09	7,788
D124	50	6,042	168	6,713	7,217	48.6	1.09	7,885
D125	50	7,076	0	7,076	7,076	49.3	1.09	7,731
D011	50	6,881	0	6,881	6,881	50.8	1.09	7,472
D037	50	6,540	69	7,351	7,009	52.1	1.09	7,611
D056	50	5,375	0	5,375	5,375	50.5	1.09	5,873
D142	50	5,769	54	6,091	6,088	52.2	1.09	6,611
D008	50	5,514	0	5,514	5,514	55.1	1.08	5,955
F043	50	5,792	85	6,630	6,312	53.0	1.09	6,854
F061	50	5,520	150	5,740	6,459	55.3	1.08	6,975
F044	51	6,498	84	7,288	7,061	54.2	1.08	7,626
F067	51	2,942	228	3,957	3,754	53.9	1.08	4,055
D001	65	8,768	0	8,768	8,768	49.8	1.10	9,645

Table C4—Groundline data for Douglas-fir (continued)

ID	Length (ft)	MOR$_{fl}$ (lb/in^2)	Failure location (in.)	Groundline stress at failure (lb/in^2)	MOR$_{gl}$ (lb/in^2)	Groundline circum-ference (in.)	Oversize adjust	AMOR$_{gl}$ (lb/in^2)
D004	65	5,559	261	6,051	6,884	51.7	1.10	7,572
D015	65	6,473	60	6,656	6,773	51.5	1.10	7,450
D027	65	6,216	48	5,979	6,444	50.6	1.10	7,089
D028	65	7,679	12	7,646	7,748	50.2	1.10	8,522
D030	65	5,420	276	6,229	6,805	49.4	1.11	7,541
D040	65	5,682	30	6,001	5,811	49.3	1.11	6,439
D045	65	4,822	360	5,493	6,429	53.2	1.09	7,025
D051	65	6,828	90	7,122	7,313	50.8	1.10	8,045
D054	65	7,320	300	7,863	9,400	49.9	1.10	10,340
D059	65	6,559	54	6,617	6,831	50.4	1.10	7,514
D072	65	6,322	294	6,694	8,072	54.0	1.09	8,820
D073	65	6,570	270	6,904	8,203	50.6	1.10	9,024
D088	65	5,740	255	6,093	7,069	50.8	1.10	7,776
D098	65	6,123	78	6,618	6,497	49.4	1.11	7,199
D130	65	7,793	240	7,598	9,469	48.2	1.11	10,493
D141	65	4,734	276	5,008	5,944	50.2	1.10	6,538
D144	65	6,003	90	6,405	6,430	49.8	1.10	7,073
D150	65	7,045	204	7,795	8,293	50.6	1.10	9,122
D153	65	7,984	30	8,084	8,165	50.5	1.10	8,981
D163	65	7,416	156	8,078	8,380	49.3	1.11	9,286
D021	65	5,750	84	6,316	6,130	46.6	1.11	6,792
D090	65	6,253	108	6,227	6,794	46.4	1.11	7,529
D127	65	6,943	0	6,943	6,943	46.0	1.12	7,753
D135	65	8,967	120	9,505	9,838	44.5	1.12	10,985
D146	65	6,094	54	5,977	6,347	46.4	1.11	7,033
D149	65	9,189	60	9,660	9,614	46.0	1.12	10,736
D155	65	7,350	252	9,114	9,028	45.1	1.12	10,081
D159	65	6,601	72	6,789	6,971	45.5	1.12	7,784
D166	65	7,199	192	8,508	8,386	44.9	1.12	9,365
D003	65	6,800	84	7,023	7,249	43.3	1.12	8,095
D005	65	7,858	180	7,825	9,061	46.2	1.12	10,118
D007	65	6,516	120	6,707	7,149	43.8	1.12	7,983
D019	65	5,747	78	5,859	6,098	45.5	1.12	6,809
D026	65	7,795	204	7,595	9,175	45.7	1.12	10,246
D033	65	5,964	138	6,373	6,640	44.4	1.12	7,414
D036	65	7,491	64	7,391	7,862	46.3	1.12	8,779
D042	65	8,354	4	8,359	8,379	45.7	1.12	9,356
D047	65	6,805	39	7,169	7,007	44.2	1.12	7,824
D055	65	7,424	30	7,476	7,592	44.6	1.12	8,478
D064	65	6,216	36	6,326	6,386	44.2	1.12	7,131
D066	65	6,223	102	6,639	6,729	46.4	1.11	7,457
D070	65	6,298	18	6,338	6,383	44.3	1.12	7,127
D075	65	5,789	0	5,789	5,789	45.0	1.12	6,464
D083	65	6,540	78	6,356	6,939	43.7	1.12	7,749
D085	65	6,847	162	7,170	7,776	43.4	1.12	8,683

Table C4—Groundline data for Douglas-fir (continued)

ID	Length (ft)	MOR$_{fl}$ (lb/in^2)	Failure location (in.)	Groundline stress at failure (lb/in^2)	MOR$_{gl}$ (lb/in^2)	Groundline circum-ference (in.)	Oversize adjust	AMOR$_{gl}$ (lb/in^2)
D089	65	6,556	12	6,583	6,615	43.8	1.12	7,386
D111	65	8,428	36	8,336	8,658	44.4	1.12	9,668
D117	65	8,092	0	8,092	8,092	44.4	1.12	9,036
D122	65	7,309	30	7,577	7,474	45.8	1.12	8,346
D129	65	7,203	6	7,270	7,235	43.2	1.13	8,139
D133	65	7,695	120	7,528	8,442	44.4	1.12	9,427
D134	65	6,654	162	7,274	7,557	43.2	1.13	8,501
D002	65	7,444	0	7,444	7,444	52.4	1.10	8,188
D063	65	6,018	168	6,272	6,869	52.3	1.10	7,556
D065	65	5,357	75	5,391	5,671	52.2	1.10	6,238
D071	65	6,415	240	6,319	7,795	53.9	1.09	8,516
D107	65	6,438	216	6,920	7,658	52.6	1.09	8,367
D114	65	5,883	54	6,065	6,127	52.1	1.10	6,740
D121	65	5,935	294	6,890	7,578	52.8	1.09	8,280
D126	65	6,943	120	6,886	7,617	53.0	1.09	8,322
D154	65	8,093	6	8,105	8,129	51.0	1.10	8,942
D165	65	6,147	216	7,261	7,312	52.3	1.10	8,043
D006	65	7,696	0	7,696	7,696	57.8	1.09	8,357
D014	65	4,574	108	4,419	4,970	54.5	1.09	5,430
D016	65	5,259	36	5,337	5,402	56.2	1.09	5,867
D017	65	7,213	54	7,428	7,512	56.9	1.09	8,158
D041	65	6,976	63	7,026	7,316	56.6	1.09	7,945
D043	65	7,082	18	7,260	7,177	55.2	1.09	7,842
D049	65	5,056	384	5,412	6,741	58.9	1.08	7,281
D050	65	8,011	31	8,050	8,198	55.3	1.09	8,958
D052	65	5,747	300	6,538	7,380	62.0	1.07	7,931
D061	65	5,348	0	5,348	5,348	61.6	1.08	5,776
D062	65	3,538	450	3,789	4,717	66.4	1.07	5,048
D067	65	4,081	0	4,081	4,081	59.5	1.08	4,407
D068	65	6,633	12	6,698	6,692	55.4	1.09	7,312
D074	65	6,800	0	6,800	6,800	55.4	1.09	7,430
D076	65	5,665	156	6,336	6,401	55.7	1.09	6,952
D077	65	5,507	96	5,865	5,927	54.0	1.09	6,475
D078	65	5,197	240	6,269	6,315	55.6	1.09	6,899
D081	65	5,716	204	6,644	6,728	55.0	1.09	7,351
D104	65	6,938	48	6,918	7,193	55.9	1.09	7,811
D109	65	5,182	216	5,800	6,164	55.7	1.09	6,694
D112	65	8,550	3	8,567	8,569	55.2	1.09	9,362
D116	65	6,837	6	6,799	6,867	56.2	1.09	7,458
D131	65	4,217	408	5,794	5,623	56.8	1.09	6,106
D138	65	7,092	0	7,092	7,092	56.3	1.09	7,701
D139	65	7,223	18	7,278	7,320	58.2	1.09	7,949
D140	65	5,394	240	5,787	6,554	57.4	1.09	7,117
D147	65	6,962	0	6,962	6,962	56.4	1.09	7,560
D157	65	6,129	216	6,918	7,290	56.8	1.09	7,917

Table C4—Groundline data for Douglas-fir (continued)

ID	Length (ft)	MOR_{fl} (lb/in^2)	Failure location (in.)	Groundline stress at failure (lb/in^2)	MOR_{gl} (lb/in^2)	Groundline circum-ference (in.)	Oversize adjust	AMOR_{gl} (lb/in^2)
D160	65	5,286	9	5,265	5,321	57.1	1.09	5,779
D167	65	6,545	24	6,543	6,663	57.0	1.09	7,236
D018	65	5,325	78	5,199	5,650	60.2	1.08	6,102
D044	65	7,160	12	7,252	7,224	59.3	1.08	7,802
D115	65	6,886	81	7,029	7,323	61.0	1.08	7,909
D020	75	5,708	258	6,218	6,829	54.4	1.10	7,512
D023	75	5,265	516	4,969	7,020	61.3	1.09	7,623
D024	75	5,871	42	5,783	6,032	52.6	1.10	6,635
D025	75	5,303	84	5,206	5,602	56.8	1.09	6,121
D029	75	5,977	132	6,298	6,525	51.8	1.11	7,230
D032	75	7,016	12	6,966	7,070	53.4	1.10	7,777
D035	75	5,392	192	5,977	6,142	52.9	1.10	6,756
D046	75	6,306	318	6,869	7,905	53.4	1.10	8,696
D048	75	7,756	57	7,506	8,048	52.8	1.10	8,853
D060	75	6,177	37	6,182	6,326	53.6	1.10	6,958
D069	75	4,721	408	5,294	6,295	53.2	1.10	6,924
D079	75	6,344	48	6,381	6,544	52.3	1.10	7,198
D080	75	4,804	192	4,820	5,472	52.7	1.10	6,020
D082	75	8,075	0	8,075	8,075	52.4	1.10	8,883
D084	75	4,127	336	4,284	5,249	56.5	1.09	5,735
D086	75	6,938	210	6,975	8,008	52.2	1.10	8,809
D087	75	6,068	165	6,633	6,780	52.6	1.10	7,458
D102	75	7,131	78	7,401	7,503	53.3	1.10	8,254
D106	75	7,472	234	8,433	8,779	53.5	1.10	9,657
D113	75	8,203	66	8,365	8,562	53.0	1.10	9,419
D119	75	7,905	108	8,549	8,488	52.8	1.10	9,337
D123	75	6,566	0	6,566	6,566	51.6	1.11	7,276
D132	75	6,025	66	6,594	6,289	54.1	1.10	6,918
D136	75	8,202	0	8,202	8,202	51.8	1.11	9,089
D143	75	7,549	90	7,640	8,007	52.3	1.10	8,808
D145	75	5,690	222	6,772	6,626	52.1	1.10	7,288
D151	75	6,784	180	6,889	7,661	52.3	1.10	8,427
D158	75	6,494	150	6,487	7,179	52.8	1.10	7,897
D161	75	6,137	192	6,053	6,991	52.3	1.10	7,690
D162	75	7,235	102	7,104	7,737	53.0	1.10	8,511
D022	75	6,000	216	6,252	6,956	56.9	1.09	7,600
D053	75	3,912	432	5,025	5,216	55.3	1.10	5,738
Excluding PSC pole tests		6,480		6,730	7,020		1.04	7,720
Adjusted for conditioning and drying in service					0.99			0.99
Adjusted value					6,950			7,643

Table C5. Groundline data for western redcedar

ID	Length (ft)	MOR_{fl} (lb/in^2)	Failure location (in.)	Groundline stress at failure (lb/in^2)	MOR_{gl} (lb/in^2)	Groundline circum-ference (in.)	Oversize adjust	$AMOR_{gl}$ (lb/in^2)
C91	55	3,410	294	2,980	4,547	54.1	1.09	4,968
C92	55	4,140	270	4,310	5,425	51.1	1.10	5,967
C93	55	5,060	72	4,960	5,401	53.4	1.10	5,941
C94	55	5,160	228	4,830	6,450	53.7	1.09	7,047
C95	55	2,830	174	2,640	3,340	54.2	1.09	3,649
W001	50	4,049	150	4,122	4,738	51.0	1.10	5,211
W002	50	3,295	180	3,081	3,991	52.1	1.09	4,361
W003	50	4,363	54	4,409	4,604	49.4	1.10	5,064
W005	50	4,256	174	4,321	5,119	50.2	1.10	5,631
W009	50	3,939	48	3,692	4,131	54.6	1.09	4,514
W035	50	4,253	150	4,365	4,976	55.7	1.09	5,404
W036	50	4,231	132	4,149	4,852	53.3	1.09	5,301
W037	50	4,229	276	3,850	5,639	58.9	1.08	6,090
W040	50	4,205	120	4,672	4,758	55.4	1.09	5,167
W046	50	3,935	138	3,213	4,542	59.5	1.08	4,906
W064	50	3,079	246	3,688	4,043	52.0	1.09	4,417
W065	50	3,985	42	4,009	4,154	52.2	1.09	4,539
W066	50	5,521	102	5,536	6,127	53.3	1.09	6,694
W067	50	3,038	78	2,780	3,286	56.5	1.09	3,569
W069	50	3,585	60	3,699	3,806	52.0	1.09	4,159
W082	50	4,673	202	4,909	5,807	50.9	1.10	6,388
W083	50	4,257	198	4,247	5,268	50.2	1.10	5,794
W084	50	5,445	0	5,445	5,445	50.5	1.10	5,990
W094	50	3,873	246	3,312	5,085	53.8	1.09	5,556
W096	50	6,187	178	6,185	7,473	50.2	1.10	8,220
W004	60	4,099	108	4,003	4,487	50.6	1.11	4,972
W008	60	6,126	148	6,076	6,948	49.6	1.11	7,699
W011	60	5,205	210	4,910	6,258	51.6	1.11	6,935
W013	60	4,793	204	5,122	5,730	51.4	1.11	6,349
W015	60	4,627	192	4,517	5,468	59.4	1.09	5,938
W016	60	3,576	192	3,372	4,226	58.6	1.09	4,589
W017	60	4,246	228	4,308	5,195	50.4	1.11	5,757
W018	60	4,867	160	4,771	5,581	55.3	1.09	6,097
W019	60	2,538	166	2,179	2,926	69.6	1.07	3,131
W020	60	4,403	240	3,819	5,451	59.0	1.09	5,920
W021	60	4,613	96	4,384	4,997	60.7	1.09	5,427
W022	60	4,384	192	3,876	5,181	61.2	1.09	5,626
W023	60	4,401	198	3,901	5,231	55.8	1.09	5,715
W024	60	3,719	174	3,363	4,322	67.9	1.07	4,624
W025	60	2,189	180	2,004	2,558	66.7	1.07	2,749
W026	60	5,210	196	4,889	6,178	52.4	1.10	6,796
W027	60	4,718	168	4,106	5,452	58.9	1.09	5,920
W028	60	5,216	198	5,001	6,200	58.9	1.09	6,732
W031	60	3,689	270	3,256	4,707	62.5	1.08	5,084
W034	60	4,942	108	4,784	5,410	62.6	1.08	5,843

Table C5. Groundline data for western redcedar (continued)

ID	Length (ft)	MOR$_{fl}$ (lb/in^2)	Failure location (in.)	Groundline stress at failure (lb/in^2)	MOR$_{gl}$ (lb/in^2)	Groundline circumference (in.)	Oversize adjust	AMOR$_{gl}$ (lb/in^2)
W038	60	3,845	210	3,858	4,623	62.2	1.08	4,993
W039	60	3,833	150	3,705	4,357	54.1	1.10	4,792
W041	60	4,542	300	3,384	5,979	60.0	1.09	6,493
W042	60	4,360	180	4,207	5,095	64.7	1.08	5,502
W043	60	4,433	24	4,244	4,520	63.6	1.08	4,882
W044	60	4,746	180	4,402	5,546	48.8	1.11	6,145
W045	60	4,617	264	4,413	5,856	51.4	1.11	6,489
W047	60	3,987	162	3,640	4,582	50.0	1.11	5,077
W048	60	5,089	210	4,965	6,119	63.2	1.08	6,608
W049	60	4,841	330	4,337	6,455	63.5	1.08	6,971
W052	60	4,170	150	4,261	4,740	49.6	1.11	5,252
W053	60	3,336	198	3,192	3,965	56.9	1.09	4,332
W054	60	4,298	102	4,344	4,681	47.3	1.12	5,227
W055	60	3,217	0	3,217	3,217	65.8	1.07	3,457
W056	60	3,395	192	3,260	4,012	63.4	1.08	4,333
W057	60	2,455	156	2,563	2,806	60.6	1.09	3,047
W058	60	4,525	264	4,947	5,739	55.0	1.10	6,313
W063	60	3,778	0	3,778	3,778	60.1	1.09	4,103
W068	60	3,023	210	2,844	3,635	58.9	1.09	3,947
W070	60	3,742	34	3,759	3,846	61.3	1.09	4,176
W071	60	4,000	108	4,190	4,379	48.5	1.12	4,890
W072	60	5,075	180	5,225	5,930	48.4	1.12	6,622
W073	60	4,209	66	4,044	4,444	48.5	1.12	4,962
W074	60	2,020	270	2,012	2,578	66.2	1.07	2,770
W075	60	4,366	192	3,822	5,160	58.2	1.09	5,638
W076	60	4,189	372	5,358	5,585	62.8	1.08	6,032
W077	60	4,849	186	4,455	5,698	59.4	1.09	6,188
W078	60	4,333	226	4,779	5,289	55.2	1.10	5,818
W079	60	5,343	184	4,907	6,265	63.4	1.08	6,766
W080	60	5,119	12	5,165	5,169	49.1	1.11	5,727
W081	60	4,061	330	4,003	5,415	61.8	1.08	5,848
W087	60	4,040	228	4,067	4,943	62.4	1.08	5,339
W088	60	4,858	198	5,163	5,774	62.2	1.08	6,236
W089	60	4,778	142	4,459	5,390	63.4	1.08	5,821
W092	60	4,628	228	4,677	5,662	50.4	1.11	6,275
W093	60	4,393	6	4,413	4,414	50.0	1.11	4,891
W097	60	4,620	46	4,228	4,795	55.8	1.09	5,239
W098	60	5,219	214	5,205	6,297	50.8	1.11	6,977
W099	60	5,202	208	4,986	6,240	48.2	1.12	6,968
W100	60	5,122	0	5,122	5,122	60.4	1.09	5,562
W006	70	5,196	0	5,196	5,196	59.2	1.09	5,677
W007	70	4,191	282	3,569	5,191	64.8	1.09	5,637
W010	70	3,898	300	3,310	4,903	64.7	1.09	5,324
W012	70	4,213	264	3,992	5,140	63.5	1.09	5,582
W014	70	4,171	192	3,919	4,801	69.0	1.07	5,159

Table C5. Groundline data for western redcedar (continued)

ID	Length (ft)	MOR_{fl} (lb/in^2)	Failure location (in.)	Groundline stress at failure (lb/in^2)	MOR_{gl} (lb/in^2)	Groundline circum- ference (in.)	Oversize adjust	$AMOR_{gl}$ (lb/in^2)
W029	70	3,740	72	3,517	3,933	70.7	1.07	4,227
W030	70	3,702	216	4,335	4,343	60.0	1.09	4,745
W032	70	5,819	264	5,692	7,099	61.2	1.09	7,757
W033	70	4,991	372	4,420	6,655	61.2	1.09	7,271
W050	70	4,407	174	4,445	5,001	70.7	1.07	5,375
W051	70	4,667	90	4,362	4,973	59.2	1.09	5,433
W059	70	4,685	162	4,090	5,268	60.0	1.09	5,756
W060	70	3,983	150	4,074	4,438	59.3	1.09	4,849
W061	70	4,387	228	4,640	5,196	56.9	1.10	5,716
W062	70	3,880	234	3,819	4,618	57.4	1.10	5,080
W085	70	4,524	88	4,506	4,812	56.5	1.10	5,293
W086	70	4,206	358	4,623	5,565	56.3	1.10	6,122
W090	70	5,040	180	4,800	5,747	56.2	1.10	6,321
W091	70	5,117	246	5,405	6,150	55.7	1.10	6,766
W095	70	4,030	300	4,345	5,069	56.8	1.10	5,576
Average		4,310		4,200	5,026		1.19	5,494
COV					19%			19%
Adjusted for conditioning and drying in service					1.1			1.1
Adjusted initial value					5,528	1.22		6,043

Appendix D—Class Oversize Adjustment

The use of discrete class sizes to classify utility poles introduces inaccuracy in the estimate of groundline strength. The ANSI O5.1 standard was developed as a specification for poles on the premise that the tabulated minimum circumferences for each size class provide the section modulus required to carry the designated class load and poles falling between consecutive class minimums are oversize for their respective class load. Annex C values were derived to correct for this by evaluating pole strengths as a weighted average capacity by first assigning an ANSI size class to each test pole, then dividing the failure moments at groundline by the minimum section modulus for the respective pole class rather than using the measured section property: the larger the pole within a given class size, the larger the groundline modulus of rupture (MOR). When applied to the minimum class dimension, these values give the average pole capacity.

In 2002, it was proposed to change the way values are derived in the specification. For poles in lengths of 50 ft and longer, test poles were first classified. The groundline strength of each test pole was then increased by the mean-to-minimum section modulus ratio (A) for its ANSI class.

The required section modulus is directly proportional to the class load, and the class load is defined by the function

$$P_i = 1200 + 250i + 50i^2 \qquad (1)$$

The mean-to-minimum-section modulus ratio (A) is equivalent to the ratio of loads as given by Equation (2). These “oversize” adjustment values are given in Table D1.

$$A_i = \frac{P_{i+0.5}}{P_i} \qquad (2)$$

The average of the size-adjusted strength values was then used to recalculate the minimum section modulus required for each class. In some cases, this required reclassification as some poles moved to the next larger class. The process was repeated until no further change was required. The use of this process, along with a 10% increase for drying in service, resulted in average adjusted fiber strength values for poles 50 ft and longer that were close to those used for poles less than 50 ft in length. As a result, the tabulated minimum circumference values remained unchanged, but they must now be recognized as providing reduced capability to carry the designated class load when compared to the minimum circumference values given for shorter poles.

ANSI tables represent a catalog of pole sizes. Class loads merely provide a basis for the size classification. The fact that values are derived differently for pole lengths either side of 50 ft is something to be documented but has little impact on design as long as users realize that class load is not synonymous with capacity.

Table D1—DFS adjustments for ANSI structural pole classes

	DFS adjustments for ANSI structural pole classes												
	0	1	2	3	4	5	6	7	8	9	10	11	12
Factor	7	6	5	4	3	2	1	H1	H2	H3	H4	H5	H6
A_i	1.12	1.13	1.13	1.12	1.12	1.11	1.10	1.09	1.09	1.08	1.07	1.07	1.07
B_i	0.96	0.96	0.96	0.96	0.96	0.97	0.97	0.97	0.97	0.97	0.98	0.98	0.98