Bolt Shear Capacities

The allowable shear capacity of bolts used in existing towers may not be readily available. As described in the previous section, bolts used in transmission towers changed from A307 to A394 and then to the currently used A394 Type 0 bolt. Other bolts types were also used and often times, the only information provided on the design or detail drawings or the bill of material is the bolt diameter. Therefore, if the bolt shear capacity or allowable tensile strength of the bolt is unknown, it is recommended that a random sample of bolts be removed from the existing towers and tested per ASTM F606 "Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers, and Rivets". The sample sizes recommended by ASTM A394 Type 0 for determination of the mechanical properties vary with the quantity of bolts. It is recommended that a minimum of one bolt for every 10,000 bolts of like diameter and tower manufacturer be tested with no less than ten bolts tested. An upper limit of 100 bolts seems like a reasonable limit for long lines that use towers fabricated and installed at the same time. Bolt tests should be completed with threads in the shear plane unless washers, recessed nuts, or other methods are used to eliminate failure through the threads.

It should be reiterated that the new A394 Type 0 bolts have a shear capacity that is approximately 20% greater than the A394 bolts originally used before 1984. Many engineers that are completing the analysis of these older towers are not aware of this difference. ASTM A394-78 (1978) tabulated shear capacities for bolts through the threads and includes a note indicating that "a cooperative test program determined that galvanized bolts having a minimum tensile strength of 415 MPa (60 ksi) would support, in single shear through the threads, the loads given ...". Values given in ASTM A394-78 were 40.5 kN (9.1 kips) for 15.9mm (5/8″) bolts and 60.5 kN (13.6 kips) for 19mm (3/4″) bolts with threads in the shear plane. Current shear capacities for A394 Type 0 bolts with threads in the shear plane are 49.6 kN (11.15 kips) for 15.9mm (5/8″) bolts and 74.1 kN (16.65 kips) for 19mm (3/4″) bolts through the threads per ASTM A394-00 (2000).

Member Use Ratios

Often times during the analysis of existing towers using new loading criteria, member loads will exceed the member design capacity according to ASCE 10-97. This overstress is the result of several factors. First, it is readily apparent that the new load requirements may result in greater members loads. Second, the load distribution in the tower members will differ between force diagrams and the more sophisticated analysis techniques used today. Finally, the allowable member capacity determined using ASCE 10-97 may produce differences with the original design capacity. The results may require that the members be reinforced with additional bracing, that bolts be added to the connection, or in some cases that the member be replaced. In some cases, however, it may be reasonable to accept member use ratios that are greater than 100%. The *member use ratio* is defined as *the member load divided by the member capacity*. If member use ratios are less than 110% and occur for non-legislated load cases, it may be agreeable to the owner to accept ratios that exceed 100% use. The risk in exceeding the allowable capacities should be considered and discussed with the owner. It is not unreasonable to accept use ratios that are slightly greater than 100%. In extreme event cases, such as a high wind case, it can often be shown that a slight reduction in the wind values will enable the member load to meet the design capacity. This reduced loading can be presented to the owner so that they have a better understanding of the risk associated with this assumption.

Man-loads on Horizontal Members

The ASCE 10-97 Commentary suggests "that tower members that may be used for support by maintenance personnel when climbing a tower be capable of supporting a vertical load of 1100 N (250 lbs.)." Many times man-loads were not even considered when sizing these members. At times this criterion will not be met on existing towers and in some cases the owner will require more severe conditions. It is suggested that in these circumstances the owner be notified of these limitations and consider alternative maintenance procedures. At the same time, it would be appropriate to bring the latest OSHA recommendations for climbing towers to the owner's attention. If necessary, selected key redundants could be replaced or braced to increase the member capacity. However, a wholesale change out of all redundants to meet this criterion would not be considered a reasonable approach.

Minimum Support By Redundant Members

Redundant members in many older towers were designed to meet maximum L/r limits. Normally, the load in the redundant required to laterally support the member it braced was not calculated. When upgrading/uprating existing towers it is often required to install additional redundants to support the main members in the tower such as the leg and cross arm chords. The new redundant members should be designed to meet the criteria set forth in ASCE 10-97. When modification of the redundant scheme is required, existing adjacent redundants should be checked for this same criterion. However, it is neither recommended nor reasonable to expect that an existing tower should have the complete redundant system replaced to meet this criterion. The historic performance needs to be considered when making changes to the redundant system.

Test Data Used in the Upgrade/Uprate of Towers

ASCE 10-97 states in Section 3.16 that "Design values other than those prescribed...may be used if substantiated by experimental or analytical investigations." Similarly, it says " Where tests and/or analysis demonstrate that specific details provide restraint different from the recommendations of this section, the values of KL/r specified in this section may be modified" in Section 3.7.4.6. This provides the designer the opportunity to use test results in making decisions that may contradict ASCE 10-97 recommendations. Therefore, use of full-scale test data as well as component test data can be valuable in the decision-making process when upgrading/uprating towers. Component testing may include connection tests, bolt tests or testing of attachment plate details. Test results may allow the designer an opportunity to use or modify the existing detail to meet the new load requirements yet not meet the recommendation provided in the ASCE 10-97 standard. In addition, ASCE 10-97 allows the engineer to use detailed analytical investigations to justify alternative solutions and at times these may be appropriate.

Other Issues

ASTM Material Specifications Used in Older Towers

Design documentation for older towers may not identify the type of steel used and at times, archive information on existing towers is limited. Therefore the following historic information may be useful to the designer when modeling older towers.

According to the AISC 5th Edition (AISC 1947), the specification for A7 steel was a combination of the "Standard Specification for Steel for Bridges (A7-36) and for Steel for Buildings (A9-36)." These specifications were originally adopted in 1901 and combined to A7 steel in 1939. AISC 5th Edition provides seven pages describing the Standard Specification for Steel for Bridges and Buildings A.S.T.M. A7-46. For designers, the most important information is the mechanical properties of A7 steel. Tensile strength (F_u) is given as a minimum of 415 kN (60ksi) to a maximum of 496 kN (72 ksi). Yield point (F_y) is given as 50% of the tensile strength but in no case less than 228 kN (33 ksi).

The AISC 6th Edition (AISC 1963) first referenced A36-63T steel in 1963. Tensile strength (F_u) values were given as 400 kN (58 ksi) to 552 kN (80 ksi) with a yield point (F_y) of 248 kN (36 ksi). ASTM Specifications indicate that A36-60T was first published in 1960. AISC 6th edition also provided the ASTM Specification for A7-61T steel. This was the last edition in which A7 steel was listed. Therefore, it appears likely that steel used for transmission towers prior to 1960 was A7 steel, between 1960-1970 either A7 or A36, and thereafter a minimum of A36 steel. In the AISC 6th Edition, high-strength steels were also introduced. These included A242-64T, A440-64T, A373-58T, and A441-63T. It is possible that one of these high-strength steels was used in existing compression members with low L/r values.

In 1964 the Canadian Standards Association introduced G40.12 steel. Mechanical properties for G40.12 steel were a minimum tensile strength (F_u) of 427 kN (62 ksi) with a yield point (F_y) of 303 kN (44 ksi) for angles less than 19.1mm (¾″) thick. The yield point was decreased to 276 kN (40 ksi) for thicker angle sizes.

Published in 1970, the AISC 7th Edition (AISC 1970) dropped reference to A7-61T and A373-58T steels. A36-70a, A242-70a, A440-70a, and A441-70a steels were still included as referenced specifications. A572 steel was first referenced by the AISC 8th Edition (AISC 1980) in 1980, but was originally included in the ASTM Specification in 1966. A588 steel was first referenced by AISC in the 7th edition in 1970 but originally included in ASTM Specifications in 1968.

Currently, ASTM A36 and A572 are the most prevalent specifications used for steel transmission tower hot rolled angles and plates in the United States. Canadian suppliers are currently using CSA G40.21 steel in lieu of A36 steel with a minimum tensile strength (F_u) value of 448 kN (65 ksi) and a maximum of 621 kN (90 ksi) and a yield point (F_y) of 303 kN (44 ksi).

Original Compression Formulas Used in Older Tower Designs

Prior to the first edition of Manual 52 (published in 1971) designers often used compression formulas that were inherent to their company. The formulas were usually straight-line curves resulting in allowable stresses, which many times are lower for L/r's less than Cc and greater for L/r's more than Cc (Bergstrom 1960). However, while the allowable stress value (F_a) for the lower L/r's may be higher than currently recommended by ASCE 10-97, K factors must now also be included. Older designs normally did not use an effective length factor (K) to modify the slenderness ratio and ultimately the allowable compression stress. Tension values were also calculated without consideration for block shear and at times without checking bearing capacity. ASCE 10-97 now limits the design tensile stress (F_t) to 90% of the specified minimum yield stress (F_y) for tension members. It also reduces the net area of unequal leg angle connected by the short leg to that of an equal leg angle using the short leg width. Historic designs normally did not consider these reductions.

Members in older towers will often indicate smaller capacities than currently calculated using today's design formulas. For example, the capacity of leg members using the ASCE 10-97 with K=1 (Curve 1) often will result in an allowable leg load that is greater than the original design capacity. Using the same L/r and ASCE 10-97 recommendations to determine F_a for a strut that is eccentric at both ends, may result in a smaller capacity than determined by the original design formulas due to the inclusion of the K factor. The engineer needs to be aware of why these differences occur, which will allow a better understanding of which members will require modification in an upgrade/uprate process.

Summary

The transmission systems at many utilities include older towers that were designed and installed prior to the current Guidelines and Standards developed by ASCE. Thus some of the design requirements within ASCE 10-97 will not be met by these existing tower designs yet at the same time many of these towers have provided excellent service life. While it is recommended that ASCE 10-97 should be used in development of new tower designs, the historic performance of the tower needs to be considered when reviewing existing tower designs. Every reasonable effort should be made to meet the requirements of ASCE 10-97 when making modifications for new loads on existing towers, but understand that it is not realistic to expect that every facet of the ASCE 10-97 can be applied to older towers. Hopefully, these guidelines will assist the engineer in making reasonable decisions when upgrading and uprating transmission structures.

References

AISC. (1947). *Steel Construction – A Manual for Architects, Engineers and Fabricators of Buildings and Other Structures, 5th Edition*, American Institute of Steel Construction, New York.

AISC. (1963). *Manual of Steel Construction, 6th Edition*, American Institute of Steel Construction, New York.

AISC. (1970). *Manual of Steel Construction, 7th Edition*, American Institute of Steel Construction, New York.

AISC. (1980). *Manual of Steel Construction, 8th Edition*, American Institute of Steel Construction, Chicago.

ASCE 10-97. (2000). *Design of Latticed Steel Transmission Structures*, American Society of Civil Engineers, Reston, Virginia.

ASCE Manuals and Reports on Engineering Practice No. 52 (Manual 52). (1988). *Guide for Design of Steel Transmission Towers, 2nd Edition*, American Society of Civil Engineers, New York.

ASTM A394-78. (1978). *Standard Specification for Galvanized Steel Transmission Tower Bolts*, pages 362-366.

ASTM A394-00. (2000). *Standard Specification for Galvanized Steel Transmission Tower Bolts.*

Bergstrom, R. N., Arena, J. R. and Kramer, J. M. (1960). *"Design of Self Supported Steel Transmission Towers"*, Journal of the Power Division – American Society of Civil Engineers, June 1960, New York.

Havard, D. G. and Perry, O. C. (2000). *"Lattice Tower Member Fatigue and its Control Using a Novel Damping Scheme"*, Paper 0-7803-6423-6, IEEE.

Spun Concrete Poles for Electrical Transmission Structure Applications - Continuing to Push the Envelop of the Technology

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Abstract

It has been nearly 100 years since the first spun concrete pole was produced in Europe, and more than 30 years since the technology was first introduced here in the United States. However, the technology advancements of the past 8-10 years that have propelled the use of spun concrete poles into an ever increasing, and significant role in today's design and construction of high voltage electrical transmission lines. Significant technology advances have occurred in three key areas. First, the types and quality of the raw materials utilized in the production of high performance concrete have dramatically improved, and in some instances have just recently been developed. Second, scaled up manufacturing methods and equipment to produce high quality, stronger, longer length poles is being realized. And third, with meaningful Research & Development investment in the technology, significant and innovative enhancements to the engineering design technology of spun concrete poles are being utilized. Spun Concrete poles today are not just relegated to small distribution, sub-transmission, or wood maintenance applications. Spun concrete poles today are cost effectively being designed and utilized in major transmission line projects up to and including 345kV and 500kV Lines. In addition to a brief history of these interesting structures, this paper will provide a "state-of-the-art" report on the technology of spun concrete poles for electrical transmission structure applications in the United States

Introduction

It has been nearly 100 years since the first spun concrete pole was produced in Europe, and more than 30 years since the technology was first introduced here in the United States. However, exciting new technology advancements that have occurred in just the past decade that have propelled the use of spun concrete poles into an ever increasing, and significant role in today's design and construction of high voltage

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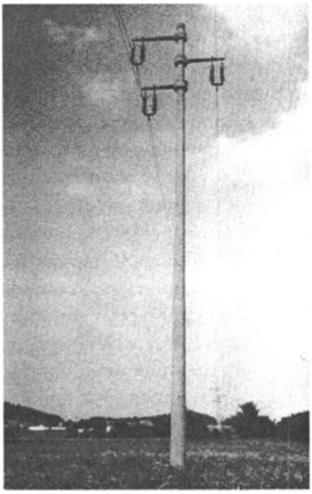
electrical transmission lines. The economics and readily available raw materials: sand, coarse aggregate, cement, and reinforcing steel, make the generic concrete pole one of the most commonly utilized manufactured poles in the world. The longer, stronger, more reliable spun concrete poles being produced today are just recently beginning to shed their nearly 100 year old identity as poles only particularly suited for lower voltage lines. Spun concrete poles today are cost effectively being designed and utilized for a much wider wide range of major transmission projects up to and including 345kV and 500kV transmission lines. Entering this new millennium is a good time to provide a state-of-the-art report on the exciting new technologies being utilized for producing spun concrete poles utilized in electrical transmission structure application in the United States.

History of Spun Concrete Poles

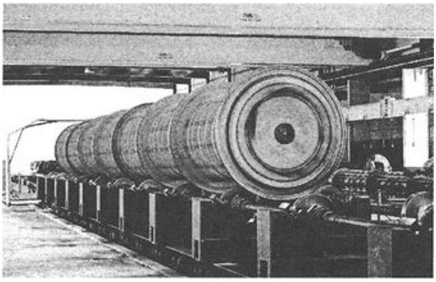


It is important in any review of current technologies to first review the historical perspective. There have been many pioneering efforts in the development of spun concrete poles in the United States. In 1907, the firm Otto and Schlosser in Meiszen, Germany, developed the first spinning machine for producing hollow, lighter weight, "spun" concrete poles (Von Fiss 1989). Acknowledgement must also be give to the spearheading efforts of Eugene Freyssinet, of France, and Gustave Mangnel of Belgium, in the 1930's and 40's, for development of the concepts of prestressed concrete. Freyssinet, who is generally regarded as the father of prestressed concrete, also was the first to apply prestressing principles in the production of concrete poles (Rogers, 1984). These prestressed concrete poles were much improved in that they were more elastic and could carry significantly higher loads without cracking. The combination of prestressing technologies and spinning technologies during the mid 1950's provided a significant catalyst in the acceptance of spun concrete poles for use by electric Utilities in Europe. These prestressed, spun concrete poles were stronger, more durable, and much lighter in weight than previous types of concrete poles. The improved characteristics made these new poles easier to transport and erect.

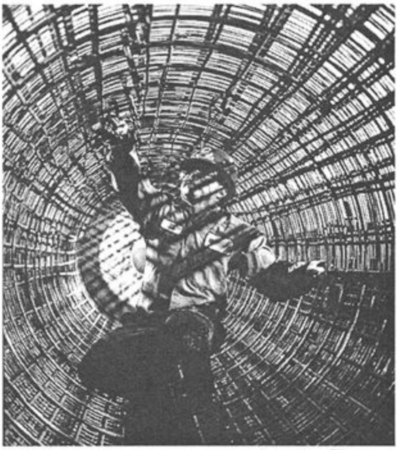
Statically cast, prestressed concrete poles were first utilized in the United States (primarily in Florida and Virginia) in the mid 1950's. A historical perspective of spun concrete pole in the United States would be incomplete without reference to the 1960's spearheading efforts of Tom Rogers of Virginia Electric Power Company, and Bayshore Concrete Products. Their combined efforts brought about the very first electrical transmission line in the United States supported by spun concrete poles. Also of significance, but perhaps not as well remembered was the mid 1970's formation of a Company called REMCO. Soon after selling his tubular steel pole company Meyer Industries, to conglomerate ITT, Roy E. Meyer formed REMCO to begin producing spun concrete poles for electrical utility applications. Due to health issues of Mr. Meyer, REMCO was short-lived, but his company was sold and renamed Power Span. It was Power Span's President, Mr. William Howard, that helped to write the initial Industry Guides for the design and use of spun concrete poles. Ultimately Power Span did not survive as a company, but in 1982, Sherman Industries, a concrete products company in Birmingham, Alabama, was contacted by Southern Company executives for the purpose of exploring the benefits of using spun concrete poles in the Alabama Power and Georgia Power systems. From those discussions, Sherman Utility Structures, Inc. (now known as Newmark International, Inc.) was created. Through Newmark, and it's affiliation with the University of Alabama – Birmingham, significant engineering research and development of spun concrete poles was initiated in 1983. Dr. Fouad H. Fouad, currently Chairman of the Department of Civil Engineering with UAB has been instrumental in leading this significant research. Additionally, specific research sponsored by Florida Power & Light, and directed by Dr. C. Jerry Wong has contributed greatly to the knowledgebase of spun concrete poles as they apply to electrical transmission structures. It is through the efforts of all of these pioneers that the spun concrete pole has evolved. But the evolution is continuing, and the rest of this paper will discuss the significant evolution of the spun concrete pole over the last decade.

Spun Concrete Poles Grow up in the 1990's

Any evolution of a product such as spun concrete poles, requires two drivers. The first driver is the technology itself. The second driver is the vision and driving force created by the end users in the product's application. The past two decades clearly has seen revolutionary developments in concrete materials and production methodologies. It is being said that the 21st century will see the emergence of high-strength, high performance concrete for use in building and maintaining the world's infrastructure (Nawy 2001). The spun concrete pole manufacturers are applying these new developments to produce taller, stronger, and more durable poles for use in electrical transmission structure applications. These developments have come in many areas.



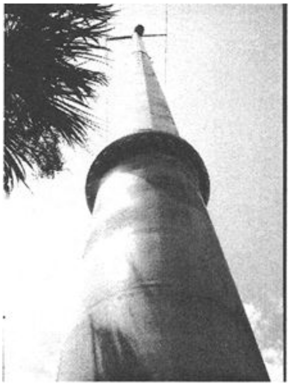
In concrete technologies, significant advancements in chemical and mineral admixtures have been made. Some of these admixtures serve to increase strength by reducing the amount of water to the minimal amounts needed for the hydration process. Others accelerate or slow down curing cycles depending on the need. Contributing also is a better understanding of the materials, including sand, coarse aggregate, and cement. New, specialty Companies, are making chemicals and other ingredients available from worldwide, rather than limited local sources. Concrete compressive strengths in excess of 14,000 psi is currently being achieved in pole structures and it is predicted that with the new admixtures and materials being available, compressive strengths in excess of 20,000 psi will soon be achievable in spun concrete poles. In the early 1990's, just ten or so years ago, compressive strengths of only 8,500 psi were being utilized in spun concrete poles.



The steels used in the reinforcement of spun concrete poles have also contributed significantly to the advancements in technology. New, higher strength, specialty steel strands and other reinforcements have been developed. Steel strand materials are available today that exceed 300 ksi tensile strength. The importance of utilizing higher strength spiral reinforcement steel with better defined and controlled spacing is also contributing to stronger, more durable spun concrete poles. These enhancements, although not as significant at the improvements in the concrete materials, still provides significant improvement over the steels utilized just a few years ago.

As significant as materials improvements, advances in the equipment and technologies of manufacturing spun concrete poles have progressed much in the last decade. Larger diameter molds (up to 1.8 m) are now available in the United States. These molds are designed to be able to withstand in excess of 3,000 kips of prestressing forces. Computer controlled, concrete batching facilities are being utilized to achieve very carefully controlled, consistent high-performance concrete. It is not uncommon to expect from these new batching systems, to produce concrete that has a 28 day strength exceeding 12,000 psi with a standard deviations of less than 250 psi. This is a 400% improvement in typical consistency from just 10 years ago. New, computer controlled spinning machines are being designed and installed that carefully ramp up spinning speeds so as not to segregate aggregates, but yet achieve the 20-30 g's of force necessary to properly compact the concrete and enhance it's durability. Significant improvement in the assembly of the reinforcing cage minimizes wire vibrations during the spinning operation. Wire vibration is one of the major reasons for voids and materials segregation. This improved knowledge, allows the possibility of constructing double and even triple layered caged poles for greater strength poles.

Perhaps as significant as the dramatic improvements in both the materials and manufacturing technologies, the advancements in the design technology also need to be recognized. New, non-linear computer modeling of both materials and structural behavior has allowed the advancements in materials and manufacturing to be effectively utilized. The design technology enhancements has also allowed the consideration of hybrid pole designs utilizing both tubular steel, and spun concrete. And many more full scale structural tests have been added to the base of knowledge and verification on the behavior of these poles for electrical transmission line applications.



Applications of Spun Concrete Poles for Transmission Line Structures

As previously mentioned, the second important driver in the evolution of any product is the vision of end users to find appropriate applications and uses for the product. That vision for applying the available technology of spun concrete poles has come from a number of key electrical utilities in the United States. FPL, the Southern Company, Entergy, TXU, Reliant Energy, and AEP (just to name a few) have all recently been instrumental in the growth and development of spun concrete poles in the United States. Without the willingness to continue to push the development of spun concrete pole technology, and then the same willingness to implement the results of that technology development, there would be no need to write this paper. Indeed, spun concrete poles have "grown up" just in time to become a significant structure alternative for designers of transmission lines.