Outside Plant Consulting Services, Inc.

Pultruded Fiber-Reinforced Composite Distribution Poles

Review of Cantilever Test Result

Rev. 1

Prepared by:

Outside Plant Consulting Services, Inc.
January 15, 2003

for Powertrusion International, Inc.

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Reason for Revision

This report represents a revision of the original April 8, 2002 document. The changes include consideration of additional data points in the quantitative analysis, and modification of the statistical treatment consistent with the independent nature of the five basic data sets representing different vintages of the evolving product. In addition, several of the references have been updated.

Due to numerous associated minor editing changes, and to maintain readability, specific changes in this issue are not highlighted.

This report is intended to help meet the needs of the National Electrical Safety Code (NESC) for the introduction of pultruded fiber-reinforced composite materials (poles, …) into the 2007 Edition of the NESC. This report is to be submitted to NESC Task Force 5.1.7.

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Executive Summary

During the past four years, several series of full-scale mechanical strength tests have been performed on Pultruded Fiber-Reinforced Composite (PFRC) poles manufactured by Powertrusion International, Inc. The objective was to determine the lateral bending characteristics relative to 40 ft. Class 4 wood distribution poles. Five sets of test data, representing several vintages of the evolving product, were analyzed with respect to the lateral bending strengths, as well as flexural stiffness and column buckling loads, of the PFRC poles, and compared to wood pole alternatives. The 40 ft. PFRC poles were characterized statistically to determine the mean and 5% lower exclusion limit (5%LEL) values, consistent with evolving reliability based design procedures and potential implementation in the National Electrical Safety Code (NESC).

The analyses show

- The historical mean strength of the 40 ft. PFRC poles exceeds that of Class 4 wood poles (2400 lbs).
- The historical 5%LEL strength of the PFRC poles significantly exceeds tha of Class 4 wood poles (approximately 1600 lbs.). The 5%LEL value represents an appropriate design strength for highly reliable construction (e.g., NESC Grade B).
- The 5%LEL strength of the PFRC poles also significantly exceeds the higher allowed strength of Class 4 wood poles (2040 lbs.) ap propriate for the widely used NESC Grade C construction.
- The determined lateral bending strengths of the constant cross-section PFRC poles may be readily extrapolated to PFRC poles of different lengths, in inverse proportion of the moment arm to the ground-line or its proximity.
- The mean stiffness of the PFRC poles is approximately half that of 40 ft. Class 4 wood poles; the 5%LEL design values, however, are considerably closer in magnitude, with a relatively small difference from some wood species.
- The calculated mean column buckling resistance of the PFRC poles (32,400 lbs.) is somewhat lower that of Class 4 wood poles; however, the 5%LEL design value (30,000 lbs.) of the PFRC poles is greater than that of two of the three major wood species considered.

The results indicate the capability of readily producing high quality PFRC poles of low variability and with sufficient strength to meet requirements satisfied by conventional wood poles. These results are considered to represent conservative estimates of the strength of the PFRC poles. The adoption of the ASTM D 4923 test procedure and simulated ground-line support details should eliminate the need to adjust (reduce) the strength to lower than that directly measured. In general, the uniform cross-section and material properties along the axial direction of the PFRC product facilitates its evaluation and characterization, and the implementation of reliability based design procedures.

Table of Contents

Background & Introduction

The most significant characteristic of utility distribution poles for power and telecommunications applications is the lateral (cantilever) bending strength. This property is typically the critical factor in the design of a distribution or transmission line, due to the transverse wind forces applied to the supported conductors (possibly ice-covered) and to the structure itself. Thus, wood poles are classified by the American National Standards Institute (ANSI) standard ANSI-05.1[1], *American National Standard for Wood Poles - Specifications and Dimensions*, according to their ability to withstand such lateral loads. ANSI-05.1 also recognizes the relatively wide variability of such naturally grown products. The variability of the wood poles is also reflected in the National Electrical Safety Code[2] (NESC), by the specification of "strength factors" to be applied to the strength levels indicated in the wood standard. For the most commonly used wood species (Southern Yellow Pine, SYP; Douglas Fir, DF; Western Red Cedar, WRC), the latter values are acknowledged to correspond to the mean strength[1]. ASTM D 1036[3] is the accepted standard for testing tapered, solid wood poles subject to lateral load, and has been used to evaluate pole strength, and stiffness, for the various species and sizes.

Wood poles are classified by "pole class" (Class 1, Class 2, …, Class 10) which defines the strength of the pole based upon a lateral load placed 2 feet from the tip. A lower class number pole is stronger than a higher class number pole. The dimensions (diameter or circumference) of a pole of a given class will be a function of its length and species. The intention is that all pole members of a given pole class have the same strength when subject to the same (specified) lateral load, placed 2 feet from the tip, regardless of the height, dimensions, or wood species of the pole. The dimensions (diameter) of the wood pole are modified accordingly to account for different length poles and material properties. Since the utilities have experience and familiarity with the wood pole classification system, it would be convenient for Fiber-Reinforced Composite (FRC) and other non-wood poles to be classified similarly, to facilitate their selection and use by the utility. However, due to the inherently different characteristics between naturally grown wood and engineered pole structures (FRC, steel, …), the equivalency of a given FRC pole size/strength, expressed as a "pole class" number, for one application may not be the same for another application. Thus, care must be used in attempting to adopt this approach, as discussed in industry literature[4,5].

FRC poles are engineered products with relatively consistent dimensions and properties, in comparison to naturally grown wood products with inherent variability and potential vulnerability to degradation and deterioration. Pultruded FRC (PFRC) products, as manufactured by Powertrusion International, Inc., are long, hollow (tubular) cylindrical objects, of external octagonal configuration, as indicated in Figure 1. The poles are uniform (straight, non-tapered) in the longitudinal (axial) direction, and are designed to be directly embedded in the soil, at depths similar to that of conventional wood poles. Typical burial depths are equal to 10% pole length plus 2 feet; for poles shorter than 40 feet, the recommendation is generally 10% pole length, plus 2.5 feet. The most

commonly used wood pole size for distribution applications has been a 40 ft. long pole, of Class 4 strength (2400 lbs.).

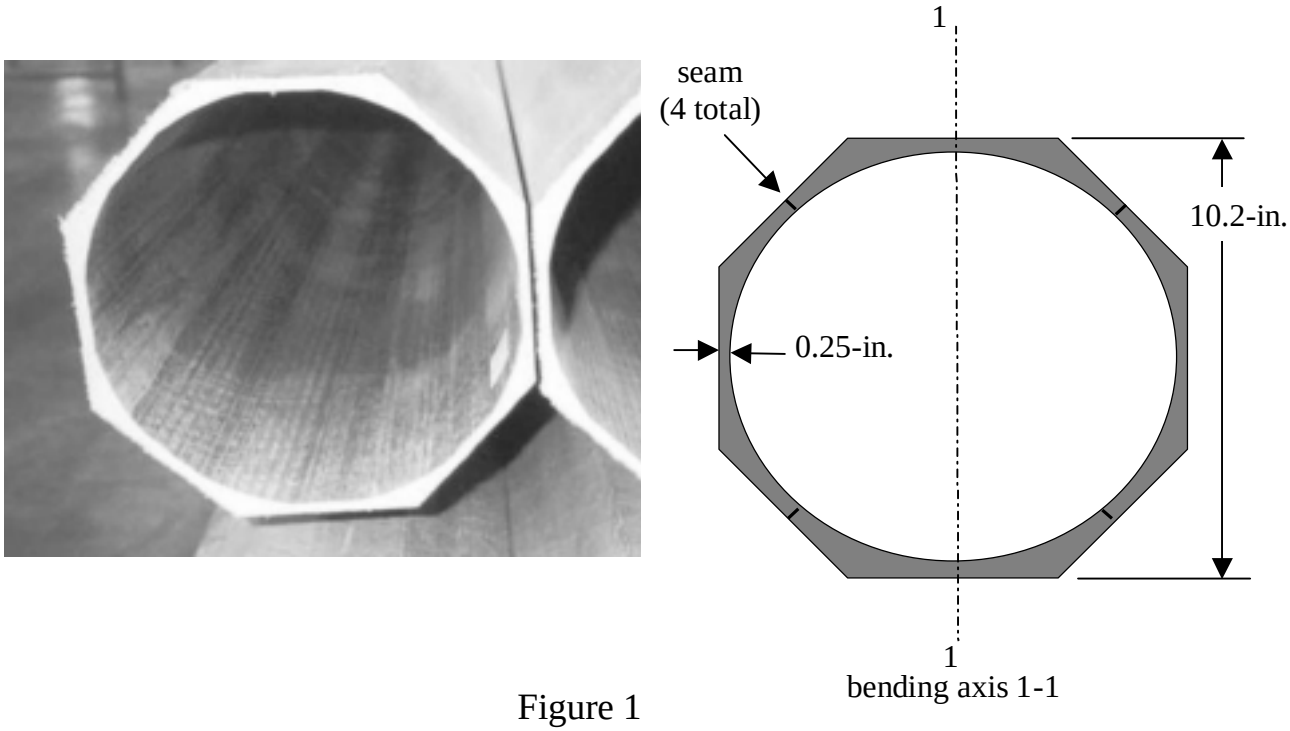

Figure 1
Powertrusion PFRC Pole Cross-Section

The NESC provides strength and loading rules as a function of "grade of construction" (i.e., reliability level), which will determine the appropriate size pole to withstand wind and ice storms, to meet basic safety requirements. The NESC defines three grades of construction relevant to pole lines:

- Grade B -- the highest grade; typically corresponds to crossings (highway, railroad, pole lines carrying varying power supply voltage levels, …)
- Grade C -- lower grade of construction than Grade B; typical power or joint-use (telecommunications and power) distribution pole applications
- Grade N -- lowest grade of construction; typical sole use telephony applications.[1]

Different grades of construction will require a different strength and therefore a differen class pole. As indicated above, due to the inherently different characteristics (e.g.,

[1] The former "Bell System Practices"[6] define a range of construction grades dependent upon the type and quantity of telecommunications circuits supported, containing the equivalent of Grade B, Grade C, and lower. It is assumed that telecommunications companies would desire Grade C equivalent poles for most applications.

statistical distributions, …) of naturally grown wood and engineered pole structures, the equivalency of a given PFRC pole size/strength (pole class number) for a Grade C construction application will in general not be the same for a Grade B application. Based upon its wide usage in typical distribution applications, Powertrusion has performed most of its testing on the 40 ft. long Class 4 equivalent pole size for Grade C construction.

In addition to the transverse load capacity of the pole, the "stiffness" or flexibility is of interest. This property relates to the amount of pole deflection when subject to a lateral force, and is also directly related to the column buckling load, with stiffer poles displaying greater buckling resistance. Greater buckling resistance also minimizes "P-Δ" effects that tend to amplify the effect of vertical loads on pole bending stresses.

The present report summarizes various documented or recorded full-scale pole tests performed on the "standard" Powertrusion 10.2-in. width (0.25-in. minimum wall thickness) PFRC product during the past four years. These tests were performed by various test laboratories, on behalf of Powertrusion, or by Powertrusion itself, at its Las Vegas facility. A review and analysis of this test data provides important information concerning the potential classification and application of the pultruded FRC product, and recommendations and guidelines for additional test data.

Description of Cantilever Pole Tests

Full-scale cantilever tests on utility poles typically follow the overall format and procedures of ASTM D 1036, originally intended for naturally-grown, solid, tapered wood poles. There are similar test procedures specifically intended for tubular FRC poles. ASTM D 4923[7] and ANSI C136.20[8] represent such standard test procedures for lighting pole applications, but which are also appropriate for the present utility applications. A significant difference between the ASTM 1036, ASTM 4923, and ANSI C136.20 procedures for cantilever strength testing relates to the support details for the pole at the simulated ground-line (GL) and below. Most of the present tests incorporate still another type of support system, with results and effects as described below.

The details of the support of the pole are important in determining the lateral load capability of a tubular pole that fails in a local collapse mode. The support can increase (e.g., if fully constrains the pole, resulting in the pole breaking several feet above the GL) or decrease (if the GL support acts as a local stress concentration point) the effective strength of the pole. The objective is therefore to simulate the actual support condition provided by the soil. The soil would be expected to provide continuous, firm resistance at the leading edge of the pole, as the soil becomes compacted under the pole deflection towards the load, but provide relatively little constraint at the rear or sides. The support saddles and slings deployed in ASTM 4923 and ANSI C136.20 attempt to provide such conditions. Conversely, the relatively rigid, uncushioned support of the wooden saddles of ASTM 1036, without a full constraint of the cross-section, may cause premature failure due to a concentrated load point at the leading edge[9]. The full-scale pole tests of tubular poles, including the Powertrusion product, have demonstrated the range of effects due to various types of support conditions. Figure 2 shows a fully constraining support

clamp that tends to increase the load capacity of the pole, as implemented in the majority of tests in Table 1, and which therefore requires adjustment (reduction) of strength, as described below.

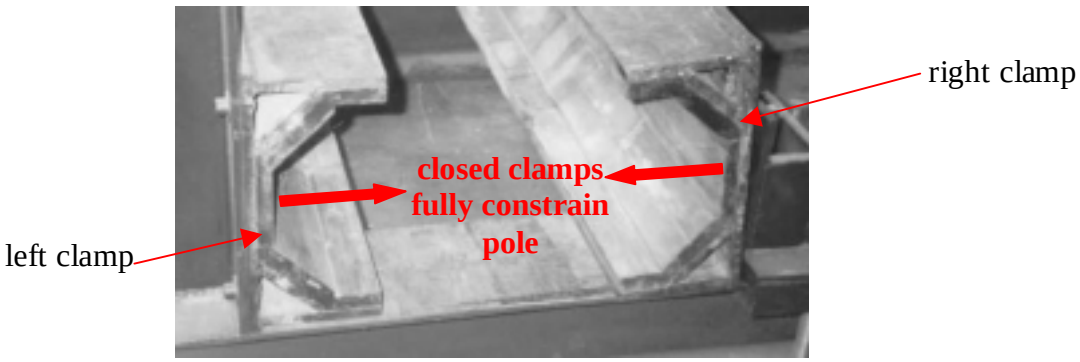

Figure 2
Fully Constraining Support System for Base of Octagonal PFRC Pole
(Clamps Shown Open)

Thus, the present report describes the results of cantilever tests of poles of primarily 40 ft. length (6 ft. below GL), in a test fixture(s) supporting the pole at the simulated GL and below, when subject to a lateral load applied 2 ft. from the tip. The poles are of the octagonal 10.2 -in. wide, 0.25-in. minimum wall thickness product of Figure 1. The ultimate strength, as well as lateral deflection of the pole at the point of load, was recorded. The latter represents a measure of the pole stiffness. Since the tests span a period of several years, at various test facilities, during a period of continuing product evolution, significant variability would be expected due to possible changes in materials and formulations, as well as the aforementioned effects associated with the specific tes procedures implemented. The primary sets of test data of interest, as identified by the test facility, are:

- Test Facility #1 (TF-1), October 1998
- Test Facility #2 (TF-2), April 2001
- Powertrusion (PT-A), July 2001 – October 2001
- Powertrusion (PT-B), January 2002
- Powertrusion (PT-C), February 2002

as summarized in Table 1. The most recent set of tests (PT-C) reflected in this report were conducted to help evaluate and standardize on a pole support system compatible

with ASTM 4923, and are discussed in further detail below. It is the intention to adopt the latter technique as representing an existing, industr -approved procedure, to continue providing credible test data based upon full -scale pole tests, consistent with the methodology being established by the Struct ral Reliability Based Design (SRBD Committee of the American Society of Civil Engineers[10]. Figure 3 illustrates the overall test configuration, as implemented at the various facilities, with the most significan differences related to the details of the supports at the GL and below.

Table 1
Full-Scale Cantilever Pole Tests
(10.2-in. x 0.25-in., Pultruded, Octagonal)

Test Series	Date	Number Poles Tested	Pole Lengths	Remarks
TF-1	10/02/1998	8	40 ft.	Fully constrained at GL and belo
TF-2	04/11/2001	5	40 ft.	Sling (ANSI C136.20) at GL and butt; one axial (buckling) test performed*
PT-A	07/20/2001 – 10/17/2001	7	40 – 45 ft.	Fully constrained at GL and below; fiv 45 ft. poles and two 40 ft. poles
PT-B	01/14/2002 – 01/16/2002	3	40 ft.	Fully constrained at GL and below; one test with seams at outer/inner/lateral faces
PT-C	02/20/2002 – 02/21/2002	2	40 ft.	Fully constrained at GL and below, vs. single-sided, cushioned support (ASTM 4923)

* The data point corresponding to the pole previously subjected to axial compression is not included in Figure 6 or the statistical summary of Table 2.

As indicated in Figure 3, a 40 ft. pole corresponds to a moment arm of 32 ft. from the load to the ground-line support location. For the several tests performed on 45 ft. poles, a "burial" depth of 6.5 ft. and moment arm to GL of 36.5 ft. (= 45 ft. – 6.5 ft. – 2 ft.) apply The slight angle of the lateral cantilever load to the original pole position is intended to maintain the moment arm of the applied load at the dis ance indicated (i.e., 32 ft. for a 40 ft. pole) when the pole is approximately at its ultimate load. In this case, the total moment load will be essentially equal to the force at failure multiplied by the original 32 ft. moment arm. (ASTM D 1036 assumes the applied load will remain perpendicular to the original pole position, requiring an adjustment to account for the slightly reduced moment arm at failure, due to the inward movement of the end of the pole.)

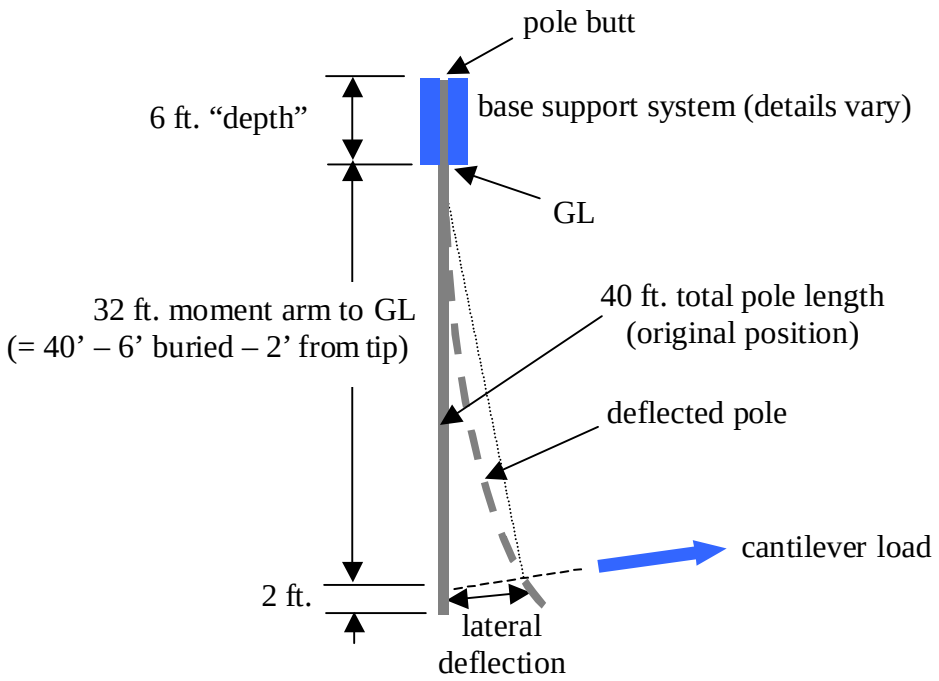

Figure 3
Typical Pole Cantilever Bending Test Configuration and Terminology
(40 ft. Total Pole Length)

The poles were typically tested in an orientation such that the longitudinal seams (see Figure 1) were deliberately located away from the outer, inner, and lateral bending faces of the octagonal surfaces of the pole. (Exceptions are noted in Table 1.) The product is intended to be deployed such that the potentially more vulnerable seams are not coincident with the more highly stressed or strained surfaces, corresponding to those induced by the transverse bending loads due to wind, and/or wire tension at an unguyed corner. These lateral loads will cause the pole to bend about an axis 1-1 illustrated in Figure 1, leading to axial compressive stresses at the inside of the bend and axial tensile stresses at the outside of the bend, as well as ovaling stresses at the outer, inner, and lateral faces of the pole cross-section, as the pole bends and flexes.

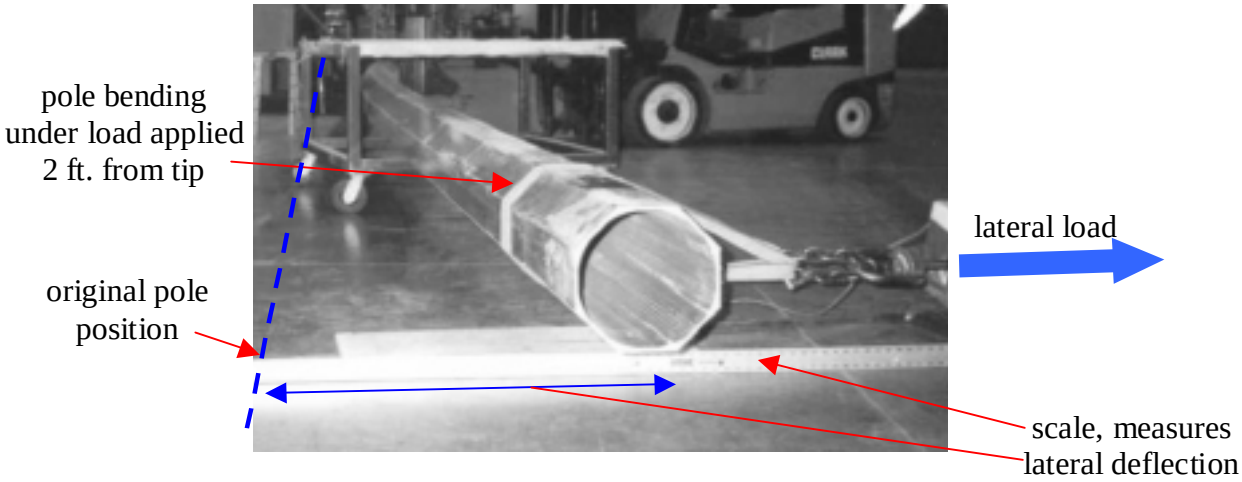

Figure 4
PFRC Pole Cantilever Bending Test

Figure 4 shows a pole deflecting laterally due to the cantilever load applied during testing. The poles fail in a local collapse mode within several feet or inches (depending upon the simulated base support condition) of the GL, subsequent to an observable ovaling of the cross-section, resulting in a severely damaged length of almost 2 ft. extent, as illustrated in Figure 5.

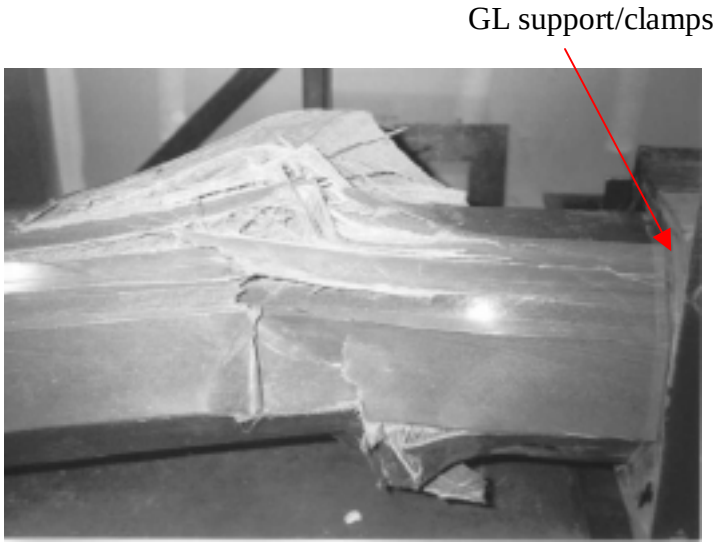

Figure 5
Ruptured/Collapsed Pole near GL Support

Analysis & Discussion

The results of the various tests described above are summarized in Figures 6 – 10 and Tables 2 and 3.

Cantilever Strength

Figure 6 and Table 2 show the results on an individual pole and on a collective basis, for the five sets of data listed in Table 1. Both the actual failure loads, as directly measured, as well as "adjusted" strengths are shown. The latter values correspond to somewhat reduced capacities, assuming the supports at the base have provided optimistically high strengths by preventing the pole from collapsing at the GL, the point of maximum bending moment. For the axially uniform PFRC poles, a characteristic, "unconstrained" collapse/breaking moment may be assumed, equal to the force at rupture multiplied by the moment arm to the collapse location (typically several feet above the GL). The adjusted (reduced) bending strengths are then obtained by dividing the characteristic moment by the moment arm to the GL (i.e., 32 ft.).[2] These adjusted values are believed to be conservative since some degree of constraint would be anticipated at the GL support, albeit relatively small -- possibly as reflected in the Test Facility #2 data using slings in which the collapse location was still somewhat above the GL. (In the latter test series, the poles failed at an average distance of 1 ft. from the GL, in comparison to the 2 - 3.5 ft. distances for the three sets of data using fully constraining supports: TF-1, PT-A, PT-B). Realistic, in-the-ground full-scale pole tests would verify the degree of actual support to be anticipated.

Table 2 lists the calculated strength characteristics illustrated in Figure 6, including the associated coefficients of variation (COV), defined as the standard deviation divided by the mean. The corresponding 5% "lower exclusion limit" (5%LEL) -- the strength exceeded by 95% (= 100% - 5%) of the poles -- has also been calculated for each set of data, assuming a normal distribution. The ASCE *Recommended Practice for Fiber - Reinforced Composite Products for Overhead Utility Line Structures*[10] indicates that the 5%LEL values should be used for all mechanical properties. The degree of variability within each test set is very low -- typically only a few percent, with several as little as 2 or less. Values for the individual test sets are shown in Table 2. The average COV across all data sets, is approximately 3½ percent. For a normal distribution, a point estimate of the 5%LEL for each data set may be directly obtained from the mean and COV, as follows:

$$5\%\text{LEL (lbs.)} = \text{mean strength (lbs.)} \times (1 - 1.645 \times \text{COV})$$

In general, the 5%LEL values in Table 2 and Figure 6 are based upon this equation. (Exceptions correspond to 5%LEL values based upon a "50% confidence level".)

For the present purposes, the data for the five sets of tests have not been directly combined into a single group for statistical analysis and determination of product variability (COV, …). The corresponding poles in each set represent identifiably

[2] The consideration of a characteristic bending moment strength ignores the possible effect of lateral shear forces at the GL. Such effects are generally not significant, except for very short poles.

different products based upon an evolving PFRC pole design and varying materia formulations. In addition, different test facilitie s and procedures have been implemented. These factors would result in a relatively wide variability in calculated strengths and pole characteristics, which would not be indicative of the actual manufacturing process. Therefore, the results of the five test sets are summarized via averages across the individual tests, with each test set weighted equally, in order to determine overall trends and product capability. This procedure was applied to both the pole bending strength and stiffness.

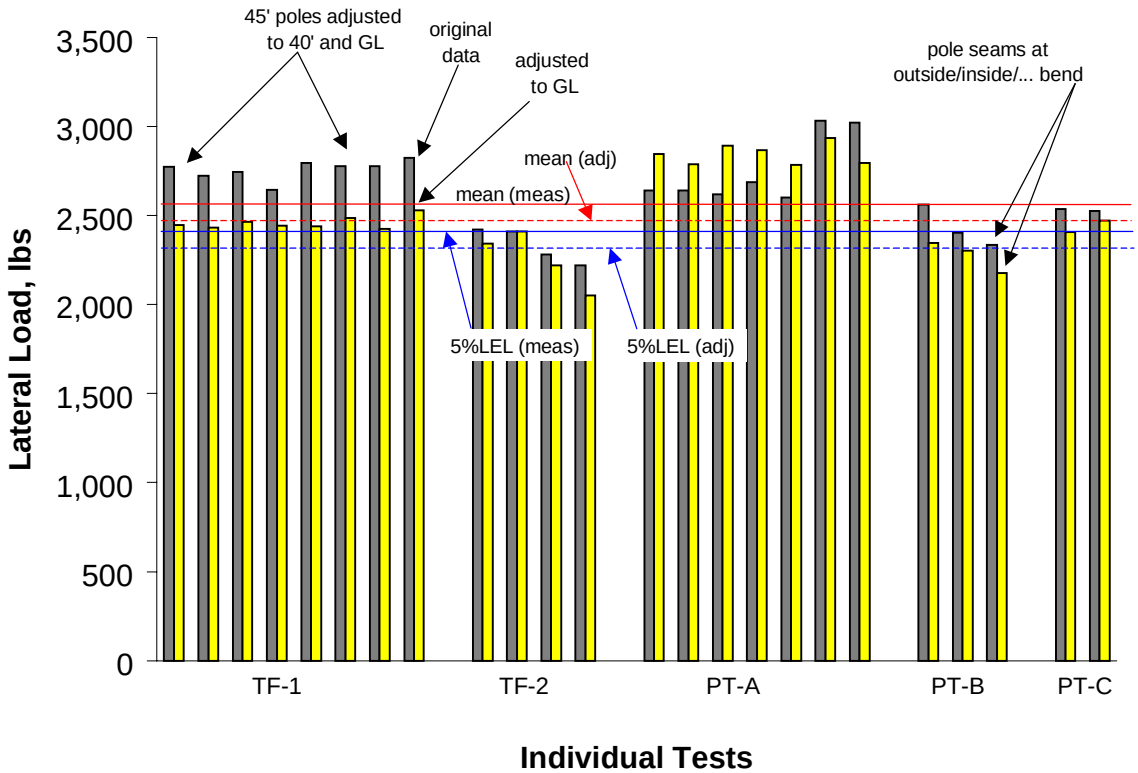

Figure 6
Strength Results of Individual Cantilever Pole Tests

Considering the 2400 lb. strength of a Class 4 wood pole, and the NESC strength factor of 0.85 for the common Grade C construction[2], an equivalent PFRC pole should provide a "minimum" strength of 2040 lbs (= 0.85 x 2400). This may be conservatively provided by a PFRC pole with a 5%LEL adjusted strength at this level. Thus, the average of the historical data of Table 2 meets this criteria (2308 lbs., adjusted). The only set of data that would not meet this condition is that of the TF -2 tests. This appears to be due to a somewhat weaker vintage of poles, combined with a relatively large variability in the adjusted values. The latter is possibly due to some uncertainty in judging the precise break location, which extends over a length of almost two feet (Figure 5). However, even in this case -- recognizing that the corresponding support slings do not fully constrain the

pole at the GL, and may represent a realistic simulation of an actual in-the-ground pole -- the 5%LEL based upon the original measured, unadjusted data exceeds the 2040 lbs level.

The Powertrusion PT-A tests include five 45 ft. poles and two 40 ft poles, thereby accounting for the relatively wide variation of the o riginal measured strengths as indicated in Figure 6 and Table 2. For a pole of uniform cross-section and properties along the length of the pole, in contrast to a tapered product, the allowable cantileve loads may be readily related in inverse proportion to the moment arms (e.g., to break location) -- similar to the procedure described above for obtaining the adjusted strengths to account for the GL support condition. This principle allows cost -effective full-scale testing to characterize the pole strength, independent of length. Thus, the adjusted values to a 40 ft. pole length (32. ft. moment arm) are very consistent, with a COV of only 2%.

Table 2
Strength Results of Individual 40 ft. PFRC Cantilever Pole Tests

Test Series	Mean Strength, Measured/Adjusted (lbs.)	COV, Original/Adjusted (%)	5%LEL, Measured/Adjusted (lbs.)	Remarks
TF-1	2757 / 2459	1.99 / 1.41	2667 / 2402	Fully constraining support
TF-2	2333 / 2202	4.22 / 8.33	2171 / 1900	Sling support
PT-A	2749 / 2844	6.98 / 2.07	2434/ 2747	Fully constraining support; 45' and 40' poles
PT-B	2433 / 2276	4.73 / 3.89	2244 / 2131	Fully constraining support; one pole with seams at outer/inner/lateral faces
PT-C	2532 / 2439	0.36 / 1.93	2517 / 2362	Fully constraining support; one pole with seams at outer/inner/lateral faces
Average	2561 / 2444	3.66 / 3.53	2406 / 2308 (2361 / 2278)*	Represents averages of the 5 individual sets of data

* The slightly lower average 5%LEL values indicated (2361/2278) reflect the relatively low number of available data points within each test set and a 50% confidence level[10]. (This quantity does not include the PT-C test set which only consists of two data points.)

The Powertrusion PT-B tests include one pole oriented with the longitudinal seams at the less desirable outer/inner/lateral bending faces, possibly accounting for the somewhat lower measured and adjusted strengths indicated in Figure 6. Nonetheless, this data poin is within the overall range of data displayed and it is comforting that this is not an excessively weak orientation, in case of misapplication in the field. Indeed, the difference between this individual data point and another PT-B test corresponding to an

"identical" product (same day manufacture, same resin, …), indicates only 3 – 6 % reduction in the original measured or adjusted strength.

The Powertrusion PT-C tests comprise two poles from the same day production lot. One pole was tested using the fully-constraining clamp supports (Figure 2), with rupture occurring at a point 20 inches above the GL. For comparison, the second pole was tested using the support system illustrated in Figure 12, consistent with ASTM 4923. This test resulted in pole collapse at a point only 8 inches above the GL. This issue is discussed further in the section: **Simulated Ground Support Condition**.

Figure 7 summarizes the cantilever strengths of 40 ft. Powertrusion PFRC poles, based upon the results discussed, in comparison to a Class 4 wood pole. The wood strengths are based upon the ANSI-05.1 standard[1], in combination with the strength factors required by the NESC. The average 5%LEL values shown in Figure 7 for the PFRC poles are based upon a 50% confidence level (see footnote Table 2), reflecting the number of available data points in each test set, consi stent with the SRBD guidelines. Both the measured and adjusted values for the PFRC poles exceed the corresponding mean (2400 lbs.) and 5%LEL (approximately 1600 lbs.) strength of the wood alternative for the three major wood species, based upon a wood CO V of 20%. The 1600 lbs. level corresponds closely to the reduced strengths required by the NESC for wood Grade B construction, based upon the 0.65 strength factor. (For example, using the formula above, a COV of 20% corresponds to the almost identical factor of 0.67 for the 5%LEL value.) The 5%LEL "minimum" strength levels of the PFRC pole also significantly exceed the greater allowed wood strength (2040 lbs.) for Grade C construction.

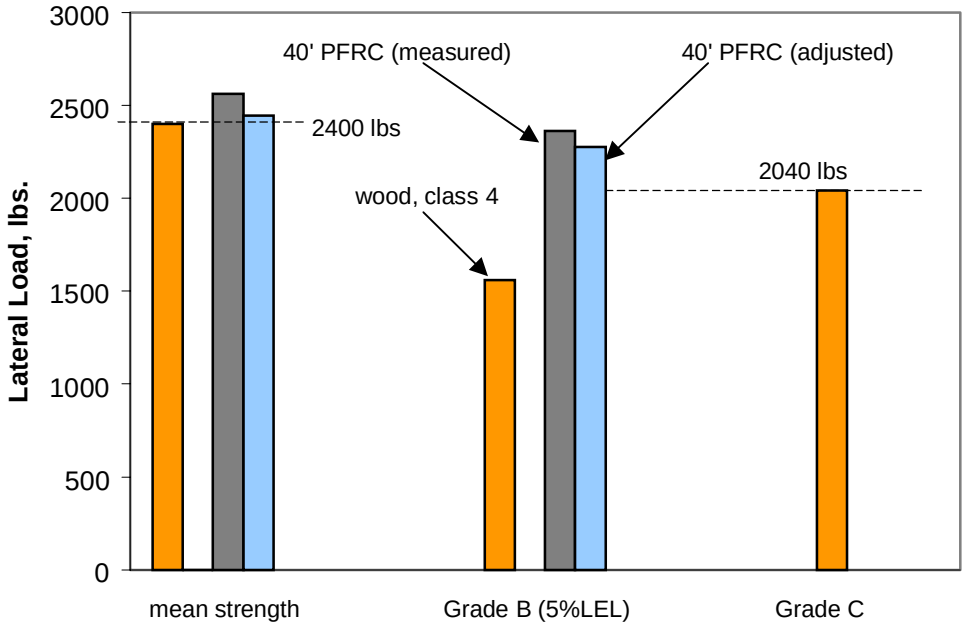

Figure 7
40 ft. PFRC Pole Strength Test Results vs. Class 4 Wood Poles

Stiffness (Deflection)

The pole "stiffness" may be determined during the cantilever tests by monitoring the deflection of the pole as the lateral load increases, as illustrated in Figures 3 and 4. The stiffness depends upon several pole parameters, including length or height above GL, axial modulus of elasticity (E), and the cross-sectional moment of inertia (I) [11]. The product "E I" is commonly used as a measure of the inherent stiffness of a pole of given material and cross-section, recognizing that the length effect may be handled separately. However, for a pole of non -uniform cross-section -- e.g., a tapered wood pole -- and/or non-constant material properties along the pole, an effective E I value must be defined, based upon an pole effective diameter. Such poles are more complex to analyze and evaluate than the uniform PFRC product.

For a given pole strength and ability to withstand various static and dynamic loads, greater pole stiffness would be considered desirable to avoid potential problems that ma be introduced by excessive deflections (e.g., clearances, ...). Increased stiffness als results in greater resistance to column buckling (and associated "P-Δ" effects), as discussed below.

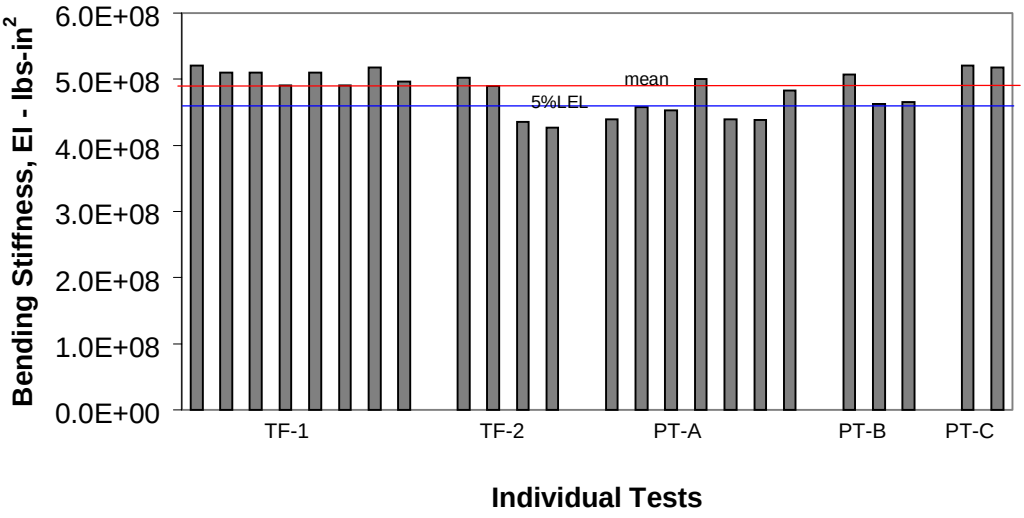

Figure 8
Stiffness Results of Individual PFRC Pole Load Tests

The individual values of stiffness parameter, E I, of the PFRC poles, as determined from the cantilever tests, are shown in Figure 8. These stiffness values were determined by application of the formula

$$\text{lateral force} = 3\,(E \bullet I) \times (\text{lateral deflection at load})/(\text{distance to load})^3$$

and accounting for possible effects due to the particular test procedure employed (e.g., local pole movement/rotation at GL, and location of pole deflection measurement). Similar to the strength analysis, he mean and 5%LEL values are also indicated in Figure 8. The average COV, across all data sets, is less than 5 percent.

The stiffness values as determined at the Powertrusion facility (PT-A, PT-B, PT-C) are slightly conservative (i.e., stiffness underesti mated) since the deflection is measured along an arc, and not perpendicular to the original pole position, as consistent with the more conventional practice (ASTM 1036). This effect underestimates the stiffness by a small degree -- approximately 1%. This amount is likely lower than the degree of accuracy in the overall data sets, including those performed at the other facilities, and is small compared to the COV.

Table 3
Stiffness Results of Individual Cantilever PFRC Pole Tests
(Lateral Load 2 ft. from Tip)

Test Series	Mean Stiffness, E I (lbs.-in.^{2}, 10^8)	COV (%)	5%LEL (lbs.-in.^{2}, 10^8)	Remarks
TF-1	5.06	2.33	4.86	
TF-2	4.63	8.17	4.01	Requires corrections for movement at base, load application point
PT-A	4.59	7.19	4.04	Slightly underestimates stiffness (1%)
PT-B	4.78	5.21	4.37	Slightly underestimates stiffness (1%)
PT-C	5.19	0.49	5.15	Slightly underestimates stiffness (1%)
Average	4.85	4.68	4.49	Represents averages of the 5 individual sets of data

Figure 9 compares the stiffness of the PFRC poles with that of 40 ft. long Class 4 wood poles, for the three common species. The taper effect of the wood poles requires an assumed location for the lateral load, and reflects the particular dimensions of the pole at

the GL and at the load. For the present purposes, the load is again assumed to be located at 2 ft. from the tip of the wood pole. For determining the cross-sectional moment of inertia, I, of an "equivalent" uniform (solid round) cross-section wood pole, an effective diameter of $(D_{GL}^3 \bullet D_f)^{1/4}$ applies, where D_{GL} is the diameter at the ground-line and D_f is the pole diameter at the point load of load applicati [3]. This diameter is located a approximately 30% of the distance to the load point from the GL. It is recognized tha this may somewhat underestimate the stiffness of the tapered wood poles for loads located at lower locations, including joint telephone and power applications, and is estimated to be on the order of several percent (e.g., 5%). The 5%LEL values for the PFRC and wood poles are also indicated in Figure 9, since these would be more appropriate for design purposes than the mean values, consistent with industry guidelines for FRC[12] and wood poles[13].

The tapered wood poles are significantly stiffer than the PFRC poles with respect to lateral deflection under transverse load. However, the wood stiffness varies widely among the different species, and the levels are considerably closer in magnitude with regard to the 5%LEL values. It should be noted that the NESC does not consider the flexibility of the structure for clearance purposes for typical distribution applications (less than 60 ft height, Grade C construction).

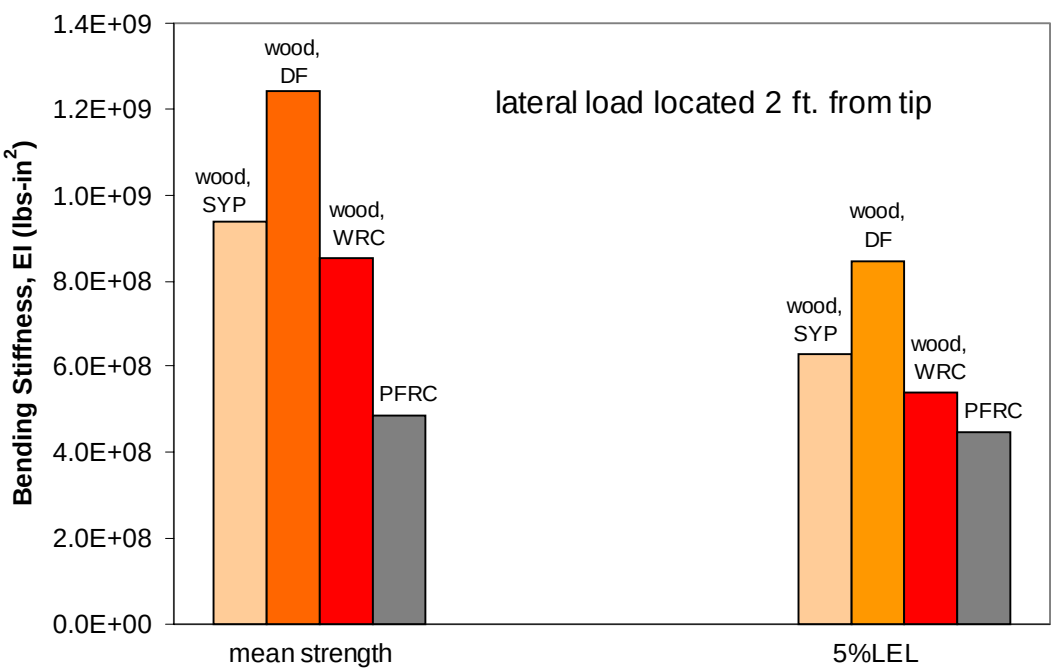

Figure 9
PFRC Stiffness vs. 40 ft. Class 4 Wood Poles

"Euler Column" Buckling

The lateral stiffness characteristics of a column or pole is a primary parameter in determining its resistance to column buckling. Other important parameters include the unsupported column length and the end support conditions. An appropria e formula for determining a conservative estimate of the column buckling resistance, for design purposes[3], is:

$$\text{maximum allowable vertical load} = \pi^2 (E \bullet I)/(\text{length from GL to load})^2$$

Figure 10 illustrates the column buckling loads for 40 ft. PFRC poles in comparison to that of similar length, Class 4 tapered wood poles. In this case, the cross-sectional moment of inertia of the equivalent uniform cross-section wood pole is based upon an effective diameter of $(D_{GL} \bullet D_w)^{½}$, where D_w is the pole diameter at the point of load application[4] For the vertical load at 2 ft. from the tip, this diameter is located at approximately 55% of the distance to the load point from the GL, and is different than that determined for the purpose of predicting lateral deflections due to transverse load, as discussed above. Thus, the effective diameter of the tapered pole depends upon the type of loading, as well as the location of the load. Similar as for stiffness, the calculations may somewhat underestimate the stiffness of the tapered wood poles for loads located at lower locations (e.g., on the order of 10%). Based upon the corresponding mean values of E I, the PFRC poles are somewhat weaker than the wood poles, although very close t that of Western Red Cedar. However, the 5%LEL strengths would be the more appropriate to consider, for which the buckling resistance of the PFRC pole compares very favorably to that of the wood poles, being exceeded by only that of Douglas Fir.

It is useful to interpret the results of a previous buckling test on a PFRC pole in relation to the present predicted strength. In addition to the cantilever bending tests performed at Test Facility #2 (TF-2 data), additional tests were performed at that facility on two poles placed under axial compression by tensioned guys attached to the very top of the 40 ft. PFRC poles secured at the GL. Due to the load application at an unsupported length f 34 ft. (= 40 ft – 6 ft. depth), slightly greater than the nominal 32 ft. nominal distance, a 5%LEL buckling load of 26,600 lbs is predicted[5] -- somewhat lower than the 30,000 lbs. level indicated in Figure 10. (A mean value of 28,750 lbs. is predicted for this test condition.) During the test, the pole withstood the maximum load capability of the test equipment -- approximately 16,000 lbs. -- without any evidence of instability.

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[3] The buckling load is strongly dependent upon the end support conditions. The formula provided is that recommended by Ref. 13, and is also generally consistent with Ref. 14, recognizing differences in the selected parameters.

[4] Based upon the Gere and Carter method[13].

[5] The end conditions for this case are different than that for an actual distribution pole application, but th predicted buckling load is also given by the conservative relation given previously.

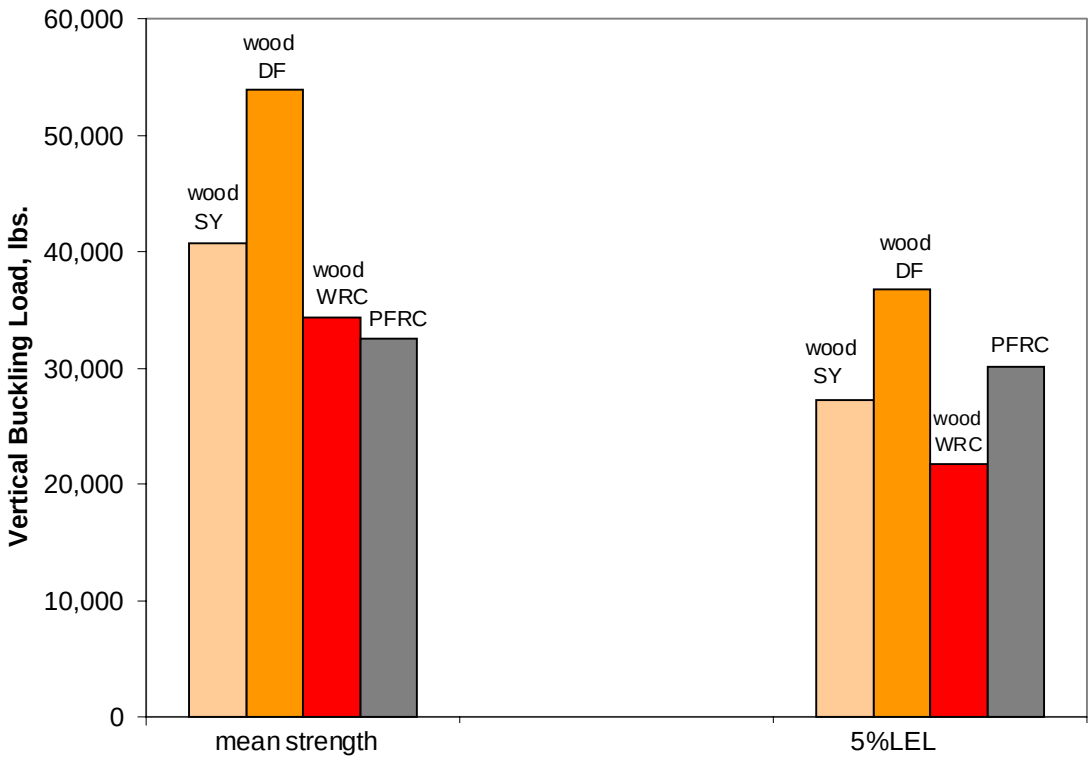

Figure 10
40 ft. PFRC Buckling Strength vs. 40 ft. Class 4 Wood Poles

Simulated Ground Support Condition

During the process of developing appropriate full-scale pole test conditions consistent with industry standards, the intent of the SRBD guidelines, and representative of actual field conditions, several tests were performed at the Powertrusion facilities, including data reflected in the results of this report. Of particular interest, is the degree of support and constraint provided at the GL and below. For example, the full -constraining support represented in Figure 2 would result in an optimistically high pole strength, requiring a possibly overly conservative, downward adjustment in strength. Thus, during January 2002, an initial attempt was made to reduce the full perimeter support at the base by inserting a wooden block at the front and rear faces of the PFRC pole, allowing ovaling at the other 6 faces of the octagon. This attempt, however, resulted in a severe load concentration and local collapse at the front face of the pole at the GL, a nd a significantly reduced pole strength -- at less than 1600 lbs, as illustrated in Figure 11. This effect is consistent that with that experienced testing tapered tubular FRC poles, using a different manufacturing process, as reported elsewhere[9]. These experiences indicate that care must be used during pole installation and soil backfill to avoid placing or wedging rocks immediately adjacent to a tubular pole, of any type or material

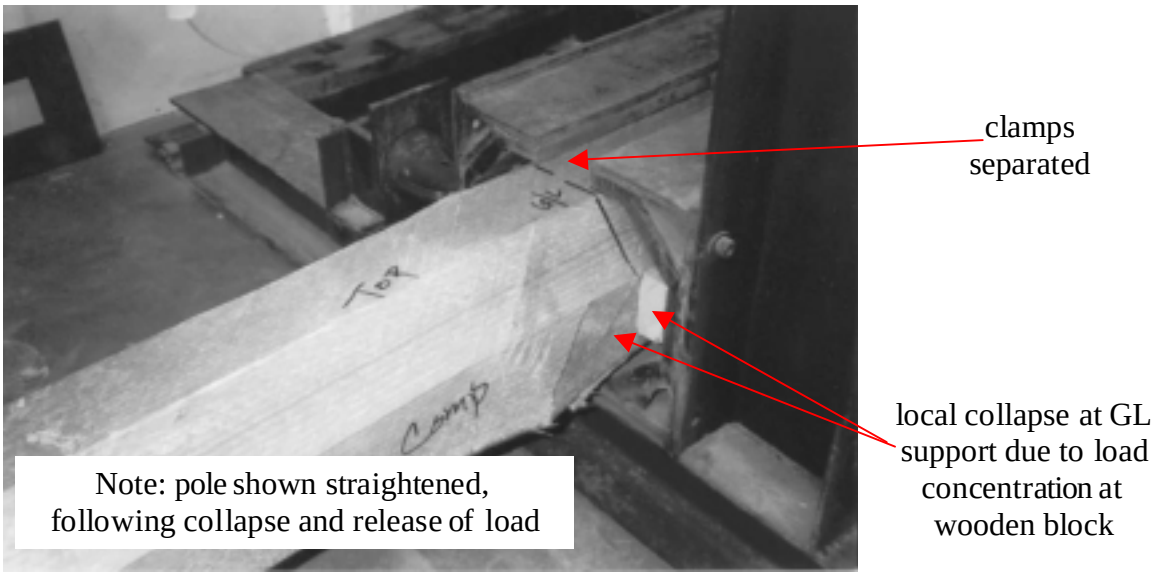

Figure 11
Local Collapse at GL Due to Load Concentration at Support

Recognizing that ASTM 4923 includes a standard test procedure for a hollow, tubular FRC pole, specifying non-constraining, single-sided supports at the GL and butt, including specific requirements for cushioning at these locations, this technique has been recently adopted for the octagonal PFRC pole (see Figure 12). The ANSI C136.20 procedure utilizing a sling support would also be a reasonable alternative, but the ASTM 4923 approach is preferred because it is intuitively more representative of actual soil conditions, as well as its adoption by the AASHTO specificati [15] for FRC light poles, and its explicit recommendation for application to utility poles as well as lighting poles.

Therefore, in February 2002, the two PT-C pole tests were performed. One test used the fully-constraining clamp supports, with resulting pole failure occurring at 2538 lbs., at a point 20-in. above the GL. For comparison, the second test was performed on an essentially identical PFRC pole (same day production), but using the support system illustrated in Figure 12. This test was successful, resulting in pole collapse at a slightly lower load of 2525 lbs., but at only 8 inches above the GL (Figure 13), appearing to mee the desired objectives. The measured and adjusted strength for both poles are included in Figure 6 and Table 2. It is the intention, however, to eliminate the need to "adjust" (i.e., reduce) the measured load when employing an appropriate test procedure, such as the ASTM 4923–consistent technique employed on the second pole, since it is likely to be unnecessary and overly conservative. It is recommended that cantilever strength tests be performed with an in-the-ground pole(s), to verify the degree of support at the GL and below, as well as the details of the failure mode. Such realistic tests would confirm the validity of the selected test procedure, simulated support conditions, and overall analysis.

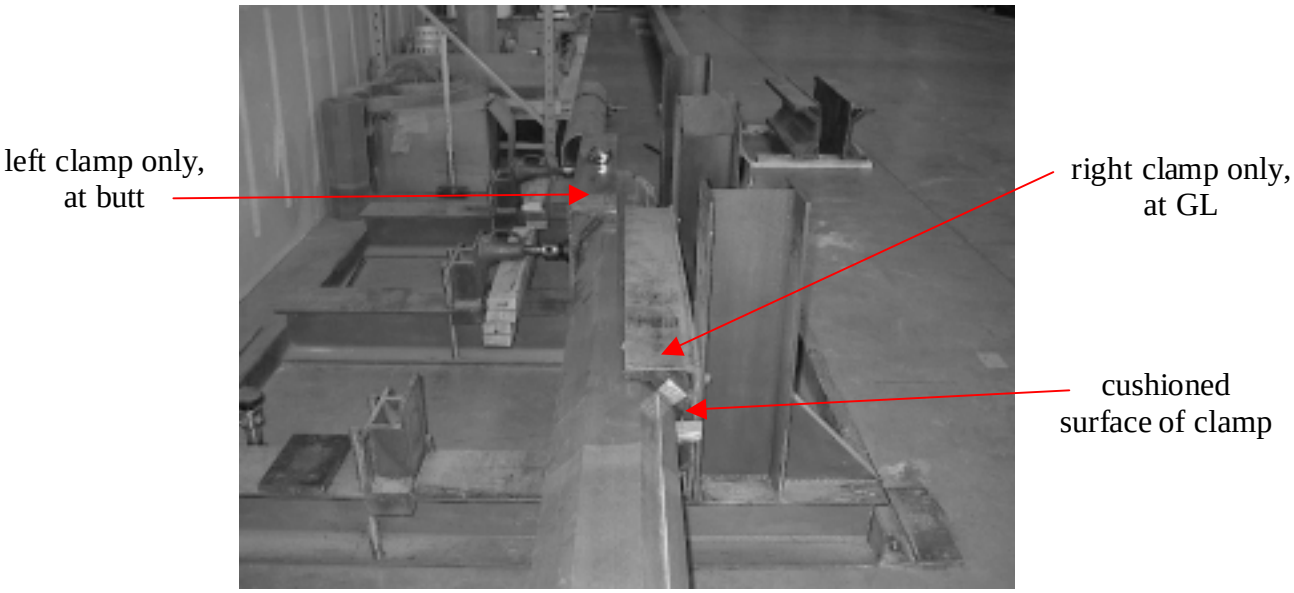

Figure 12
ASTM D 4923 - Consistent Supports (Singl -Sided, Cushioned)

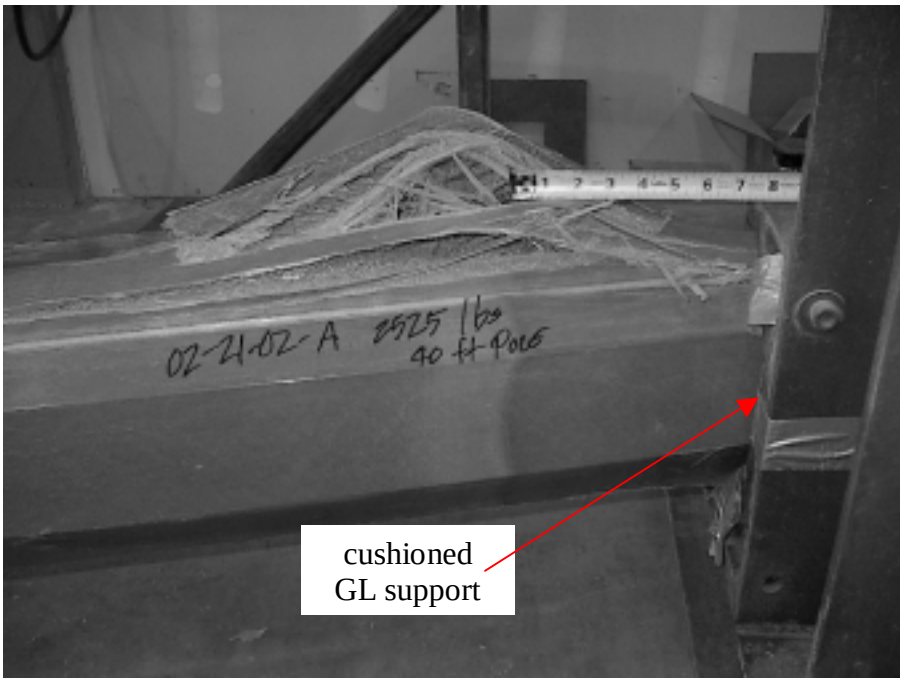

Figure 13
ASTM D 4923 - Consistent Cushioned Supports
(Pole Collapse at 8-in. From GL)

Summary & Recommendations

During the past four years, several series of full-scale mechanical strength tests have been performed on representative Pultruded Fiber-Reinforced Composite (PFRC) poles manufactured by Powertrusion International, Inc. Some of these tests were performed at independent laboratories, and others, more recently, at Powertrusion facilities in Las Vegas, NV. The large majority of data was collected for 40 ft. long poles, of 10.2-inch width (0.25-in. minimum wall thickness) and octagonal configuration. This pole represents an alternative to the commonly used 40 ft. Class 4 wood poles -- including Grade C construction as defined in the National Electrical Safety Code (NESC) and representative of typical utility applications. The poles were subject to lateral bending loads, applied 2 ft. from the tip, consistent with standard cantilever type tests for wood poles. The load-carrying capability of the PFRC poles is dependent upon the details of the support condition at the simulated ground-line (GL), due to the local collapse failure mode associated with hollow, tubular products, and the results were therefore interpreted and adjusted accordingly. The test results were analyzed with respect to the lateral bending strengths, as well as flexural stiffness and column buckling loads (based upon lateral and vertical loads, respectively, applied at 2 ft. from the tip), and compared to wood pole alternatives. The 40 ft. PFRC poles were characterized statistically to determine the mean and 5% lower exclusion limit (5%LEL) values, consistent with evolving reliability based design procedures and potential implementation in the National Electrical Safety Code.

The analyses show

- The historical mean strength of the 40 ft. PFRC poles (2561 lbs. measured, or 2444 lbs. adjusted to account for ground-line support conditions) exceeds that of Class 4 wood poles (2400 lbs).
- The historical 5%LEL strength of the PFRC poles (2361 lbs. measured, or 2278 lbs. adjusted) significantly exceeds that of Class 4 wood poles (approximately 1600 lbs.). The 5%LEL value represents an appropriate design strength for highly reliable construction (e.g., NESC Grade B).
- The 5%LEL strength of the PFRC poles also significantly exceeds the higher allowed strength of Class 4 wood poles (2040 lbs.) appropriate for the widely used NESC Grade C construction.
- The determined lateral bending strengths of the constant cross-section PFRC poles may be readily extrapolated to PFRC poles of different lengths in inverse proportion of the moment arm to the ground-line or its proximity.
- The mean stiffness of the PFRC poles is approximately half that of 40 ft. Class 4 wood poles; the 5%LEL design values, however, are considerably closer in magnitude, with a relatively small difference from some wood species.
- The calculated mean column buckling resistance of the PFRC poles (32,400 lbs.) is somewhat lower that of Class 4 wood poles; however, the 5%LEL

design value (30,000 lbs.) of the PFRC poles is greater than that of two of the three major wood species considered.

The results indicate the capability of readily producing high quality PFRC poles of low variability and with sufficient strength to meet requirements satisfied by conventional wood poles. Based upon the product history, and independent sets of test data, a coefficient of variation (COV) of approximately 3½ percent is indicative of the process. The variability would be expected to further decrease as the product design and manufacturing procedures stabilize, resulting in potentially higher 5%LEL levels, for given mean values.

The evaluation of the PFRC poles relative to Class 4 wood poles is based upon a 40 ft. total pole length. For longer PFRC poles of the same 10.2-in. width, 0.25-in. thickness, octagonal cross-section, the strength and stiffness would be reduced relative to Class 4 wood poles whose dimensions (diameter) are increased to compensate for the greater length. Conversely, the strength and stiffness of shorter PFRC poles would be increased relative to wood poles. In general, the uniform cross-section and material properties along the axial direction of the PFRC product facilitate its evaluation and characterization, and the implementation of reliability based design procedures.

The quantitative results are considered to represent conservative estimates of the strength of the PFRC poles. The adoption of the ASTM 4923 test procedure and simulated GL support details should eliminate the need to adjust (reduce) the strength to lower than that directly measured. In recognition of the importance of the simulated ground support conditions, it is recommended that realistic in-the-ground strength tests be conducted to verify that the soil support condition, and corresponding failure mode, is properly represented by the details of the selected test procedure.

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