Guide for the Design of Prestressed Concrete Poles

Prepared by

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This document provides guidelines for the design, manufacture, testing, installation and erection of prestressed concrete poles. Both spun-cast and statically cast poles are considered, including cantilevered, guyed, framed and combined structures. The report was developed jointly by the ASCE Task Force on Concrete Poles and the PCI Committee on Prestressed Concrete Poles. Reader comments on the contents of this document are invited.

PREFACE

Between 1982 and 1984, three reports, titled "Guide Specifications for Prestressed Concrete Poles," "Guide for Design of Prestressed Concrete Poles," and "Concrete Poles: State-of-the-Art," were developed by members of the Precast/Prestressed Concrete Institute's Committee on Prestressed Concrete Poles and subsequently published in the PCI JOURNAL.

In April 1987, after approximately 2 years of meetings and discussions, the Concrete Pole Task Committee of the Committee on Electrical Transmission Structures of the Structural Division of the American Society of Civil Engineers (ASCE) published a conference paper titled "Guide for the Design and Use of Concrete Poles." The committee that produced this paper encompassed several disciplines: pole producers, users, designers and members of the academic community.

As the pole industry grew, however, it became increasingly apparent that it would be helpful, from the standpoint of both users and designers, to be able to refer to a single document on concrete poles that combined the attributes of all four of the above mentioned publications. More companies were entering into the pole business, making both spun and statically cast poles, while technological advances were rapidly being made in the development of new materials and applications of computers and software. The need for clearer communication with the end user and the ability to offer more complete design assistance was also growing.

Coupled with the introduction of newer technologies, producers and users of poles were gaining experience very rapidly. Because the need to revise and update the existing publications was apparent, it became a natural evolution to form a joint ASCE/PCI committee that would include members of both organizations as well as some non-member users and advisors. Such a committee was formed in early 1989. It is the work of that group that has resulted in this document, which is an earnest attempt to combine the best resources of both organizations.

While every effort has been made through various review groups to strive for accuracy and clarity, the user is reminded to always consider the structures described herein as an integral part of a larger system. The user is, therefore, cautioned that the application of these structures should come only after sound engineering judgment has been applied with regard to a particular desired result. Furthermore, as an overall treatise covering a wide variety of applications, this document cannot conceivably satisfy all conditions. The user should bear in mind that sometimes specific local requirements may dictate design and usage conditions that differ from those described herein.

The committee is grateful for the input of its advisory members as well as the comments from those who participated in the development of this report through correspondence. The committee also wishes to express its appreciation to the members of PCI's Technical Activities Council who reviewed this report. Comments and suggestions are invited from readers and users of this document in order to further improve any future revisions of this report. These may be addressed to the chairman of the PCI Committee on Prestressed Concrete Poles or the Technical Director at PCI Headquarters.

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INTRODUCTION

Prestressed concrete poles were among the first applications of prestressing that the French prestressing pioneer Eugène Freyssinet developed in the 1930s. Today, prestressed concrete poles are used in most parts of the world as transmission and distribution structures, substation structures, lighting supports, highway sign and traffic signal structures, and communication structures.

In some countries, such as India, concrete poles are used almost exclusively. In North America, their use is confined to specific regions such as the southeastern United States. Generally, where timber is plentiful, wood poles are used more often. However, the increased cost of wood and the environmental issues associated with the preservation of trees have resulted in an increased use of concrete poles. Therefore, the potential for a much greater use of precast, prestressed concrete poles in the United States and throughout the world is promising.

There are two types of prestressed concrete poles: spun-cast and statically cast. Within those types, several cross-sectional shapes may be available. Spun-cast poles are usually round, but may also be hexagonal, octagonal, or special architectural shapes. The most common shape for statically cast poles is square, although they may also be cast in octagonal, flanged I, or other special shapes.

Because it is inherent in the process, spun poles will always have a hollow core. The size of the hollow core is dependent on the wall thickness. Statically cast poles, however, may be solid or have a hollow core formed by the use of retractable mandrels or fiber voids.

This document provides guidelines and a thought process recommended to be undertaken by the user for the proper design and use of prestressed concrete poles.

CHAPTER 1 — STRUCTURAL CONFIGURATIONS AND POLE APPLICATIONS

1.1 General

Prestressed concrete poles fall into four structural configurations as follows:

- Cantilevered
- Guyed
- Framed
- Combined

Applications for these configurations include electrical transmission, distribution structures and substation structures, highway signs and traffic signals, lighting supports, and supports for communication systems. This chapter includes a description of the configurations and the applications of prestressed concrete poles.

1.2 Configurations

1.2.1 Cantilevered Structures

Most concrete pole structures are cantilevered single poles directly embedded in the earth or supported by a foundation. Typical cantilevered structures are shown in Figs. 1.2.1.a through 1.2.1.d.

Cantilevered structures, often called self-supporting structures, are designed to withstand various combinations of vertical and horizontal loads. Although shear and torsional loads cause stresses on the structure, the design of a cantilevered structure is predominantly controlled by the bending stresses associated with horizontal loads. Horizontal

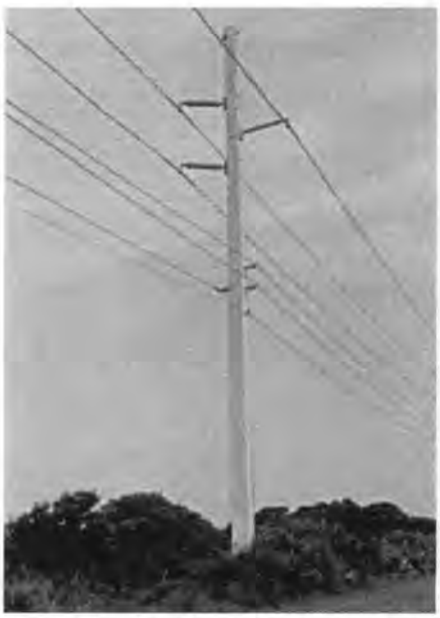


Fig. 1.2.1.a. Cantilevered transmission structures. Left: Square, static cast transmission poles with polymer post insulators and distribution underbuild. Right: Urban transmission line; spun poles with horizontal vee insulators.

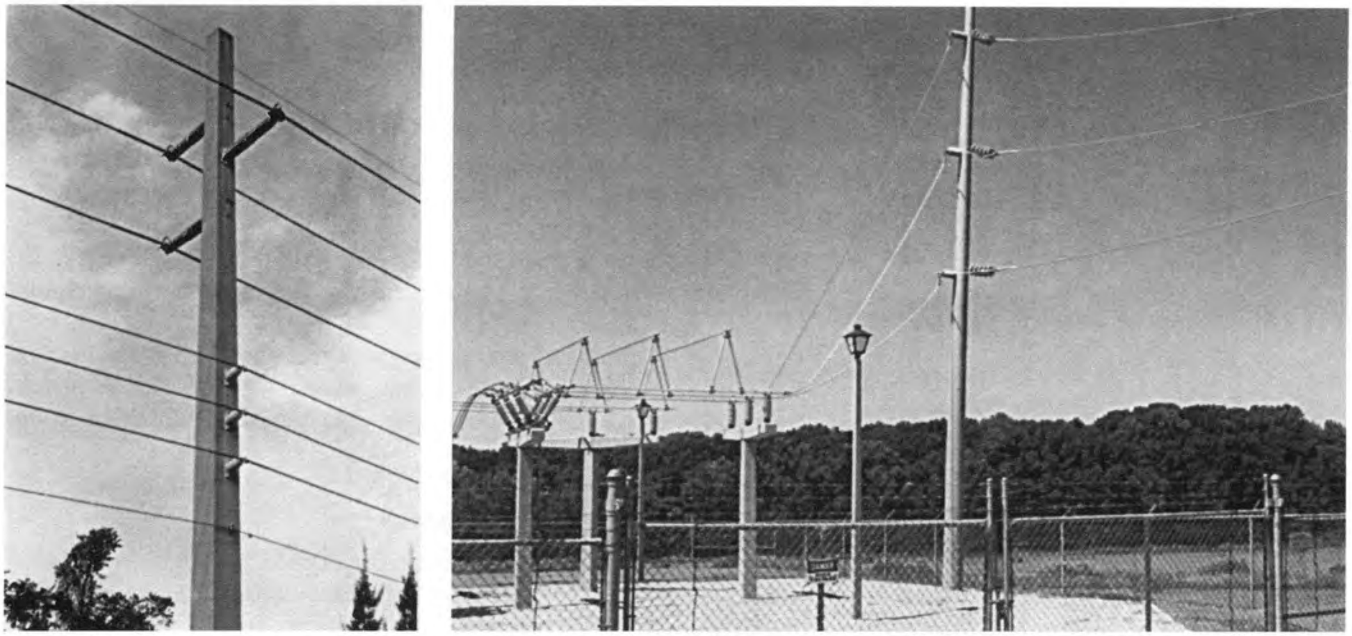


Fig. 1.2.1.b. Cantilevered transmission structure with distribution. Left: Square static-cast transmission poles with horizontal porcelain post insulators. Right: Spun pole with slack spans into substation. Static-cast square poles with T-arms for station post bus supports; square post top lighting pole.

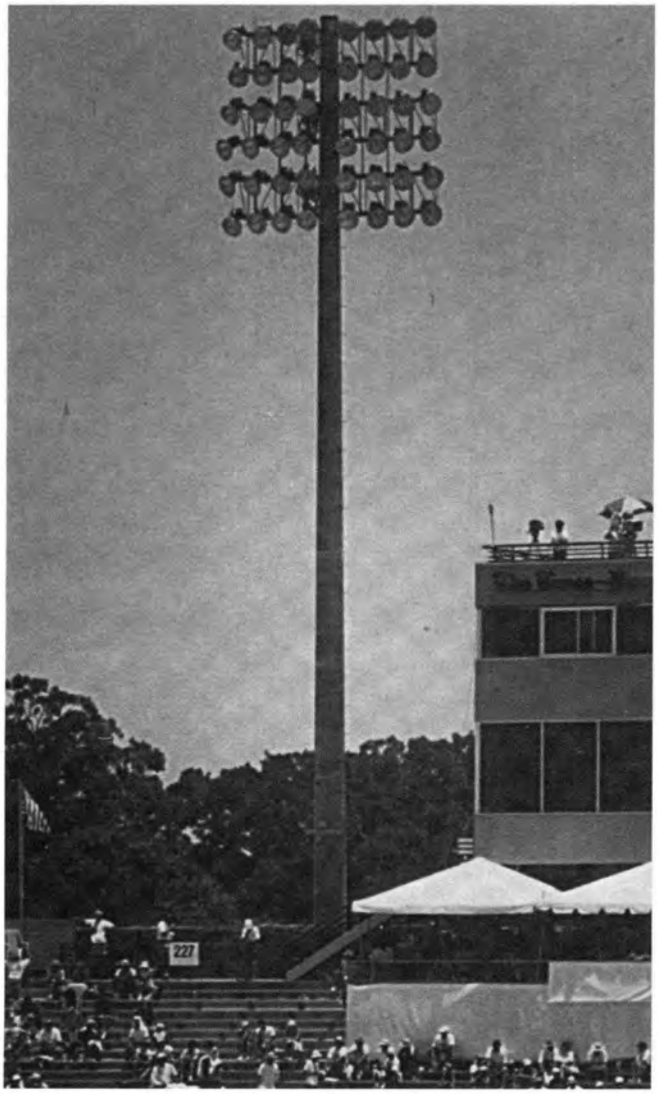


Fig. 1.2.1.c. Sports lighting pole.

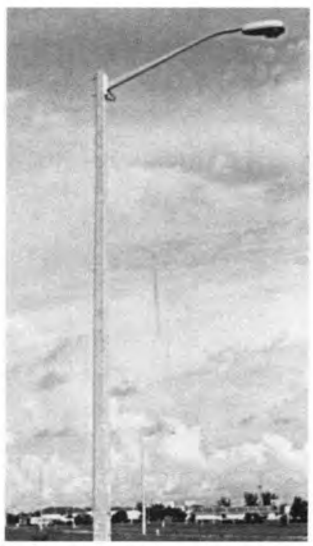


Fig. 1.2.1.d. Typical street lighting poles. Top Left: Round spun lighting pole with 8 ft (2.44 m) arm and cobra head lighting fixture. Bottom Left: Square, static cast lighting pole with arm and fixture. Top Right: Spun octagonal pole with terrazzo type finish and decorative fixtures.

Fig. 1.2.2. Guyed transmission structures. Top Right: Guyed angle structure with post insulators to offset jumper cable. Bottom Left: 90-degree corner; downguys in one direction with span guys to a stub pole in the other. Bottom Right: Fully guyed 90-degree corner pole.

loads are usually the result of wind forces on the structure, equipment, and/or conductors.

Eccentric vertical loads can also cause bending stresses. Eccentric vertical loads can be caused by equipment and conductor loads and by the vertical load of the structure in a deflected state. A discussion on loadings for prestressed concrete structures is included in Chapter 4.

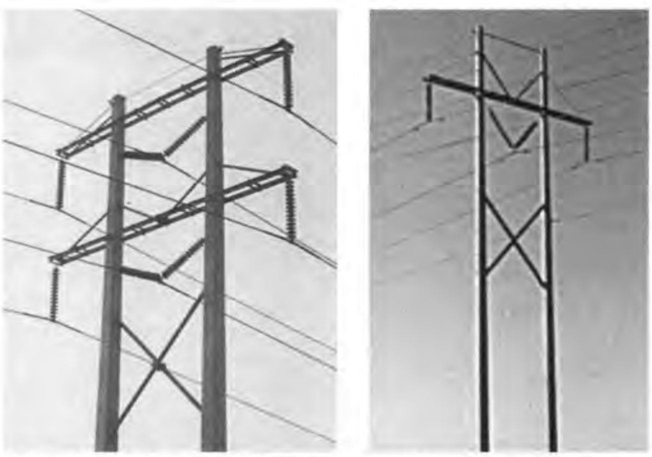


Fig. 1.2.3. H-frame with knee braces. Left: Double circuit H-frame. Right: Single circuit H-frame.

1.2.2 Guyed Structures

Another category of prestressed concrete structures is guyed structures. In order to reduce the bending stresses associated with cantilevered structures, guys can be installed to transmit the horizontal loads imposed on the structure to the ground. Although guys significantly reduce the bending stresses on the structure, the vertical component of the guy force adds to the vertical load on the pole. These vertical loads should be considered in the design. Guyed transmission structures are shown in Fig. 1.2.2.

It is important to recognize that guyed structures must be analyzed as a system. The guy wire size, orientation, pretension and maximum guy load should be specified to the structure designer.

1.2.3 Framed Structures

Framed structures are assembled from numerous members. They consist of two or more single cantilevered poles attached to one another by other members such that the poles and connecting members act as a system to resist applied loads. Framed structures are configured in a manner that allows for horizontal loads to be transmitted to the ground through the total stiffness of the structural support system. The stiffness is achieved by using bracing members with pinned connections, moment-carrying connections, or a combination of the two. H-frame structures are shown in Fig. 1.2.3.

Like guyed structures, it is important to recognize that framed structures must be analyzed as a system. The structure designer should determine the size, orientation and connection details of all members in the frame unless they are specified by the user.

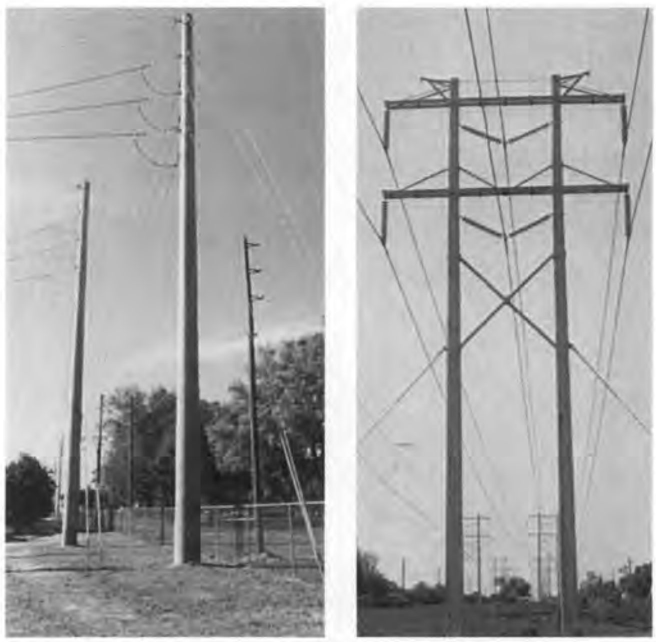


Fig. 1.2.4. H-frame structure with X-brace. Left: Single pole structures; guyed in one direction and free-standing in the other. Right: H-frame structure; primary loads resisted by rigid cross-arm braces and X-brace, storm guys added below X-brace to resist uplift during extreme winds.

1.2.4 Combined Structures

Structures may be designed for some combination of cantilevered, guyed and framed members. Two examples include an H-frame structure that is cantilevered above the cross-arm and an H-frame structure that is guyed at the bottom of the X-brace. Fig. 1.2.4 shows combined structures that resist wire tensions through guys and resist transverse horizontal loads through bending. An H-frame structure, however, is also a cantilevered structure in the longitudinal direction.

1.3 Applications

1.3.1 Transmission and Distribution Structures

Single pole cantilevered structures are typically used for tangent and small angle applications. Guyed pole structures are typically used for large angles, long spans, and dead-end applications. Horizontal loads due to conductor tensions may be too large for angle and dead-end structures to be cantilevered. In these cases, the most common method for supporting the load is to guy the structure at some or all of the conductor positions.

A common application for framed structures is the transmission line H-frame. The H-frame structure is often used for long cross-country transmission lines. Because of the additional load that can be carried by the frame action, utilities are able to use longer span lengths and/or larger size wire than for single pole structures. H-frame structures are usually not used in urban areas because they require a wide right-of-way.

Electric utilities use combinations of cantilevered, guyed and framed structures for several applications. Two of these applications include guyed H-frame structures and dead-end structures that are guyed in only one direction (see Fig. 1.2.4). Guyed H-frames are used extensively where uplift is a problem. Dead-end structures guyed only in one plane are used at locations where one side is slack and the other side is in full tension.

1.3.2 Substation Structures

Cantilevered pole structures are the most commonly used configurations in substation structures. Bus support structures for higher voltages typically use one single pole structure per phase. For lower voltages, structures can be installed for each phase or a "Tee" structure to carry all three phases. Single or multipole structures can be used to support disconnect switches, lightning arrestors, potential and current transformers, wave traps and other electrical equipment. Cantilevered and guyed structures are also used to support shield wires for lightning protection.

Fig. 1.3.4. Communication structures. Foreground: Free standing concrete antenna pole. Background: Guyed lattice type antenna structure.

1.3.3 Lighting Supports, Highway Sign and Traffic Signal Structures

Concrete poles of various shapes, textures and colors are commonly used with steel and aluminum luminaire arms to provide support for street lights. Similar poles use tenons or inserts to support lights for area and walkway lighting. Another common application of poles is to support fixtures for sports lighting by use of cross-arms or steel cages.

Cantilevered or guyed concrete poles are used as supports for highway signs and traffic signals. The highway signs and traffic signals are attached to span wires or arms that are supported by concrete poles.

1.3.4 Communication Structures

Concrete poles are used to support antennas for all classes of communication service including AM, CATV, FM, Microwave, TV and VHF (see Fig. 1.3.4).

CHAPTER 2 — INITIAL CONSIDERATIONS

This section details the information that users should include in their specifications to allow the designers of the structure to properly and efficiently accomplish their tasks.

2.1 Physical Characteristics

The basic pole structural configuration and location of all attachments should be made clear to the structure designer. However, the designer should be allowed as much latitude as possible to determine the design details of the structure.

2.2 Deflection

The user may specify to the designer deflection limitations under certain loading conditions. Limiting the deflection of a structure is sometimes necessary to ensure that clearances are maintained from the structure and its attachments to other objects, such as the edge of the right-of-way, buildings or bridges.

The appearance of the structure can also be affected by deflections. Sustained loads on structures may cause the pole to bow and be aesthetically unpleasant. To improve the appearance, a structure can be raked or the stiffness of the pole increased. Guys can also be installed to limit deflections. Stringent deflection limitations may increase the cost of the structure.

2.3 Decorative Applications

Many decorative colors, aggregates and textures may be specified for various architectural applications at additional cost. The colors are cast integrally throughout the pole during the manufacturing process. The pole surface may be polished to form a smooth terrazzo-like appearance or sandblasted to expose the aggregate and give a textured finish. Depending on the extent of the surface blasting, the section properties may have to be modified. Coatings may be applied to enhance aggregate color and to aid in the removal of graffiti.

2.4 Transportation and Erection

The design of the structure should consider loads caused by loading, unloading, hauling, assembly, erection and stringing. The limitations of the handling equipment and the job site access should also be considered.

Concrete poles can be designed to be lifted or erected with one-point picks or may require multiple-point picks. Designing for a one-point pick without cracking may not be economical unless warranted by special conditions. The manufacturer should clearly indicate the proper procedure for handling, transporting and erecting the product.

2.5 Attached Items

The user should inform the designer what accessories are to be mounted on the poles as well as the weight of those accessories so that the poles may be properly designed. Locations of bolt holes and inserts should also be provided. Holes or inserts within a hardware pattern, such as brackets, arms or X-braces, should be identified to the manufacturer.

2.6 Guying

The user should define as many known conditions as possible, such as right-of-way limitations; size, grade, and allowable load of guys; guy angle limits; quantity of guys; placement tolerances; and terrain considerations.

2.7 Climbing and Maintenance

One important concern for the user is the ability to climb the pole and access those areas of the pole where hardware is attached. The two most common climbing systems are step bolts and ladders.

Step bolts for climbing are normally staggered at approximately 15 in. (380 mm) intervals. Additional step bolts may be placed around the pole to provide a working level. Step bolts are installed in threaded inserts cast into the face of the pole.

Ladders are placed in clips bolted to the face of the pole using threaded inserts. In areas where maintenance is required, clips can be installed in multiple faces of the pole. A variety of ladder styles are available for use at maintenance locations. Manufacturing constraints may limit the location of inserts. The manufacturer should coordinate final placement of inserts with the user.

2.8 Grounding

The user should specify the grounding method. Grounding of concrete poles can be external or internal. For an external ground, threaded inserts can be embedded in the pole for clamping the ground to the pole's surface. Internal grounds can be embedded in the concrete or pulled through the center void of the pole with pig tails or grounding pads as required. Many users in areas of high levels of lightning occurrence or high ground resistance bond all hardware to the grounding system.

2.9 Load Testing

The user should specify whether a full-scale structure test is required. A test may be performed to verify the design concept, meet legal obligations, determine the level of reliability or to better understand structural, foundation or system behavior under certain loading conditions. The height and type of structure and all loading cases to be tested should be clearly identified.

Two types of testing are used to determine the flexural behavior and flexural capacity of poles under static loading conditions: pole testing and structure testing. Pole testing is used to verify the design and quality of production of the poles. Poles are generally tested in a horizontal position. The tests will check the cracking moment, ultimate moment and deflection of the poles. Structure testing is the simulation of the structure as it is to be used. The design loads are applied incrementally to check structural behavior.

2.10 Foundations

The pole design can be affected by foundation rotation, which causes secondary moments due to additional deflection. Therefore, the type of foundation to be used is an important design consideration. For more detailed information, see Chapter 9.

CHAPTER 3 — MATERIALS

3.1 Concrete

3.1.1 Design Compressive Strength

The minimum design 28-day concrete cylinder compressive strength f_c' is 5000 psi (33 MPa), with 6000 to 12,000 psi (40 to 80 MPa) strength being quite common.

3.1.2 Stress-Strain Curve

A typical stress-strain curve for concrete in compression is shown in Fig. 3.1.2. The elastic modulus of concrete can be defined as the secant modulus at $0.5f_c'$.

3.2 Prestressing Steel

Prestressed concrete poles are typically reinforced with either uncoated, stress relieved steel wire (ASTM A421) or uncoated low relaxation and stress relieved seven-wire strand (ASTM A416). The steel is placed inside the form and stressed to the required tension. A typical arrangement of pole reinforcement is shown in Appendix A.

Strand is also available with coatings such as epoxy (ASTM A882) and galvanizing to provide protection in extremely corrosive environments. However, the galvanizing process may result in the prestressing steel having lower breaking strengths and a slightly lower modulus of elasticity. Increased development length due to epoxy coating should be considered.

3.2.1 Prestressing Steel Characteristics

Mechanical properties of commonly used prestressing steel are given in Table 3.2.1. A typical load-elongation curve for a strand is shown in Fig. 3.2.1.

3.2.2 Allowable Stresses

The permissible stresses in prestressing steel according to ACI 318-95 are given in Table 3.2.2, where f_{pu} is the ultimate strength of the steel and f_{py} is the specified yield strength (ACI, 1995). For wire and strand, the yield strength is defined as the stress at which a total extension of 1 percent is attained.

3.3 Spiral Reinforcement

Spiral reinforcement enclosing the strands helps to resist radial stresses caused by the wedging effect of the strand at release. It can also control or minimize cracks due to torsion, shear, shrinkage or temperature-induced stresses.

The wedging effect from the release of the pretensioning forces causes tensile stresses at every cut-off strand location throughout the pole. Thus, along the length of transfer (about 50 times the strand diameter), strands produce radial pressure against the surrounding concrete, which could develop longitudinal cracks unless properly contained by adequate spiral reinforcement.

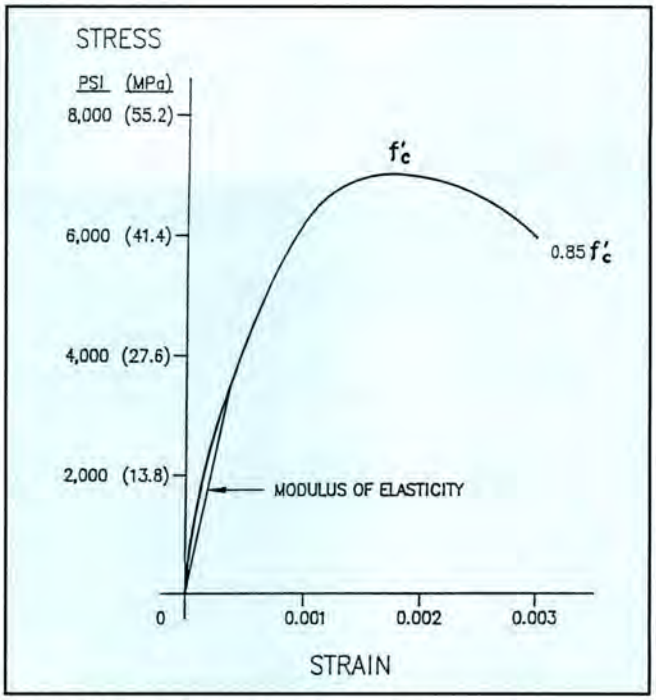


Fig. 3.1.2. Typical stress-strain curve for concrete in compression.

The spiral reinforcement generally conforms to ASTM A82 and its size should be in the range of No. 5 to 11 gauge wire, depending on the pole use and size. The minimum area of spirals should be computed as 0.1 percent of the concrete wall area in a unit length increment. More spiral reinforcement is required at the tip and butt segments of the pole to resist the radial stresses that occur at transfer of prestress. The minimum clear spacing of spiral is four-thirds of the maximum size of coarse aggregate and should not be less than 1 in. (25 mm). The maximum center-to-center spacing should not exceed 4 in. (100 mm), unless it is shown through tests that the performance of the pole is not impaired.

Poles subject to high shear forces, such as those with short embedment lengths, may require additional calculations of spiral requirements in the embedded section. Embedment length is also related to the minimum distance required to develop the ultimate strength of the prestressing wires.

3.4 Mild Steel Reinforcement

Mild steel reinforcing bars or dormant strand may be used in addition to the prestressed reinforcement to increase the ultimate moment capacity of the pole. The bar reinforcement is usually placed at the critical section(s) only and does not extend throughout the entire length of the pole. Because mild reinforcing steel will yield at strains much less than prestressed reinforcement, the designer should be aware that if the structure undergoes deflections large enough to yield the mild steel, it will no longer recover fully after release of load.

Table 3.2.1. Characteristics of prestressing steel.

Galvanized stress relieved strand

Nominal strand diameter mm	Nominal strand diameter in.	Grade MPa	Grade ksi	Minimum breaking load kN	Minimum breaking load lbs	Minimum load at 1 percent extension kN	Minimum load at 1 percent extension lbs	Nominal steel area* mm²	Nominal steel area* sq in.
9.53	3/8	1725	250	94.5	21,250	75.6	17,000	54.84	0.085
11.11	7/16	1725	250	127.7	28,700	102.1	22,950	74.19	0.115
12.70	1/2	1725	250	169.9	38,200	136.1	30,000	98.71	0.153
12.70	1/2	1860	270	183.7	41,300	156.1	35,100	98.71	0.153

* Steel area prior to galvanizing.

Uncoated stress relieved strand ASTM A416

Nominal strand diameter		Grade		Minimum breaking load		Minimum load at 1 percent extension				Nominal steel area	
						Normal relaxation		Low relaxation			
mm	in.	MPa	ksi	kN	lbs	kN	lbs	kN	lbs	mm²	sq in.
7.94	5/16	1725	250	64.5	14,000	54.7	12,300	58.1	13,050	37.42	0.058
7.94	5/16	1860	270	71.2	16,000	60.5	13,600	64.1	14,400	38.06	0.059
9.53	3/8	1860	270	102.3	23,000	87.0	19,550	92.1	20,700	54.84	0.085
11.11	7/16	1860	270	137.9	31,000	117.2	26,350	124.1	27,900	74.19	0.115
12.70	1/2	1860	270	183.7	41,300	156.1	35,100	165.3	37,170	98.71	0.153
15.24	0.6	1860	270	260.7	58,600	221.5	49,800	234.6	52,740	140.00	0.217

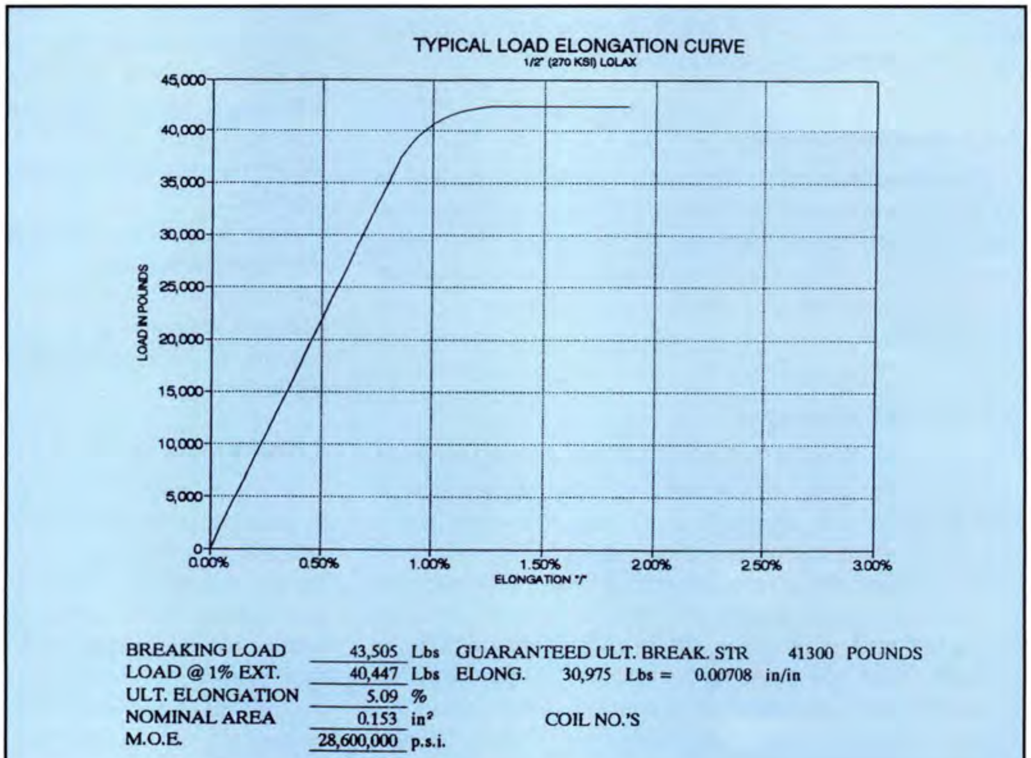
Uncoated stress relieved wire ASTM A421

Nominal wire diameter mm	Nominal wire diameter in.	Grade MPa	Grade ksi	Minimum breaking load kN	Minimum breaking load lbs	Minimum load at 1 percent extension kN	Minimum load at 1 percent extension lbs
5	0.196	1655	240	33.6	7550	28.6	6418
6.35	1/4	1655	240	52.4	11,780	44.5	10,013
7	0.276	1620	235	62.5	14,050	53.1	11,943

Table 3.2.2. Permissible stresses of prestressing steel.

1	Due to jacking force but not greater than $0.80f_{pu}$ or maximum value recommended by manufacturer of prestressing steel or anchorages.	$0.94f_{py}$
2	Immediately after prestress transfer but not greater than $0.74f_{pu}$.	$0.82f_{py}$
3	Post-tensioning steel at anchorages and couplers immediately after anchorage.	$0.70f_{pu}$

Fig. 3.2.1. Load-elongation curve for 1/2 in. (12.7 mm) diameter stress-relieved seven-wire strand Grade 270.



4.1 General

This chapter discusses the types of loadings that might be used for the design of each of the following structure applications:

- Transmission and distribution structures
- Substations
- Lighting supports, highway signs, and traffic signals
- Communication structures

In addition to the discussion of loadings for these applications, a general discussion of other loads is included. For consistency throughout this document and consistency with other ASCE Design Guides, design loads without multiplication by load factors will be referred to as "unfactored loads." Design loads multiplied by load factors will be referred to as "factored loads."

4.2 Transmission and Distribution Structures

Transmission and distribution lines are designed to withstand loading conditions that have been specified by the user and/or governmental agencies responsible for ensuring the safe, reliable and economic operation of the system.

The loading conditions typically considered to determine the required strength of the transmission and distribution structures are the ASCE Guidelines for Electrical Transmission Line Structural Loading (ASCE, 1991); the National Electrical Safety Code (NESC) loads; state and local safety code loads; local meteorological loads such as combinations of wind, ice and temperature conditions; longitudinal loads such as line terminations and broken conductor loads; and construction and maintenance loads.

For certain load cases, structure deflection may govern the design. Load factors are applied to the various loading cases as required by code or as determined to be appropriate by the utility or the designer. The "overload capacity factors" of the NESC (NESC, 1993) are one example of code load factors. Load factors for climatic, security and construction loads are suggested in the ASCE Loading Guide (ASCE, 1991). Other than load factors for code loads, there is no required standard for the various load cases and the load factors should be determined using engineering judgment or utility guidelines.

The NESC provides a set of minimum loads (heavy, medium, light and extreme wind), with specified overload capacity factors, for the various grades and types of construction. Most states have adopted the NESC; however, some states or local governments have written and adopted their own safety code to satisfy regional safety requirements (California, for example, has adopted General Order 95 in lieu of NESC). The Rural Electrification Administration (REA) has adopted the NESC but has modified some overload capacity factors and strength requirements.

Meteorological loads are associated with local climatic conditions that may occur during the life of the line. These loads are generally set by the utility or selected by the designer. Typical loads consist of wind, ice and temperatures, taken singly or in combination. Generally, a high (extreme) wind load and a combination of wind and ice load are both used for the design. The ASCE Guidelines for Electrical Transmission Line Structural Loading (ASCE, 1991) may be referred to for the development of meteorological loads as well as other typical loads.

Longitudinal loads on a structure fall into three major categories: (1) permanent loads due to line termination or change in ruling span; (2) temporary loads due to unbalanced ice and wind conditions; and (3) loads due to a broken or slack wire.

Longitudinal loads resulting from a difference in wire tensions from one side of the structure to the other are relatively easy to determine for dead-end structures. Suspension structures are more difficult to analyze because of the displacement of the suspension insulator, which acts to balance wire tensions with the longitudinal load experienced by the structure. A longitudinal load may be selected that approximates the loading conditions of the suspension structure.

Unbalanced longitudinal loads may induce torsion in pole type structures and this torsion must be considered in the strength evaluation of the structure design. Broken wire loads may also be considered.

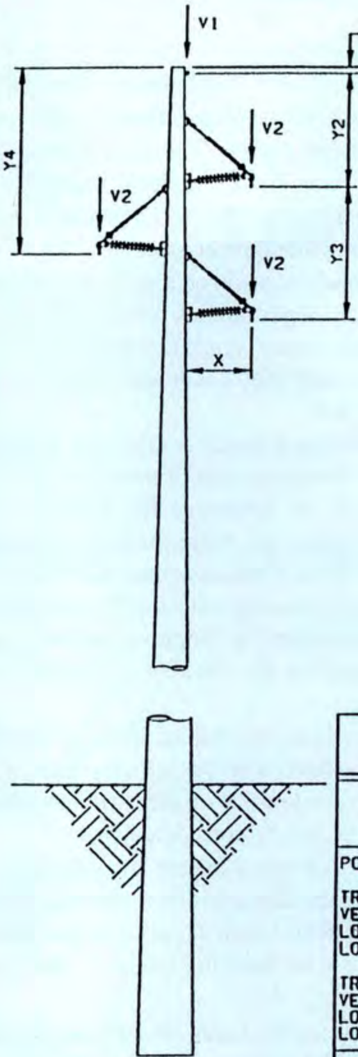
Construction and maintenance loads should be considered to ensure the safe assembly, erection, loading and operation of the system. Loads commonly considered as construction loads are wire stringing loads, snub-off loads and clipping-in loads.

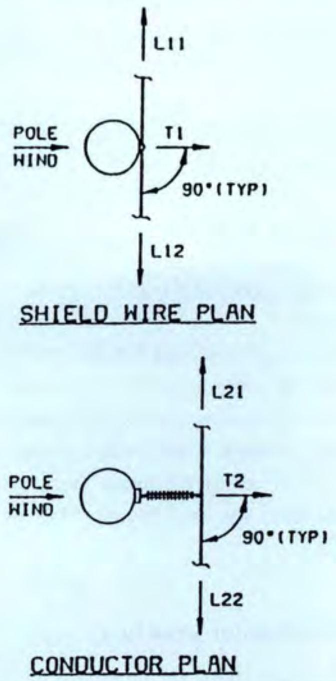
Wire stringing loads are unbalanced wire tensions when the running board or wire may become caught in the stringing block and get "hung-up." Snub-off loads are the temporary dead-ending of the conductors and shield wires on one longitudinal side of the structure to the ground during stringing operations. Clipping-in loads are the loads for lifting the conductor from the block after the conductor has been brought to the initial sag position.

Maintenance loads are worker and equipment loads associated with procedures such as changing insulator strings and hardware. The construction and maintenance loads usually occur with a nominal wind and at a temperature likely to occur during those operations.

Various combinations of loads are considered to predict structure deflections. These deflections are used to determine clearances, right-of-way width, raking of the pole and other special requirements.

It is recommended that loading conditions be expressed as load trees, using an orthogonal coordinate system as shown in Fig. 4.2. Conductor and shield wire loads should be shown at the conductor and shield attachment points. The weight of all attachments, such as hardware and insulators, should be included in these loads. Wind pressure on the structure itself should also be specified. All loads should be shown as factored loads. The load factors are usually equal to one for the load cases used to check cracking.





LOAD TABLE

LOAD DESCRIPTION	LOAD CASE 1	2	3	4	5
POLE WIND	0.036	0.023	0.026	0.000	0.009
TRANSVERSE LOAD - T1	0.54	0.33	0.33	0.08	0.15
VERTICAL LOAD - V1	0.18	0.25	0.17	0.50	0.17
LONGITUDINAL LOAD - L11	2.75	2.60	1.03	1.24	2.36
LONGITUDINAL LOAD - L12	2.75	2.60	1.03	1.44	2.36
TRANSVERSE LOAD - T2	1.42	0.86	0.86	0.04	0.38
VERTICAL LOAD - V2	0.51	0.69	0.46	2.80	0.46
LONGITUDINAL LOAD - L21	6.05	5.36	4.25	2.02	4.87
LONGITUDINAL LOAD - L22	6.05	5.36	4.25	2.02	4.87

CASE DESCRIPTION

CASE 1 - ASCE EXTREME WIND, VERTICAL OLF = 1.1
CASE 2 - NESC LIGHT, VERTICAL OLF = 1.50
CASE 3 - NESC EXTREME WIND, VERTICAL OLF = 1.0
CASE 4 - CONSTRUCTION LOAD, VERTICAL OLF = 1.0
CASE 5 - SPECIFIED CRACKING LOAD, VERTICAL OLF = 1.0

NOTES

1. WIRE LOADS ARE IN KIPS. POLE WIND IS IN KIPS PER SQUARE FOOT.
2. LOADS SHOWN INCLUDE OVERLOAD FACTORS.
3. EACH LOADING CASE SHALL BE CONSIDERED INDEPENDENTLY.

Fig. 4.2. Typical load tree.

4.3 Substation Structures

Electrical substation structures are generally designed for meteorological loads, short circuit forces, seismic loads, equipment loads and construction/maintenance loads. The design of substation structures is frequently controlled by deflection criteria necessary to ensure proper performance of electrical equipment and to satisfy aesthetic requirements.

When designing for high (extreme) winds, there are several procedures for computing the pressure due to wind. Two of these procedures are outlined in ASCE 7-88, "ASCE Minimum Design Loads for Building and Other Structures," and the National Electrical Manufacturers' Association (NEMA) Publication SG 6, Part 36, titled "Outdoor Substations."

Determination of ice loading is usually based on the prior experience of the location of the particular system. In the absence of such particular criteria, ice loads should follow the procedures outlined in ASCE Manual 74 (1991).

The forces caused by a short circuit in a substation may be significant and should be considered in substation structure design. The repulsive force between two phases during a short circuit generally creates a moment in the structure at the points of insulator attachment and within the structure if a single support is used for each phase. Short circuit forces can be calculated using the IEEE "Guide for Design of Substation Rigid-Bus Structures," ANSI/IEEE Standard 605 (1987). The calculated short circuit force multiplied by the insulator length provides the moment applied to the structure.

Another approach used when the insulator selection is predetermined is the calculation of structure loadings by applying a moment larger than would be produced when the rated insulator cantilever breaking strength is applied at the end of the insulator. This procedure should result in breakage of the insulator before damage occurs to the structure.

Equipment loads consist of dead loads and operating loads. Dead loads include the weight of the equipment and associated hardware. Operating loads include dynamic forces due to moving parts of the equipment. These loads are given by the equipment manufacturer.

Construction and maintenance loads should be considered to ensure the safe assembly, erection and operation of the system. Generally, this entails accounting for workmen and tools in addition to the system.

In addition to stress requirements, structures must be sufficiently rigid so that deflections of members will not exceed the limits specified by either NEMA or the equipment manufacturer, whichever is more stringent. The NEMA SG 6, Part 36, outlines deflection limits based on structure class.

It is recommended that loading conditions be expressed as load trees, using an orthogonal coordinate system. Bus, equipment and insulator loads should be shown at the appropriate attachment points. Loads shown should be the factored loads.

4.4 Lighting Supports, Highway Sign and Traffic Signal Structures

Lighting supports, highway sign and traffic signal structures include many types of structures. Lighting support structures include common light standards, post top standards, high mast light structures and stadium flood lighting poles. Highway traffic sign supports and marker support structures include overhead and roadside sign supports. Traffic signal support structures include structures for post top mounted traffic control signals, structures with cantilever arms, bridge mounted traffic control signals and span wire mounted traffic control signal support structures.

Because all of these support structures are included in the AASHTO "Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals" (AASHTO, 1985), it is recommended that the loading criteria of this standard be used. The standard discusses the loads to be applied to each of these different support structures including dead load, live load, ice load, wind load and combination or group loads. The term effective projected area (EPA) is used to designate the effective surface area of lighting fixtures. When using EPA rated fixtures, the use of additional shape factors is not required.

4.5 Communication Structures

The Electronic Industries Associate Standard EIA/TIA/222-E "Structural Standards for Steel Antenna Towers and Antenna Supporting Structures" (1994) is recommended for the determination of loads, tolerances, foundations, anchors, guys and allowable twist and sway values. When using the working loads from this standard, a minimum load factor of 1.25 is recommended for concrete structures. This factor is the same as that used in the AASHTO "Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals" (1994).

4.6 Other Loads

Structures located in areas subject to earthquakes should be analyzed for the effects of seismic forces. By studying ASCE 7-95, the applicable building code and other appropriate standards, the designer can determine the earthquake zone in which a particular structure is located. For very important structures, such as unguyed (dead-end or heavy angle) structures or structures having stringent safety requirements, special analyses may be required.

Handling loads should also be considered. These loads are generated during transportation and erection of the structures. The lifting of the entire structure from the horizontal position is typically the controlling handling condition. This load is caused by the weight of the structure itself plus the weight of any items that may be attached to the structure.

To allow for shock loads that may occur while the structure is being lifted, an impact factor of 1.5 should be applied to the dead weight of the structure and attached accessories. Also, the reduced strength f'_{ci} should be considered for stripping and in-plant handling. The manufacturer should indicate the locations of single- or multiple-point picks, unless otherwise specified by the user. See Section 8.1 on Handling for further information.

CHAPTER 5 — DESIGN

5.1 General

Prestressed concrete poles may be analyzed using classical reinforced concrete theory. These poles exhibit both linear and nonlinear behavior. Prior to exceeding the tensile strength of the concrete (below the cracking moment), the pole has a relatively constant modulus of elasticity and deflects in a linear manner. Above the cracking moment, the pole behaves mostly nonlinearly because of the altered properties of the cracked section.

During this state, greater deflection will occur than that of an uncracked section for a similar increase in load. These deflections cause secondary moments in the structure due to the offset axial loading of the pole's center of gravity coupled with the weight of the conductors and insulators (the so-called P-Δ effect).

It is important that the effects of nonlinearity be considered in the structural analysis, not only because of the secondary moments induced but also because deflection can become significant when an extreme wind loading causes a structure, in combination with the swing of the conductors, to approach the allowable clearance to the right-of-way edge (a condition known as "blow out").

5.2 Design Method

The design of prestressed concrete poles is a relatively complex process that involves consideration of various loading conditions, time-dependent and nonlinear material behavior, ultimate strength and serviceability.

Prestressed concrete poles should be designed primarily by the ultimate strength method. Service loading conditions, such as first circumferential crack, reopening of cracks and deflection, should be investigated with unfactored loads.

The cross-sectional area of a pole is determined using an iterative design process. Starting with a specified pole height and a load tree, the designer assumes a trial pole cross section. Then, for each section that is incrementally investigated, specified limit states must be satisfied. If not satisfied, the trial-and-error process is repeated until a solution is found.

The four distinct design conditions that may be considered in the design of a prestressed concrete pole are: (1) ultimate flexural strength; (2) cracking strength; (3) zero tension strength; and (4) deflection.

5.2.1 Ultimate Strength

The ultimate flexural strength of a pole is the moment at which the pole will fail, usually by crushing of the concrete. The pole should be designed to have the ultimate strength at all sections of the pole exceed the required strength calculated from the appropriate factored loads applied to the structure. Factored loads are specified in codes (NESC, 1993), guidelines (ASCE, 1991, Loading Guide) or other documents.

5.2.2 Cracking Strength

The cracking strength of a pole is the moment at which the first circumferential crack will occur. Under this condition, the moment in the pole causes the tensile strength of the concrete to be exceeded on the tension face of the pole. The tensile strength is a function of the concrete modulus of rupture. These cracks will close upon release of the load. The pole should be designed to have the cracking strength exceed the moments calculated from the service loads. A typical service load is NESC District loading without a load factor.

5.2.3 Zero Tension Strength

The zero tension strength is the moment at which a crack that was previously created by exceeding the cracking moment strength will open again. Under this condition, an applied moment will not cause any tensile stress in the concrete. This strength will always be less than the cracking moment strength. Structures that are subjected to a permanent lateral load, such as unguyed dead-end or angle structures, or structures controlled by deflection should be designed to have the zero tension strength exceed the moments calculated from service loads or sustained loads. This would avoid having a crack remain open for the life of this structure type. Avoiding open cracks is important in extremely corrosive environments, such as placement in sea water or proximity of industrial contaminants, in order to protect the steel reinforcement.

5.2.4 Deflection

The maximum allowable deflection of a structure, as specified by the user, may control the design of the structure. The user should specify to the pole designer the loading conditions that are to be considered in determining the pole deflection. The pole stiffness (EI) should be sized so that the pole deflection calculated from the specified loading conditions does not exceed the maximum allowable deflection.

5.2.5 Shear and Torsion

See Section 5.7.

5.3 Prestress Losses

The magnitude of the prestressing force in the pole is not constant but decreases with time. This decrease in the prestressing force is referred to as the prestress loss. Some prestress losses are instantaneous and some are time-dependent. Instantaneous losses are due to elastic shortening, anchorage slippage and friction, in the case of post-tensioning. Time-dependent losses are mainly due to shrinkage and creep of concrete and steel relaxation.

A detailed analysis of losses is not necessary except for unusual situations where deflections could become critical. Lump sum estimates of losses are commonly used. Depending on the materials used, 15 to 25 percent for total losses are common design assumptions. A good source for information on the calculation of prestressing losses may be found in the Fourth Edition of the PCI Design Handbook (1992).

5.4 Principles and Assumptions of Ultimate Moment Capacity

The ultimate moment capacity of a pole at any given cross section is a function of the strains in the prestressing steel and concrete. The factored design moment should not exceed the ultimate moment capacity.

The following assumptions are made in computing the ultimate moment capacity of poles:

- Plane sections remain plane.
- The steel and concrete are adequately bonded.
- The steel and concrete are considered in the elastic and plastic ranges.
- The concrete compressive stress at failure is $0.85f_c'$.
- The tensile concrete strength is neglected in flexural computations.
- The ultimate concrete strain is 0.003.

While the first two assumptions become somewhat less valid after the section has cracked, the overall behavior of the member can still be predicted adequately.

5.5 Determination of Ultimate Moment Capacity

5.5.1 Equilibrium of Section

Based on the above assumptions and the provisions in the ACI 318 Building Code (1995), the assumed rectangular compressive stress distribution in the concrete is used herein for simplification and is represented by a statically equivalent concentrated force, defined by the cylinder compressive strength f_c', the parameter β_1, and the quantity Kc, which locates the centroid of the stress block (see Fig. 5.5.1).

Equilibrium of the section requires equal forces in the prestressing steel and concrete. The equation of equilibrium is (without axial loads):

$$C_c = T_s$$

where C_c is the concrete compression and T_s is the steel tension.

The compression in the concrete is then computed from:

$$C_c = 0.85 f_c' A_a$$

where A_a is the area of the concrete in compression as defined by a rectangular stress block of depth $\beta_1 c$. The parameter β_1 is defined as 0.85 for a concrete strength of 4000 psi (27.5 MPa) and less, and is reduced by 0.05 for each 1000 psi (7 MPa) in excess of 4000 psi (27.5 MPa) with a minimum value of 0.65. The computation of the compressive concrete area A_a for a round hollow pole and the location of the centroid of compression are derived in Appendix B.

The steel tension is expressed as:

$$T_s = \sum_{i=1}^{n} A_{psi} f_{sei}$$

where A_{psi} and f_{sei} are the area and stress of the ith strand, respectively. The determination of f_{sei} is given in Appendix C. Trial and error iteration of the location of the neutral axis c is used to solve for the depth of the stress block, such that equilibrium between tension and compression is satisfied.

5.5.2 Ultimate Moment Capacity Equation

The ultimate moment capacity of a pole section is given as the sum of the moments of tensile and compressive forces with respect to the neutral axis:

$$\phi M_n = \sum_{i=1}^{n} e_i A_{psi} f_{sei} + c C_c (1 - K)$$

where $e_1 = d_1 - c$ and ϕ is the capacity reduction factor (0.90 for flexure).

Note that A_{psi}, f_{sei} and c are previously defined, Kc is the position of the centroid of the reduced compressive concrete area (pressure line), d_i is the distance of the ith strand from the extreme compressive fiber, and e_i is the distance of the ith strand to the neutral axis.

The quantity $e_i A_{psi} f_{sei}$ is positive when the ith strand is located below the neutral axis (tension zone) and negative when it is located above (compression zone). In the case of braced H-frames and guyed structures, the formula for the ultimate moment capacity should incorporate the effect of the applied axial loads.

5.6 Cracking Moment and Zero Tension Moment

Cracking starts when the tensile stress in the extreme fiber of the concrete reaches its modulus of rupture. The cracking moment can be computed by elastic theory to predict the behavior of poles.

For a symmetrically reinforced prestressed concrete pole section, a uniform stress P/A_g acts on the gross sectional area A_g due to the effective prestress P. Because of the external moment M, the section area is subject to the extreme tensile stress My_t/I_g, where y_t is the distance from the centroidal axis to the extreme tensile fiber and I_g is the gross moment of inertia of the section. The cracking moment may be calculated using the following relationship:

$$M_{cr} = \frac{f_r I_g}{y_t} + \frac{P I_g}{A_g y_t}$$

where $f_r I_g / y_t$ is the resisting moment due to the modulus of rupture of concrete (f_r) and $P I_g / A_g y_t$ is the moment due to the direct compression of the prestress.

In ACI 318 (1995), the modulus of rupture is given as $7.5\sqrt{f_c'}$ where f_c' is the concrete compressive strength (in psi).

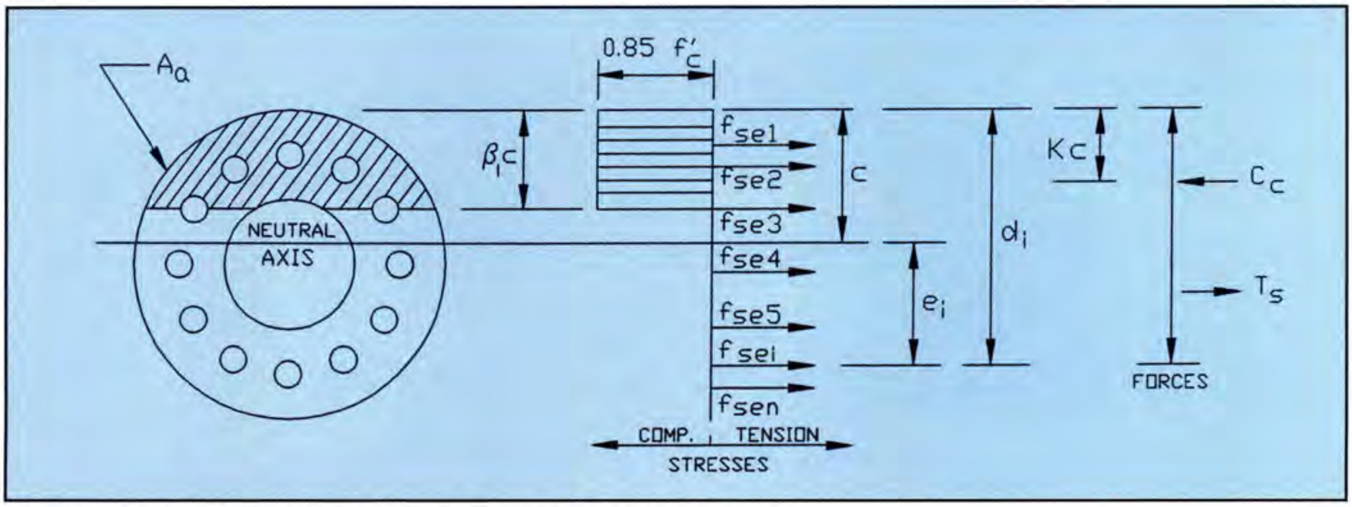


Fig. 5.5.1. Concrete stress area and assumed stress distribution in pole section.

The zero tension moment M_o may be calculated from the relationship:

$$M_o = \frac{PI_g}{A_g y_t}$$

The stress distribution in a pole section at cracking and zero tension is shown in Fig. 5.6.

5.7 Shear and Torsion

5.7.1 Shear

The design of concrete pole cross sections subject to shear shall be based on:

$$V_u \le \phi V_n$$

where V_u is the factored shear force at the section considered, ϕ is taken as 0.85, and V_n is the nominal shear strength computed by:

$$V_n = V_c + V_s$$

where V_c is the nominal shear strength provided by the concrete and V_s is the nominal shear strength provided by the shear reinforcement.

For square or rectangular prestressed concrete members with an effective prestress force not less than 40 percent of the tensile strength of the flexural reinforcement, V_c may be computed as:

$$V_c = \left[0.6\sqrt{f_c'} + 700\left(\frac{V_u d}{M_u}\right)\right] b_w d$$

But, V_c need not be less than $2\sqrt{f_c'}\, b_w d$ nor shall it be greater than $5\sqrt{f_c'}\, b_w d$.

The quantity $V_u d / M_u$ shall not be greater than 1.0, where M_u is the factored moment occurring simultaneously with V_u at the section considered. The variable d shall be the distance from the extreme compression fiber to the centroid of the prestressing reinforcement and b_w shall be the width of the web.

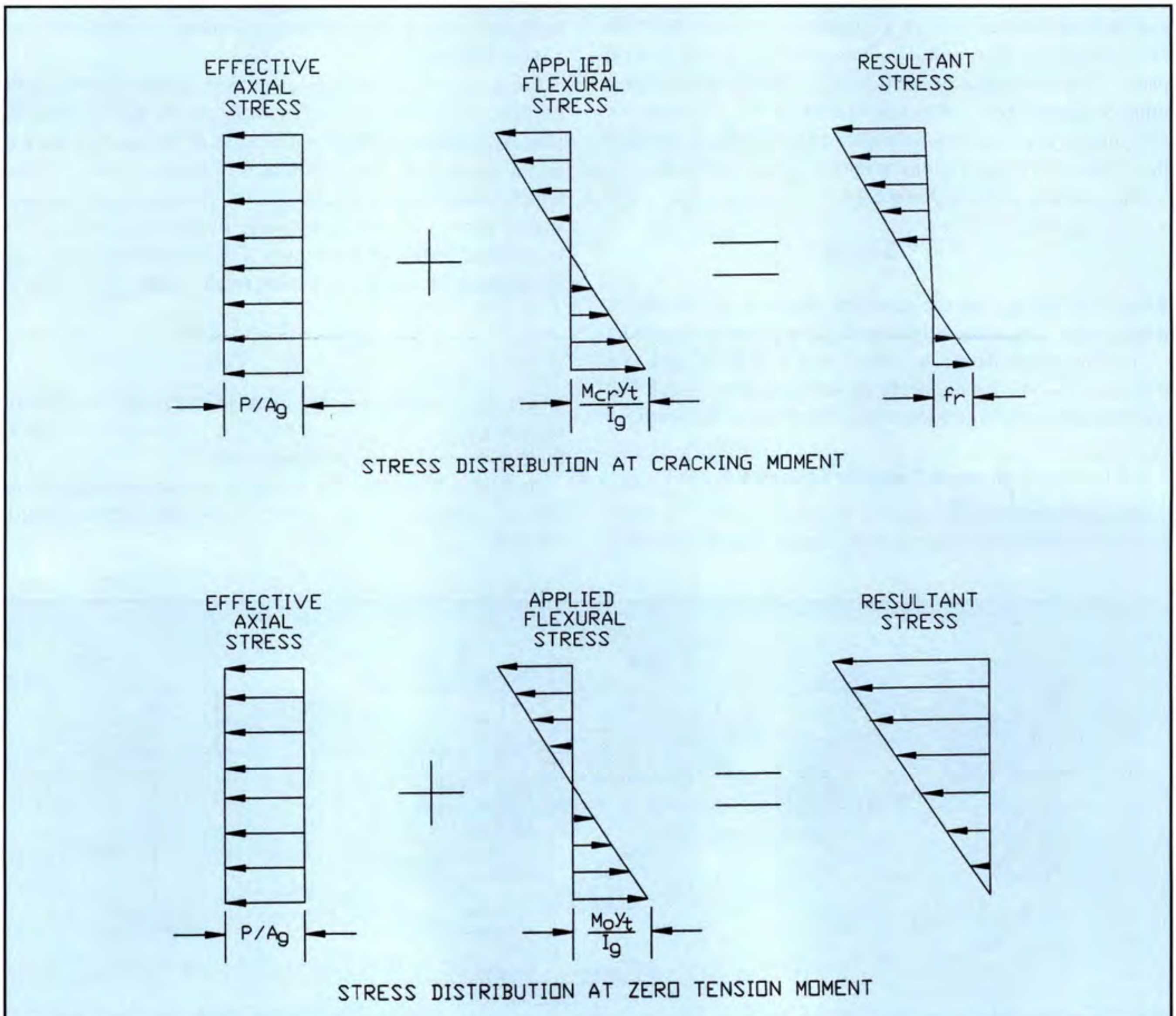


Fig. 5.6. Stress distribution in pole section at cracking and zero tension.

For circular prestressed concrete members:

$$V_c = \frac{\sqrt{F_t^2 + F_t f_{pc}}}{\dfrac{Q}{2It}}$$

where

F_t = tensile strength of concrete taken as $4\sqrt{f_c'}$

f_{pc} = effective compressive stress in concrete due to prestress

Q = moment of area above centroid

I = moment of inertia of cross section

t = wall thickness

For the shear force V_s contributed by the steel:

$$V_s = \frac{A_v f_y d}{s}$$

where A_v is the area of the shear reinforcement within a distance s, f_y is the yield strength of the steel, and d is the distance from the compression force to the centroid of the prestressing steel, or 0.8 times the outside diameter of the section, whichever is greater.

5.7.2 Torsion

The design of concrete pole cross sections subjected to torsion shall be based on:

$$T_u \leq \phi T_c$$

where T_u is the factored torsional force at the section considered, ϕ is taken as 0.85, and T_c is the torsional resistance of the prestressed concrete member.

For square or rectangular cross sections (Lin and Burns, 1981):

$$T_c = 6\sqrt{f_c'}\sqrt{1 + \frac{10 f_{pc}}{f_c'}}\sum \eta x^2 y$$

where

$$\eta = \frac{0.35}{0.75 + \dfrac{b}{d}}$$

and x is the shorter overall dimension of the rectangular part of the cross section, y is the longer overall dimension of the rectangular part of the cross section, and b is the width of the compression face of the member.

For circular cross sections:

$$T_c = \frac{J}{r_o}\sqrt{F_t^2 + F_t f_{pc}}$$

where J is the polar moment of inertia and r_o is the outside radius of the section.

For members subject to simultaneous flexural shear and torsion, the following interaction equation may be used to represent the strength of the member:

$$\left(\frac{V_u}{0.85 V_n}\right)^2 + \left(\frac{T_u}{0.85 T_c}\right)^2 = 1.0$$

5.8 Critical Buckling Loads

Although buckling of a concrete pole is unlikely under normal circumstances, in some cases of guyed structures it may be critical. A useful reference on the subject is the ASCE "Guide for the Design of Guyed Transmission Structures" (1997).

The best estimate of buckling loads for non-prismatic members can be obtained using numerical methods to solve the differential equations obtained from classical elastic stability theory or using a nonlinear finite element formulation. Such numerical or finite element techniques are not practical without computers. These methods are described in advanced analysis textbooks. Today, finite element software programs are available at a modest cost.

For hand calculations, simplified techniques for determining buckling loads are available that give conservative results for most cases. For poles with constant cross sections or uniform taper, the critical buckling load can be approximated using the classical Euler buckling equation with appropriate effective buckling lengths. Using this approach, the critical buckling load may be determined by:

$$P_{cr} = \frac{\pi^2 EI}{(kL)^2}$$

The most difficult aspect of applying this equation is determining an appropriate value for EI. In place of a more precise calculation, EI for computing the buckling load may be taken as:

$$EI = \frac{E_c I_g}{2.5}$$

For poles of uniform cross section, I_g is the gross moment of inertia of the concrete. For uniformly tapered poles, I_g may be conservatively taken as the gross moment of inertia at a distance of one-third L from the smaller end of the unbraced length.

For cantilevered poles, the buckling length L to be evaluated should be from the centroid of the applied external loads to a point one-third of the setting depth below the ground line. The pole should be assumed fixed at the lower point and free at the upper end, giving a theoretical effective length factor $k = 2.0$.

For poles guyed in both directions, the buckling length L to be evaluated should be the distance from the bottom guy attachment to a point one-third the setting depth below the ground line. The pole should be considered fixed at the lower point and pinned at the upper end, giving a theoretical effective length factor $k = 0.7$. In practice, however, a value of $k = 0.8$ is preferable. Single poles guyed only in one direction should be treated as cantilevers for buckling purposes because they are free to deflect in the unguyed plane.

For H-frames, two modes of buckling should be checked. First, buckling in the plane of the cross brace should be checked using a length from the bottom cross brace attachment point to a point one-third of the setting depth below the ground line. The pole should be considered fixed at the lower point and pinned at the upper, giving a theoretical effective length factor $k = 0.7$ (recommended value $k = 0.8$). In addition, buckling in the plane perpendicular to the cross brace should be checked. This mode of buckling should be treated the same as the cantilever.

5.9 Deflections

Structure deflections should always be checked. For loads less than those causing the first crack, elastic deflections can be determined using the classical structural analysis methods. For loads in excess of cracking, an inelastic pole design method should be utilized. For the sake of appearance, excessive deflection under sustained loads should be avoided.

Additional structural deflection can occur due to concrete creep. This is the plastic deformation of the concrete due to application of loads over an extended time period. This could result in increased deflections for poles used as strain poles, self supporting dead-ends or guyed structures. For most pole applications, creep is not a major design consideration. However, it can be of significance for nonuniform stress distribution resulting from the combined effect of sustained load and prestress. Refer to ACI 318-95 for design procedures involving creep.

5.9.1 Determination of Elastic Deflection

For loading conditions that do not exceed the cracking capacity of the pole, an elastic method may be used. This could include virtual work, the conjugate beam method, slope deflection, or a finite element computer analysis.

5.9.2 Determination of Inelastic Deflection

Up to the point of cracking, the deflection may be computed using elastic methods previously described. After cracking, the modulus of elasticity E becomes both stress and time dependent and the moment of inertia I becomes crack dependent. Because the product EI varies with stress, time and pole geometry, the process for computing inelastic deflections is too complicated for hand calculations and, therefore, lends itself to iterative computer computations.

The inelastic deflection can be approximated using reduced values of the elastic product EI. These values may range from E_cI_g at a level of moment at cracking to $E_cI_g/3$ as the member approaches ultimate strength.

5.10 Joints and Connections

5.10.1 Connections

Connections between poles and attachments should be designed such that the allowable stresses of the connecting part and the concrete pole are not exceeded and excessive deformation or rotation is not induced. Hardware may be attached using through holes, bands or inserts, depending on the type and magnitude of load.

Factors to be considered in connection design include the load transfer mechanism, load factors, ductility, durability, required tolerances, aesthetics and economics.

5.10.2 Bolted Connections

Most hardware is bolted to concrete poles with galvanized through bolts. Good practice dictates that the bolts do not overload the concrete and that they be properly tightened. Bolts such as ANSI C135.1 or ASTM A307 are commonly used. Designing for use of lower strength bolts helps to ensure that the bolt loads do not exceed the allowable concrete bearing stress. Because the low strength bolts are readily available, those which require replacement will be replaced with bolts of the correct strength. Sleeving of holes may be necessary as a means of reducing concrete bearing stress, particularly when higher strength bolts are used.

To spread the concentrated loads under the head of the bolt and under the nut, a square curved washer or other similar plate should be placed between the head or nut and the pole. For A307 bolts over 1 in. (25.4 mm) in diameter or A325 bolts over $^3/_4$ in. (19.05 mm) in diameter, use either two $^1/_4$ in. (6.4 mm) thick washers or a single $^3/_8$ in. (9.5 mm) washer. Use of cast washers is not recommended. The turn of the nut method is applicable only to high strength bolts (A325 bolts). When A325 bolts are used, they should not be pretensioned to avoid overloading the hollow section.

For shear connections in which the bolt will bear against the side of the through hole, the maximum bolt bearing load will be determined by multiplying the diameter of the bolt times the effective wall thickness times the bearing strength of the concrete. In the absence of confirming tests, it is assumed that the bolt-to-concrete interface carries all of the load and none of it is carried through friction. The maximum effective wall thickness for calculating the bearing load is the least of 3 in. (76 mm), four bolt diameters, or the actual wall thickness.

5.10.3 Climbing Attachments

It is recommended that every individual part of the climbing system where a lineman could conceivably place his foot should be designed to withstand a static load of 750 lbs (3337.5 N) without permanent deformation and a load of 500 lbs (2225 N) dropped 18 in. (457.2 mm) without breaking, or the most recent Occupational Safety and Health Administration (OSHA) recommendations for any other requirements.

5.10.4 Inserts

Inserts should be made of materials that will not deteriorate in the environment in which they are placed. Care should be taken to ensure that the materials in the concrete, the insert and the bolt do not react unfavorably with each other.

The anchorage of the inserts in the concrete should be such that they do not pull out under the design load. Preferably, they are designed and anchored in such a manner that the bolts will fail first.

Consult the appropriate ACI and PCI design guides for proper insert design loadings. It is necessary to ensure that bolts do not bottom out in the insert. This may require coordination between the user and/or one or more suppliers. Inserts can be installed at various locations without reducing the strength of the pole; however, in some cases the location and quantity of strands may be affected.

5.11 Splicing

Prestressed concrete poles can be spliced with several different types of connections to meet production, handling and transportation requirements, or to attain additional lengths. Four splices are considered here and details of these splices are shown in Appendix D.

5.11.1 Slip Joint Splice

This splice consists of a steel collar with the same taper as the pole. The upper part is simply slid over the top of the lower part.

5.11.2 Flange Plate Splice

This splice consists of two flat steel plates that are held in place by the combination of strands and wedges. The two flanges are bolted together similar to a pipe connection.

5.11.3 Bolted Splice

This splice consists of bolts embedded into the lower section of the pole, which is topped with a steel plate to which the prestressing strands are attached. The upper section has a steel plate to which the strands are attached and blockouts in the embedded side of the plate to act as voids for the bolts.

5.11.4 Welded Splice

In this splice, steel plates are prestressed to the ends of the pole sections, which are then welded together in the field during erection.

5.12 Additional Design Considerations

5.12.1 Field Drilling

When it is necessary to field drill holes in concrete poles, a rotary hammer drill or core drill should be used. Care should be taken not to cut the prestressing strand. If the strands must be cut, the pole should be checked for structural integrity.

5.12.2 Prestressing Steel Spacing

ACI 318 (1995) recommends a minimum clear distance between prestressing steel strands to be either four-thirds times the maximum aggregate size or three times the strand diameter, whichever is larger. In the event that this condition is not met at the pole tip, a closer spacing would be permitted provided the placement of concrete can be accomplished satisfactorily, adequate stress transfer can take place, and appropriate confining reinforcement is added.

5.13 Wood Pole Equivalency

Many utilities have based their standard transmission and distribution designs on specific wood pole lengths and strengths. When a new design is to incorporate concrete poles, the designer may either equate concrete poles to wood poles or design a concrete pole based on specific conductor loadings.

If the designer decides to simply equate concrete poles to wood pole classes, he must specify to the concrete supplier how this equivalence is to be defined. NESC overload factors (NESC, 1997) include allowances for many characteristics of wood poles including initial strength assumptions, rates of decay over pole life, and the inherent variability of the wood pole. This section describes common methods of "equating" concrete to wood.

The American National Standard Institute publication "Specifications and Dimensions for Wood Poles," ANSI 05.1, Appendix B (1992) defines wood pole strength by a specified horizontal load applied 2 ft (0.61 m) from the top of the pole. The value of the horizontal load varies with the class of wood pole and is intended to cause equal stress at the ground line for the various classes of wood poles.

5.13.1 Ultimate to Ultimate Strength Comparison

Under this comparison, an equivalent concrete pole has the same ground line moment capacity as the 5 percent exclusion limit strength of a new wood pole (per ANSI 05.1) of a given length and class.

5.13.2 NESC Factored Loads Comparison

For the District Loads associated with Rule 250B, the NESC has traditionally recognized a difference between wood and concrete poles by specifying transverse wind load factors of 4 and 2.5, respectively. Based on this difference in load factors, the ultimate factored transverse load that a concrete pole should carry at 2 ft (0.61 m) below the tip is equal to 2.5/4 times the load that should be carried by the wood pole equivalent per ANSI 05.1 (ANSI, 1992). These numbers are shown in Table 5.13.2.

This method is recommended for line design; however, for guyed poles and poles with sustained bending or ex-

Table. 5.13.2. Equivalent ultimate loads of wood and concrete.

Wood designation	Standard wood ultimate load (lbs)	Prestressed concrete NESC Grade B ultimate load* (lbs)
H-6	11,400	7125
H-5	10,000	6250
H-4	8700	5450
H-3	7500	4700
H-2	6400	4000
H-1	5400	3375
1	4500	2825
2	3700	2325
3	3000	1875
4	2400	1500
5	1900	1200

Note: 1 lb = 4.45 N.

* Based on ratios of "when installed" overload factors.

Loads are assumed to be applied 2 ft (0.6 m) from the pole tip.

treme wind, the actual design loads should be considered rather than equating them to a wood pole. Table 5.13.2 delineates the appropriate ultimate loads required for prestressed concrete when used in conjunction with transverse loads.

5.13.3 Deflections

In general, due to the relative stiffness of the concrete and wood poles, an equivalent concrete pole will deflect much less than a wood pole. Hence, clearance and right-of-way due to conductor swing-out are generally not problems.

CHAPTER 6 — MANUFACTURING AND QUALITY ASSURANCE

6.1 General

Prior to approving bids from a concrete pole manufacturer, the user should be satisfied that each bidder has procedures in place to ensure that every pole supplied will be in compliance with the specifications. The manufacturer should provide either a full copy or a summary of the quality assurance program if requested. The user may inspect the manufacturer's equipment and process facility to ensure that the procedures are in accordance with the quality assurance program.

The exact contents and procedures will vary depending on the production process (e.g., static-cast or spun-cast), the types of poles being manufactured (e.g., mass produced street lighting poles or custom manufactured transmission line poles), and the general quality control philosophy of the manufacturer. There are, however, several considerations that should be covered by all quality assurance programs.

The following guidelines may serve in preparing specifications that include a quality assurance program:

6.2 Design and Drawings

The quality assurance specification should indicate the degree of involvement by the user and the procedure for review of the design concept, detailed calculations, stress analysis and the manufacturer's drawings. Stress analysis of the main structure and all of its component parts, including all attachments and connections, should be considered. The manufacturer's drawings should be checked to ensure that they contain proper and sufficient information for manufacturing and erection in accordance with the requirements of the user's specification.

6.3 Manufacturing Process

Prestressed concrete poles can be spun-cast or static-cast.

6.3.1 Spun-Cast Poles

To manufacture spun-cast concrete poles, concrete is pumped or placed into a self-stressing steel form consisting of two separable halves equipped with rolling rings. These rings rest on the wheels of a spinning machine that rotates the form at high speeds. The spinning provides centrifugal compaction to the concrete mixture and creates an inner core void.

The high consolidation forces and low water-cement ratios produce exceptionally dense concrete with a high compressive strength. The spinning process also results in improved bond between steel and concrete, greater shrinkage reduction and a smoother, denser surface finish.

6.3.2 Statically Cast Poles

Statically cast poles are typically made in tapered configurations that are square, octagonal or "H" shaped in cross section. The square or octagonal shaped sections can be either solid or made hollow by the use of retractable mandrels or fiber tube voids. Solid poles can have a wire raceway provided by a plastic tube extending through the center. Although not as dense as spun concrete [dry unit weight of 145 to 150 lbs per cu ft (2323 to 2403 kg/m^3) for static-cast poles vs. 155 to 165 lbs per cu ft (2483 to 2643 kg/m^3) for spun-cast poles], statically cast poles can also achieve high compressive cylinder strengths.

6.3.3 Materials

The specification should include the requirement for review and agreement on the manufacturer's materials specifications, sources of supply, material identification, storage, traceability procedures and acceptance of certified material test reports.

The manufacturer should maintain records of mill certifications and test reports from material suppliers to show that all materials used conform to applicable ASTM specifications. Tests on the concrete mix should be maintained, whether these tests are performed by independent laboratories or by the manufacturer. In either case, these tests should be conducted in accordance with ASTM procedures.

6.3.4 Tolerances

Appendix E lists some of the recommended manufacturing tolerances.

6.3.5 Sealing Strand Ends

The ends of strands must be properly sealed against water intrusion. It has been demonstrated that the helical prestressed strand reinforcement can act as capillary tubes and draw water up into the member. Hence, it is extremely advantageous to burn back the strand into the member approximately 1 in. (25.4 mm) and then seal the area with an epoxy grout or similar impervious material. This is especially desirable for pole installations in areas with high water tables or for those placed directly in sea water. Wherever strands are terminated in a pole, care should be taken to ensure that the strands are protected against weathering and corrosion.

6.3.6 Quality Control

A review should be made and agreement reached on all quality control procedures. Rejection criteria should be established and agreed upon prior to the start of any fabrication. All structures ready for shipment should have complete and proper identification in order to avoid confusion at the delivery point. The markings should coincide with the type, length, strength, weight and identification (ID) number required by the customer and approved on the shop drawings.

CHAPTER 7 — STRUCTURE TESTING

7.1 General

A pole structure test may be considered to verify structural design. This test is the ultimate check on the adequacy of the entire design and manufacturing process. Poles may be tested in either a horizontal or an upright position. If only the pole is being tested, a horizontal test is satisfactory and easier to carry out than an upright test (see Appendix F). In instances where the pole is being tested as a part of an entire structure, the entire assembled structure should be tested in the vertical position.

The contract documents should designate the organization that is responsible for the structural design specifications set forth in the contract. Overall responsibility for the testing of the structure should lie with one person representing this organization. This person should be completely familiar with the design of the structure and approve the proposed procedure for structural testing. Also, this person should be present at all times during the testing sequence and approve each decision made during the process. The person handling these responsibilities should be called the Responsible Test Engineer.

In a traditional proof test, the test setup conforms to the design conditions (i.e., only static loads are applied), the structure has level, well-designed foundations and the restraints at the load points are the same as in the design model. This type of test will verify the adequacy of the main components of the structure and their connections to withstand the static design loads specified for that structure as an individual entity under controlled conditions.

Proof tests provide insight into the actual stress distribution of unique configurations, fit-up verification, the performance of the structure in a deflected position and other benefits. This test cannot confirm how the structure will react in the transmission line where the loads will be both static and dynamic, the foundations may be less than ideal and there is some restraint from intact wires at the load points.

Sections 7.2 through 7.13 present guidelines for performing a proof test using a test frame that has facilities to install a single structure in an upright position, to load and monitor pulling lines in the vertical, transverse and longitudinal directions and to measure deflections. Guidelines for a horizontal test are presented in Section 7.14.

7.2 Foundations

It is unlikely that soil conditions at the test site will match those at the installation site. Fortunately, if a few precautions are taken, it will make very little difference to the test results.

7.2.1 Single Pole Structures

The primary consideration in designing and installing a single pole foundation is to be able to control the ground line rotation so as not to exceed the allowable design rotation. For test purposes, the actual amount of rotation makes very little difference within a wide range except under very heavy vertical loads, where secondary moments can be significant.

7.2.2 H-Frame Structures

Normally for an H-frame, the critical point in the structure is at the top of the X-brace. The magnitude of the ground line rotation has very little effect on the structure at the top of the X-brace. It is important, however, that the uplift and down-thrust be adequately contained so that the structure does not suffer premature failure due to unanticipated loads as a result of twisting the structure.

7.3 Material

The test structure should be made of materials that are representative of the materials that will be used in the production structures. Test results should be available for each important member in the test structure. All test structure material should conform to the requirements of the material specified in design.

7.4 Manufacture

Manufacture of the prototype structure for testing should be done in the same manner and to the same tolerances and quality control as specified for the production structures.

7.5 Assembly and Erection

The test structure should be assembled in accordance with the manufacturer's recommendations. It may be desirable to specify detailed methods or sequences for the test structure to prove the acceptability of proposed field erection methods. Pick-up points designed into the structure should be used during erection as part of the test procedure. The completed structure should be set within the tolerances permitted in the construction specification.

After the structure has been assembled, erected and rigged for testing, the user or his designated representative should review the testing arrangement for compliance with the contract documents. Safety guys or other safety features may be loosely attached to the test structure and used to minimize consequential damage to the structure or to the testing equipment in the event of a premature failure, especially if an overload test to failure is specified.

7.6 Test Loads

The loads to be applied to the test structure should be the loads specified for design and should include all appropriate load factors. Wind-on-structure loads are normally applied in a test as concentrated loads at selected points on the structure in a pattern to make a practical simulation of the in-service uniform loading. The magnitudes and points of application of all design loads should be developed by the structure designer and approved by the user before the test.

7.7 Load Application

Load lines should be attached to the load points on the test structure in a manner that simulates the in-service load application as much as possible. The attachment hardware for the test should have the same degrees of movement as the in-service hardware.

V-type insulator strings should be loaded at the point where the insulator strings intersect. If the insulators for the structures in service are to be a style that will not support compression, it is recommended that wire rope be used for simulated insulators in the test. If compressed or cantilever insulators are planned for the structures, members that will simulate those conditions should be used.

As the test structure deflects under load, load lines may change their direction of pull. Adjustments must be made in the applied loads so that the vertical, transverse and longitudinal vectors at the load point in the deflected shape are the loads specified in the structure loading schedule.

Test rigging should be designed with an adequate safety factor for the specified test loads.

7.8 Loading Procedure

The number and sequence of load cases tested should be specified by the structure designer and approved by the user. It is recommended that those load cases having the least influence on the results of successive tests be tested first. Secondly, the sequence should simplify the operations necessary to carry out the test program.

Loads are normally incremented to 40, 50, 75, 90 and 100 percent of the maximum specified load and to the load at which the concrete first cracks (usually in the range of 40 to 60 percent). If the test facility does not have the capability for continuous recording of loads, an additional increment to 95 percent may be added. There should be a pause after each load increment application to allow time for reading deflections and to permit the engineers observing the test to check for signs of structural distress. The maximum load for each load case should be held for 5 minutes.

In most cases, loads should be removed between load cases. In some non-critical situations, with the permission of the Responsible Test Engineer, the load may be adjusted as required for the next load case. Unloading should be controlled to avoid overstressing any members.

7.9 Load Measurement

All applied loads should be measured as close to the point of application to the test structure as possible. Loads should be measured through a suitable arrangement of load cells or by predetermined dead weights. The effects of pulley friction should be minimized. Measurement devices should be used in accordance with the manufacturer's recommendations and calibrated before and after the conclusion of the testing sequence.

7.10 Deflections

Structure deflections under load should be measured and recorded. Points to be monitored should be selected to verify the deflections predicted by the design analysis. Deflection readings should be made for the before-load and load-off conditions as well as at all intermediate holds during loading. Deflections should be referenced to common base readings, such as the initial plumb positions, taken before any test loads are applied.

Upon release of test loads after a critical load case test, a structure will normally not fully return to its undeflected starting position. The testing specifications should state how much deviation is acceptable.

7.11 Failures

The provisions of the test procedure must specify whether failure occurs when there is: (1) structure collapse; (2) initial cracking; (3) zero tension condition; or (4) permanent deformation.

If a premature structural failure occurs, the cause of the failure mechanism should be determined and corrected. Failed and damaged members should be replaced. The load case that caused the failure should be repeated. Load cases previously completed normally are not repeated.

After the structure has successfully withstood all load cases, and assuming that the structure was not tested to destruction, the structure should be dismantled and all members inspected.

7.12 Disposition of Test Structure

The test specification should state what use, if any, may be made of the test structure after the test is completed. Undamaged components are usually accepted for use in the line. If an overload test to failure has been performed, caution should be exercised in accepting parts that appear to be undamaged because they may have been loaded beyond the elastic range.

7.13 Report

The testing agency should furnish a test report in the number of copies required by the job specifications. The report should include:

1. The designation and description of the structure tested.

2. The name of the utility that will use the structure.

3. The name of the organization that specified the loading and test arrangement of the structure.

4. The name of the Responsible Test Engineer.

5. The name of the manufacturer.
6. A brief description and the location of the test facility.
7. The names and affiliations of the test witnesses.
8. The dates of testing each load case.
9. Design and detail drawings of the structure including any changes made during the testing program.
10. A rigging diagram with a detail of the point of attachment to the structure.
11. Calibration records of the load measuring devices.
12. A loading diagram for each load case tested.
13. A tabulation of deflections for each load case tested.
14. In the case of a failure:
    (a) Photographs of the failure.
    (b) Loads at the time of failure.
    (c) The remedial actions taken.
    (d) The physical dimensions of the failed members.
    (e) Test coupon reports of failed members, if required.
15. Photographs of the overall testing arrangement and rigging.
16. Air temperature, wind speed and direction, any precipitation and other pertinent meteorological data.
17. Mill test reports for steel and concrete cylinder compression breaks taken at the time of the test.
18. Additional information specified by the purchaser.

7.14 Horizontal Testing

Horizontal testing is primarily used to test a single pole. Most of the previous sections of this chapter also apply to horizontal testing. A full-scale horizontal destruction test should verify the structural integrity of the pole to withstand the maximum design stresses. All critical points along the pole shaft should be tested to maximum design load.

7.14.1 Test Arrangement

The structure is normally placed in a horizontal position as shown in Appendix F. Location(s) along the shaft will be selected as the load pulling point. The purpose of the load pull will be to duplicate the maximum design stress at all critical points in the pole shaft based on the cross-sectional geometry of the shaft and yield strength of the materials. Critical points are those points on the shaft with the highest stress.

7.14.2 Equipment Used in Test

The load is pulled at a predetermined point(s) along the shaft by a crane or other suitable pulling device. Loads should be determined with a calibrated load instrument located in the pulling line. A tape or transit should be used to take deflection measurements.

7.14.3 Test Procedure for Pole Test — Horizontal Pull

The pole is placed between the reaction blocks and locked in place. One or more wheeled support devices should be used to support the weight of the cantilevered end of the pole. An initial load of at least 10 percent of the maximum test load should be applied to "set" the pole into the blocking. When the "setting" load is removed, the zero position is then established from which to measure subsequent deflections.

In order to obtain accurate results, it is very important that the wheeled support device operates with a minimum of friction. Ideally, the setup includes steel wheels with bearings or steel rollers, either of which will roll on the steel plate. All of the rolling surfaces must be kept free of debris.

CHAPTER 8 — ASSEMBLY AND ERECTION

8.1 Handling

One of the most critical handling phases for any pole is lifting it clear of all supports while it is in the horizontal position because the moment generated by its own weight may be significant. Concrete poles tend to be heavier than other types of members; therefore, more attention must be paid to the manner in which they are lifted.

Some poles are designed to be lifted with a single-point pick at the center of gravity and some require multiple-point picks. The manufacturer should provide the user with lifting instructions for their particular poles and the user should transmit these instructions to the construction personnel.

The proper placement of slings will allow either two-point handling or single-point erection without any cracking. It is advantageous to haul, handle and erect poles without incurring flexural cracks. The presence of flexural cracks indicates that the concrete has already given up its initial tensile value, leaving only the "zero tension" (repeated crack) load for service as previously described.

8.2 Hauling

Common sense is important in determining good hauling practices. A particular setup that may be acceptable for hauling over a smooth paved highway may be entirely inappropriate for hauling the same load over a plowed and frozen field. In general, no more than one-third of the length of the pole should be cantilevered and, if the terrain conditions indicate that the pole will be handled roughly, the length should be less than that value.

In instances where hauling equipment cannot be driven adjacent to the setting location, it may be necessary to drag the pole along the ground. As is expected with the dragging of any pole, careful procedures are required to avoid damage to the pole.

8.3 Framing

Concrete poles are generally framed using through bolts. The turn-of-the-nut method for tightening bolts is preferred to torquing bolts and nuts, particularly when they are galvanized. In most cases, the bolt will be properly tightened if the nut is first tightened snugly (defined as the degree of tightness caused by the first impacting of an impact wrench). Then, the nut receives an additional turn depending on bolt length as follows:

- Short bolts (length less than four times the diameter) — one-third turn
- Medium length bolts (length between four and eight diameters) — one-half turn
- Long bolts (length greater than eight diameters) — three-quarters turn

The strength of the pole is sufficient to withstand a reasonable degree of bolt tightness, except near the ends of a hollow pole that has not been plugged. If a hollow spun-cast pole shows signs of cracking longitudinally when the bolts are tightened, either the bolts can be tightened less, a steel sleeve can be used in the hole or the end of the pole can be plugged if that is where the cracks are occurring.

This recommended tightening procedure will keep the bolts tight and protect the pole from damage by over-tightening. The turn-of-the-nut method should be used with caution in order to avoid pulling out inserts or yielding A307 bolts.

8.4 Field Drilling

Most concrete poles will be sent from the manufacturer with the necessary holes already in place. Occasionally, it may be necessary to field-drill holes. This can be accomplished with a rotary hammer drill and a carbide tipped bit or a diamond tipped core bit. An appropriate set of instructions are given herein for the two most common types of pole reinforcing methods.

8.4.1 Full Length Reinforcing Steel

Some manufacturers determine the amount of steel required by the ground line design moment capacity and carry that quantity of prestressing steel throughout the entire length of the pole, even though less steel is needed in the upper portions. Because holes are normally drilled in the upper portions of a pole where there is a considerable excess of steel, it is permissible to cut a limited number of strands in the drilling process.

The steel requirements are determined near the ground line, the lower end of the top section of a two-piece pole, and near an X-brace attachment in H-frame construction. Cutting the steel in these areas may weaken the pole below its design requirement.

If there is any question as to the advisability of cutting prestressing steel, the pole manufacturer should be contacted for guidance. By referring to the manufacturer's drawings, it may be possible to find areas where drilling can occur without cutting prestressing steel.

Once permissibility to drill the pole has been determined, the location and drill should be marked. If steel is struck when using a carbide bit, drilling should be stopped and either continued with a diamond tipped core or a cutting torch (to burn the steel). It is recommended that holes be drilled from the outside to the inside. Drilling straight through the pole usually spalls concrete on the opposite side. Mold marks, which are usually visible on the pole, make handy reference points from which to locate the hole on the opposite face.

8.4.2 Drop Out Reinforcing Steel

As the need for steel decreases toward the top of the pole, some manufacturers terminate a portion of the steel by masking out some of the strand with plastic tubing, dropping the tendons out through the side wall or installing additional steel in critical areas by use of post-tensioned strands. When using these methods, there is not an excess of steel near the pole tops and the steel should not be cut.

This situation does not preclude drilling these poles. It means, however, that care should be used to ensure that the steel is not cut. Because there is less steel in pole tops of this type, there is more space between the prestressing steel. Thus, it is easier to avoid the prestressing steel during the drilling process but cutting a strand means that the pole may be weakened below its design strength. The actual drilling of these poles is accomplished in the same manner as stated in the previous section.

8.4.3 Circumferential Steel

Cutting of circumferential steel (spiral reinforcement) is difficult to avoid, but is acceptable unless the pole is to be subjected to severe torsional loads.

8.5 Field Cutting

There will be occasions in which it is desirable to shorten a pole in the field. This can be accomplished without damage to the pole by cutting with a handheld concrete saw and an abrasive cut-off blade. The blade will cut both the concrete and the steel. After cutting, the exposed strands should be burned back and the voids sealed (see Section 6.3.5).

8.6 Erection

Concrete poles are erected in the same manner as other poles. Assuming that the poles were properly placed before they were framed, a single-point pick with a choker is usually permissible. The choker should be placed well above the center of gravity. This means that as the pole is raised from the horizontal position, much of its weight stays on the ground until the pole is nearly in the vertical position. Once it reaches the vertical position, it will not be damaged by lifting its full weight with a single-point pick.

Because the surface of a concrete pole is smooth and hard, care should be taken when using chokers. Improper use of chokers can result in the pole slipping and causing injury or property damage. Chokers must be tight around the pole. A positive stop against sliding can be provided by attaching the choker below a solid piece of hardware. (Note that ladder clips and step bolts do not qualify as solid hardware.)

8.7 Climbing

Concrete poles are climbed with the use of step bolts and ladders. Construction personnel may be required to install clips and climbing provisions.

8.8 Field Inspections

Questions about cracks in concrete poles are frequent. Although some types of cracks may be detrimental, concrete poles are expected to crack under certain loading conditions.

Concrete poles should be delivered to the user by the manufacturer in an uncracked condition. It is the user's responsibility to instruct the contractor's personnel how to handle the poles during erection so as not to crack them.

Concrete poles that have circumferential cracks that have closed have the same ultimate moment capacity as designed, but have a reduced cracking moment capacity. Circumferential cracks that do not close when the poles are either properly supported on the ground or are erected indicate poles in which the steel has been stretched beyond its elastic limit and should be replaced. Longitudinal cracks are less common. At either end, they may have been caused by the application of prestress loads. Longitudinal cracks may also be caused by over-tightening of the through bolts and by stacking poles too high in storage piles.

As long as the cracks are only hairline cracks, as opposed to open cracks, they should not be detrimental to the life of the pole in a noncorrosive environment. Normally, unsealed cracks narrower than the thickness of a common sheet of paper [approximately 0.1 mm (4 mils)] are not detrimental to the life of the pole. For more information on cracking, refer to ACI Committee 224 documents (AC1 224-2, 1995; ACI 224-3, 1995).

Any open cracks should be investigated for the cause and a determination should be made as to the structural adequacy of the pole. If it is decided that the pole is to remain in service, the cracks should be filled and sealed from the weather to prevent further degradation of the pole.

CHAPTER 9 — FOUNDATIONS

9.1 General

Foundation requirements for a concrete pole structure depend on the loads that will be transmitted to the foundation and the surrounding soil conditions.

9.2 Design Considerations

9.2.1 Loads

The foundation must be adequately designed to resist the structure reactions, which include shears, overturning moments, torsion, uplift or compression due to the factored loads. If the foundation is to be designed stronger than the structure that it supports, reduced strength factors can be used together with the foundation strength formula as described in the ASCE Loading Guide (ASCE, 1991).

9.2.2 Soil Exploration

In order to properly design a foundation, the foundation designer must have information pertaining to the existing soil conditions. The amount and type of information required will depend on the designer's experience with similar structures and soil types. Conditions may dictate soil testing. These tests are usually performed by geotechnical or soils engineers. The test results generally include the bearing capacity, lateral soil strength, cohesion values (clays and silts), unit weights, angle of internal friction (sands), depth of water table and a detailed soil description.

9.2.3 Performance and Reliability

Foundation deflection and rotation should be considered in the design. Excessive deflection or rotation of the foundation will create an undesirable appearance and may cause unfavorable load redistribution. Foundation design should incorporate adequate design factors to minimize future plumbing or adjustment of the structure.

9.3 Foundation Types

Foundations for concrete pole structures can normally be classified as one of the following three types:

- Direct embedment foundation
- Cast-in-place foundation
- Precast foundation

9.3.1 Direct Embedment Foundation

Direct embedment (see Fig. 9.3.1) is the most common type of foundation used on concrete pole structures. It con-

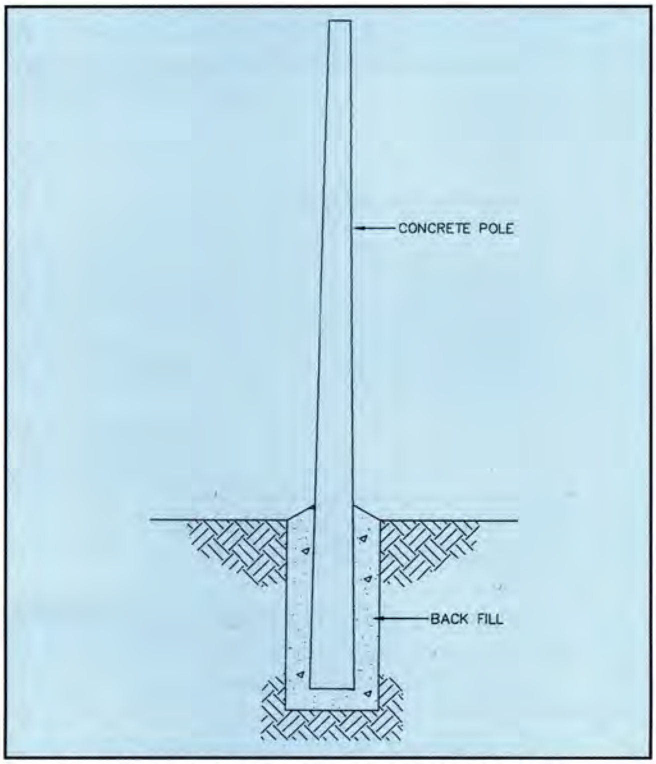


Fig. 9.3.1. Direct embedment foundation (drilled shaft).

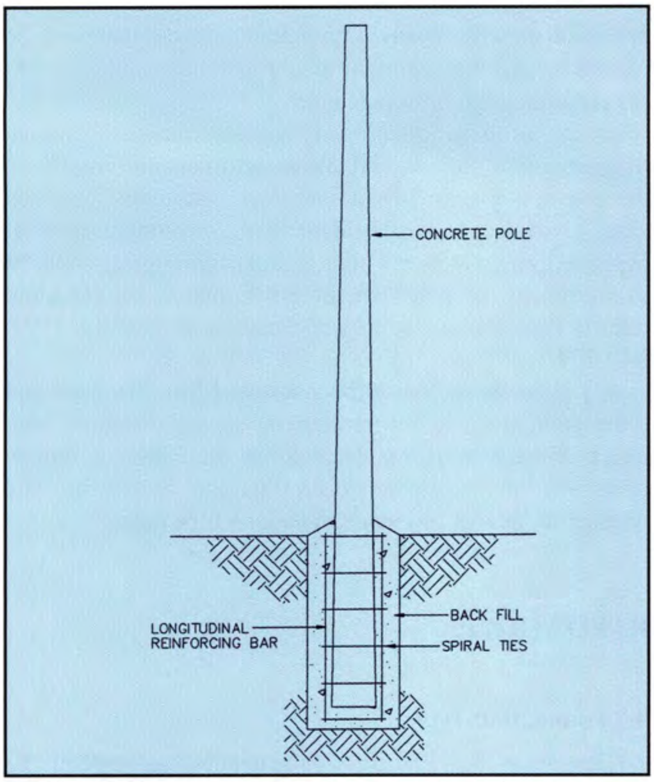


Fig. 9.3.2.a. Cast-in-place foundation (reinforced concrete drilled pier).

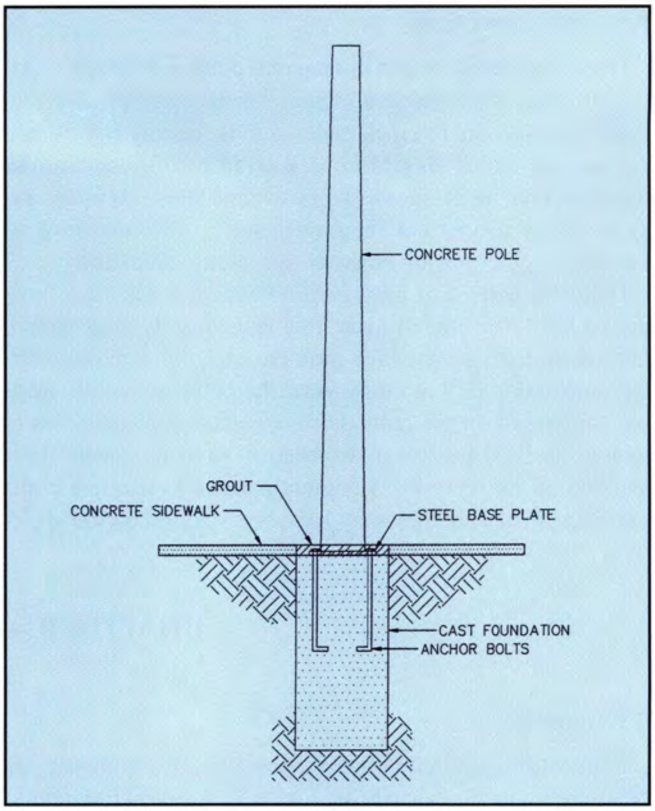


Fig. 9.3.2.b. Cast-in-place foundation (steel base plate).

sists of placing the pole directly in the ground by excavating a drilled shaft or by use of a jetting device.

The drilled shaft method involves placing the base of the pole in an excavated hole and backfilling the hole with compacted sand, gravel or concrete. The backfill material type and compaction should be specified depending on the anticipated performance of the foundation.

The jetting device method uses hydrostatic pressure to hydraulically displace the soil and allow the pole to "slip" into place. This method of embedding poles is best suited for granular or loose soils.

9.3.2 Cast-In-Place Foundation

Cast-in-place foundations (see Figs. 9.3.2.a and 9.3.2.b) refer to the excavation of soil at the structure location and the placement of steel reinforcement and ready-mixed concrete. Common types of cast-in-place foundations are the reinforced concrete drilled pier, the spread footing and light pole bases.

The reinforced concrete drilled pier is an excavated shaft with longitudinal reinforcing bars and circumferential spiral ties. The concrete pole can be placed inside and cast with the reinforcement or may be attached to the top of the drilled pier through the use of anchor bolts and a base plate cast to the base of the pole.

Spread footings may be either square or rectangular in shape. They consist of shallow excavations with mats of reinforcing bars. Spread footings are typically specified due to poor subsurface soils or where there may be underlying obstructions that would preclude the use of an excavated drilled shaft. The most common method for attaching the

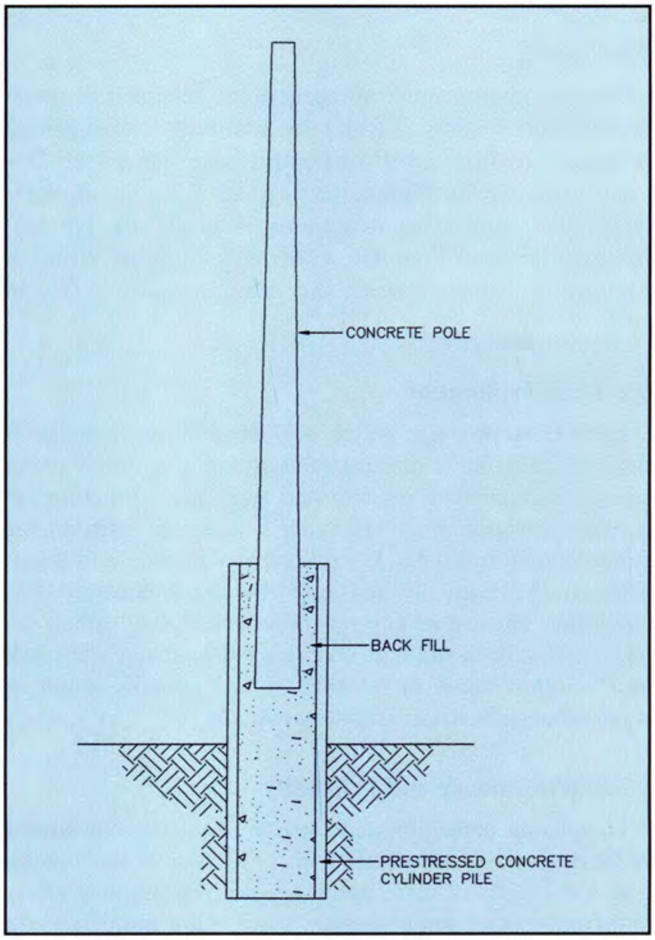


Fig. 9.3.3. Precast foundation (prestressed concrete cylinder pile).

pole to the spread footing is the use of anchor bolts and a base plate cast to the base of the pole.

Light pole bases are usually small cast-in-place foundations with anchor bolts for attachment with anchor base style poles. These bases are designed with or without reinforcing bars, depending on soil conditions and base size.

For street light applications, the base may be recessed below the sidewalk so that the finished connection may be covered with grout. For parking lot applications, the bases are formed extending above the pavement to protect the pole from damage by vehicle impact loadings.

9.3.3 Precast Foundation

Precast foundations (see Fig. 9.3.3) are foundations that are cast in a manufacturing facility or fabricated near the site to be erected at the final structure location. The two most common types of precast foundations are prestressed concrete cylinder piles and prestressed concrete piles.

Prestressed concrete cylinder piles are fabricated by spinning concrete in segmental molds and post-tensioning the segments together to the length specified. These piles are usually available in diameters ranging from 36 to 66 in. (914 to 1676 mm). Cylinder piles may be driven, jetted or excavated and placed to the desired embedment depth. The concrete pole is placed inside the cylinder pile and held in place by specified backfill.

The concrete cylinder pile may be used to obtain additional structure height without increasing the pole length. This is accomplished by specifying a pile length that includes the embedment depth plus additional above-ground length for the required structure height.

Precast, prestressed concrete piles may also be installed by driving, jetting or excavation and placed to the desired embedment length.

Lightly loaded structures may require a single pile foundation. The concrete pole may be attached to the pile by plates anchored to the end of the pile and the base of the pole. These plates may be either bolted or welded together. The concrete pole may also be attached to the single pile by bands or through bolts. Heavily loaded structures may require groups of piles connected together by a pile cap to resist the loads. The concrete pole may be cast with the pile cap or attached to the pile cap by a base plate and anchor bolts.

GLOSSARY

This Glossary serves as an aid to the reader in understanding the terms used in this Design Guide. Many of these terms have more than one meaning in the technical literature. The definitions provided herein have been specifically worded so as to be relevant to the practices used in the concrete pole industry.

Cantilevered Structure — A structure that is assumed fixed at one end and free to translate and rotate at the other end.

Circumferential Cracks — Cracks that parallel a cross section of a concrete pole.

Dead End Structure — A transmission or distribution structure on which conductors are terminated. These structures may be guyed or unguyed.

Design Bending Moment — The moment, at various points in a structure, generated by design factored loads.

Distribution Structures — Structures used to distribute electricity of low voltage (normally up to 50 kV).

Dynamometer — An instrument used to measure force. Dynamometers commonly have dial type scales that allow the reading of loads.

First Crack — The load (cracking moment) on a pole at which the concrete extreme fibers can no longer contribute tensile resistance.

Guyed Structure — A structure in which cable supports are used to increase its lateral load resistance.

Line Designer — The engineer(s) with overall transmission/distribution line design and specification writing responsibilities. A line designer is either employed by or is a hired consultant of a utility or company that uses transmission structures.

Load Cell — A device used to measure test loads.

Longitudinal Cracks — Cracks in concrete that parallel the long axis of the pole.

Manufacturer — The company responsible for the fabrication of the structures. The manufacturer fabricates the structures based on design drawings developed by the structure designer.

Multiple-Point Picks — The process where a pole or structure is suspended or picked up from more than one point.

Overload Capacity Factor — A term used by the NESC to designate a load factor. Service loads are multiplied by the overload capacity factor to obtain ultimate loads.

P-Δ Moment — The secondary moment created by vertical loads acting on the deflected structure.

Rake — The amount of horizontal pole top displacement created by installing a pole tilted out of plumb. It is used to negate the pole top deflection anticipated for everyday loading conditions.

Responsible Test Engineer — The person assigned overall responsibility for a structure test.

Single-Point Pick — The process where a pole is suspended or picked up using a single lifting point.

Snug Tight — A term used to describe the condition of the bolts in a connection when the plies of the joint are in firm contact. This is normally attained by a few impacts of an impact wrench or the full effort of a worker using an ordinary spud wrench.

Spiral Reinforcement — Mild steel reinforcement that encloses the longitudinal reinforcement. In concrete poles it is usually continuous throughout the pole length.

Spun-Cast — Fresh concrete placed in a mold and spun to form a pole.

Statically Cast — Fresh concrete placed by gravity and vibrated in a mold.

Strain Poles — Single poles subjected to permanent pull from cables, such as poles used to support traffic signals and highway signs.

Structure Designer — The engineer(s) with specific responsibility for the structural design of the poles. This person is usually employed by or is a hired consultant of a company that fabricates concrete pole structures.

Sweep — A measure of deviation from straightness.

Taper — The unit of measure describing the change in diameter of a section over a unit of length.

Test Rigging — Collectively, all the ropes, chains, cables and tackle used to apply load to a structure being subjected to testing.

Transmission Structures — Single pole or H-frame structures used to transmit high voltage electricity (above 50 kV).

Ultimate Load — A maximum design load that includes the appropriate overload capacity factors and any additional factor of safety specified.

Ultimate Moment Capacity — The resisting strength of a member, also referred to as ϕM_n, or the nominal strength multiplied by the capacity reduction factor ϕ.

User — The person(s) responsible for the acquisition of concrete pole structures.

Zero Tension — The load at which a crack, having previously opened by exceeding the tensile stress, will open again (also known as the decompression load or the repeated cracking load).

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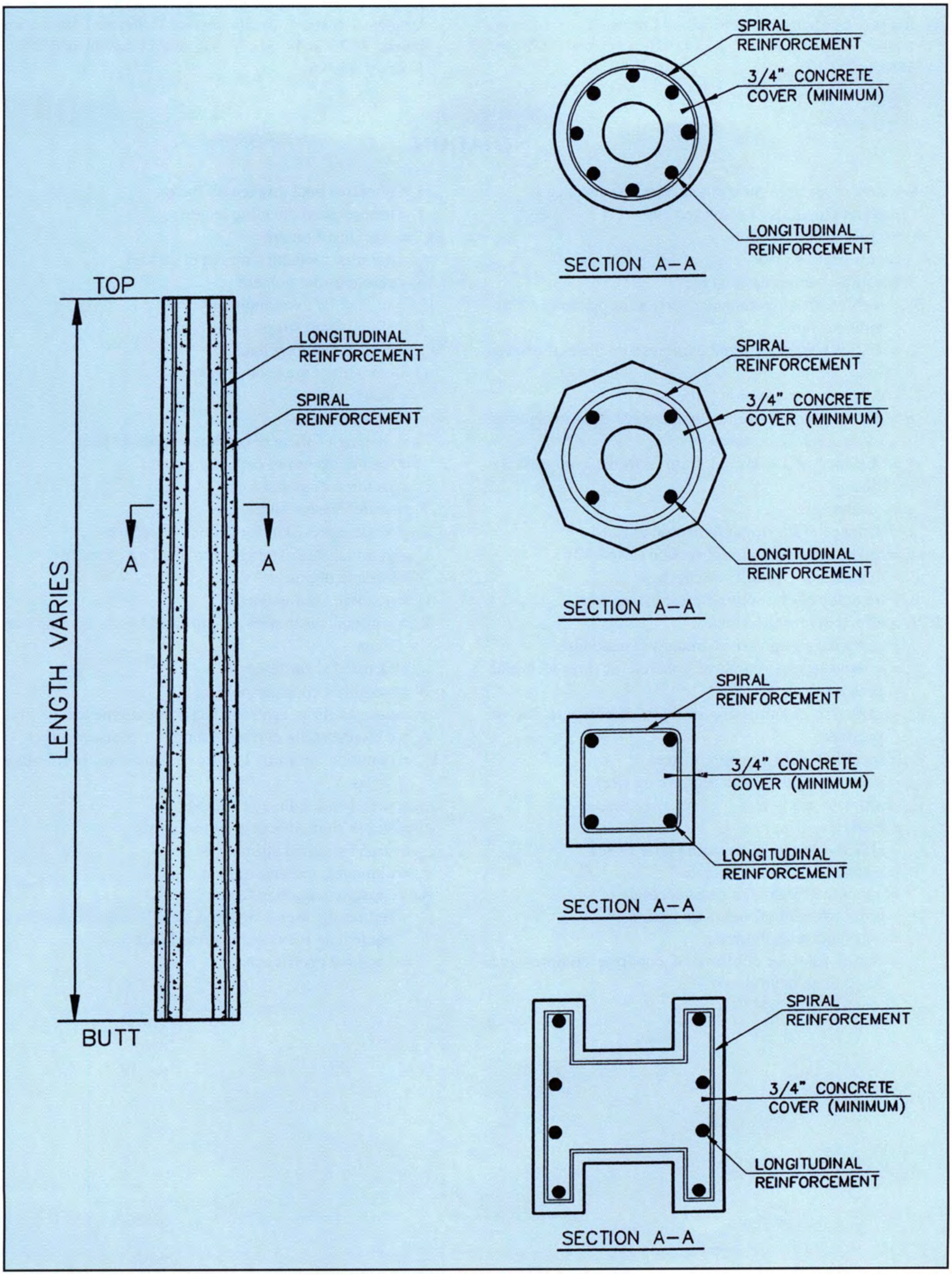
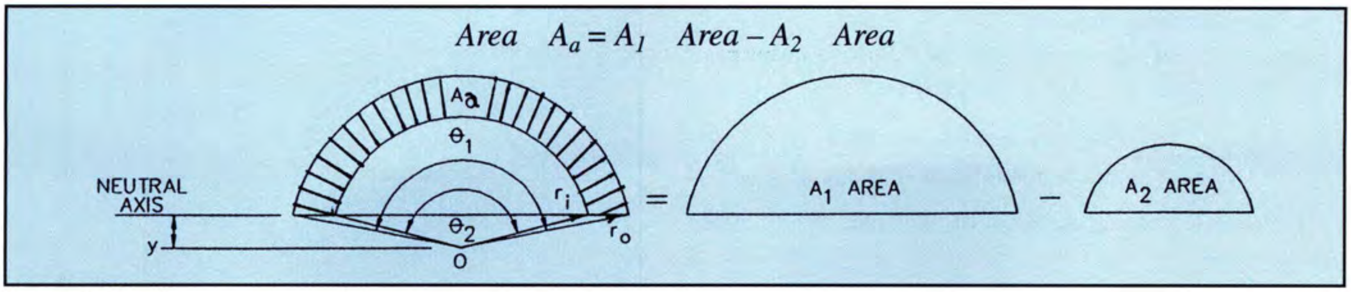
NOTATION

A_v = area of shear reinforcement within a distance s
A_a = area of annulus of cross section
A_g = gross area of section
A_{psi} = area of ith strand
b = width of compression face
b_w = web width of rectangular section or diameter of circular section
c = distance from extreme compressive fiber to neutral axis
C_c = concrete compression force
d = distance from extreme compressive fiber to centroid of steel reinforcement
d_i = distance of ith strand from extreme compressive fiber
e = eccentricity
e_i = distance of ith strand to neutral axis
E = modulus of elasticity of section (also MOE)
E_c = modulus of elasticity of concrete
E_s = modulus of elasticity of prestressing steel
EPA = effective projected area
f_c' = specified compressive strength of concrete
f_{ci}' = compressive strength of concrete at time of initial prestress
f_{pc} = effective compressive strength of concrete due to prestress
f_{py} = specified yield strength of steel
f_y = yield strength of mild reinforcing steel
f_{pu} = ultimate strength of prestressing steel
f_r = modulus of rupture
f_{sei} = effective stress in ith strand after losses
F_t = tensile strength of concrete
I = moment of inertia of cross section
I_g = gross moment of inertia of section
J = polar moment of inertia
K = factor relating centroid of concrete compressive force C_c to neutral axis
k = effective buckling length factor
L = unsupported buckling length
M_{cr} = cracking moment
M_n = nominal moment strength at section
M_o = zero tension moment
M_u = applied factored moment
P = effective prestress
P_{cr} = critical buckling load
Q = moment of area above centroid
r_i = inside radius
r_o = outside radius
s = spacing of shear or torsional reinforcement
t = wall thickness of concrete section
T_u = factored torsional force
T_s = strand tension force
T_c = torsional resistance of concrete section
V_c = nominal shear strength provided by concrete
V = volume of pole
V_n = nominal shear strength
V_s = nominal shear strength provided by shear reinforcement
V_u = factored shear force
W = weight of concrete pole
y_t = distance from centroidal axis to extreme tensile fiber
ε_o = concrete strain corresponding to maximum stress
ε_{ce} = concrete strain at level of strand due to effective stress
ε_{sb} = strand strain due to bending
ε_{se} = strand strain due to effective stress
ε_{su} = strand strain at ultimate
ε_u = ultimate concrete strain at rupture
ϕ = capacity reduction factor
β_1 = reduction factor applied to c to obtain depth of equivalent rectangular stress block
ζ = torsional coefficient

APPENDIX A — TYPICAL ARRANGEMENT OF REINFORCEMENT



APPENDIX B —
AREA AND CENTROID OF ANNULUS



Area of Annulus

1. Determination of A_1 area.

Consider S, half of the area A_1:

$$r_o^2 = x^2 + y^2, \text{ hence } y = \sqrt{r_o^2 - x^2}$$

$$S = \int_d^{r_o} dA = \int_d^{r_o} y\,dx = \int_d^{r_o} \sqrt{r_o^2 - x^2}\,dx$$

Let: $x = r_o \cos\phi$ and $dx = (-r_o \sin\phi)d\phi$

Therefore:

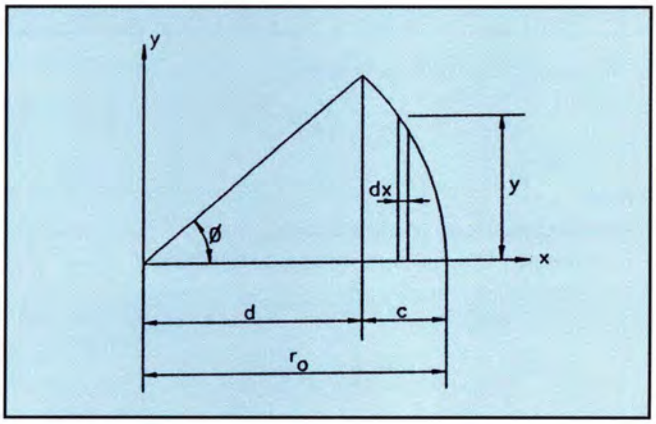
$$S = -\int_\phi^o \sqrt{r_o^2 - r_o^2\cos^2\phi}\,(r_o\sin\phi)d\phi$$

$$= -\int_\phi^o r_o^2 \sin\phi \sin\phi\, d\phi$$

$$= -r_o^2 \int_\phi^o \sin^2\phi\, d\phi$$

$$= -r_o^2 \left(\frac{\phi}{2} - \frac{\sin^2\phi}{4}\right)_\phi^o$$

$$A_2 = \frac{r_o^2}{2}(2\phi - \sin 2\phi) \text{ or } A_1 = \frac{r_o^2}{2}(\Theta_1 - \sin\Theta_1)$$



in which the central angle $\Theta_1 = 2\phi$:

$$\Theta_1 / 2 = \tan^{-1}\left(\sqrt{r_o^2 - y^2} / y\right)$$

2. Determination of A_2 area.

$$A_2 = r_i^2(\Theta_2 - \sin\Theta_2)$$

with radius r_i and central angle Θ_2:

$$\Theta_2 / 2 = \tan^{-1}\left(\sqrt{r_i^2 - y^2} / y\right)$$

Centroid of Annulus

The centroid of the annulus is the point of intersection of two axes which place the body in equilibrium. Axis 1 bisects the area of the annulus and Axis 2 divides the annulus into two equal areas.

Area of annulus:

$$A_1 = {}^1\!/_2 r_o^2(\Theta_1 - \sin\Theta_1) - {}^1\!/_2 r_i^2(\Theta_2 - \sin\Theta_2)$$

where

$$\Theta_1 / 2 = \tan^{-1}\left(\sqrt{r_o^2 - y^2} / y\right)$$

and

$$\Theta_2 / 2 = \tan^{-1}\left(\sqrt{r_i^2 - y^2} / y\right)$$

Area above Axis 2:

$$A_2 = {}^1\!/_2 r_o^2(\Theta_3 - \sin\Theta_3) - {}^1\!/_2 r_i^2(\Theta_4 - \sin\Theta_4)$$

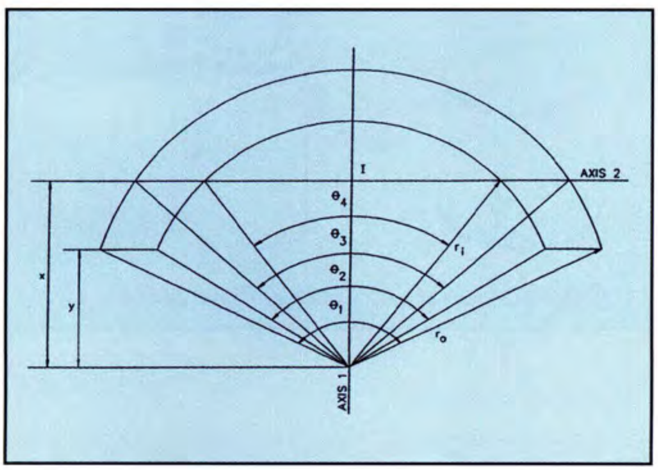
where

$$\Theta_3 / 2 = \tan^{-1}\left(\sqrt{r_o^2 - x^2} / x\right)$$

and

$$\Theta_4 = \tan^{-1}\left(\sqrt{r_i^2 - x^2} / x\right)$$

Since y is known, x is found by trial and error until $A_2 = {}^1\!/_2 A_1$.



APPENDIX C — DETERMINATION OF STRAND STRESS

1. Strand strain at ultimate:

Strand strain at ultimate refers to the following quantities:

A. Strand strain due to effective prestress:

$$\varepsilon_{se} = \frac{\text{Effective prestress of strand}}{\text{Area of strand} \times \text{modulus of elasticity of strand}}$$

B. Concrete strain at the level of the strand due to effective prestress:

$$\varepsilon_{ce} = \frac{\text{Total effective prestress of strands}}{\text{Gross area of concrete} \times \text{modulus of elasticity of strand}}$$

C. Strand strain due to bending:

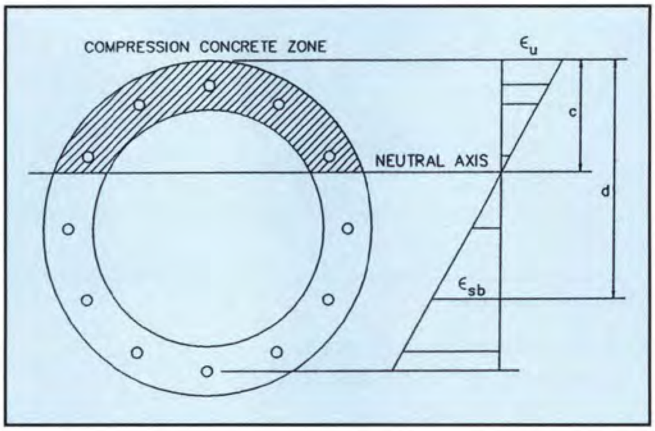
$$\varepsilon_{sb} = \frac{\varepsilon_u(d-c)}{c}$$

where

ε_u = concrete strain at ultimate = ε_{sei}

d = distance from compression side to strand

c = location of neutral axis



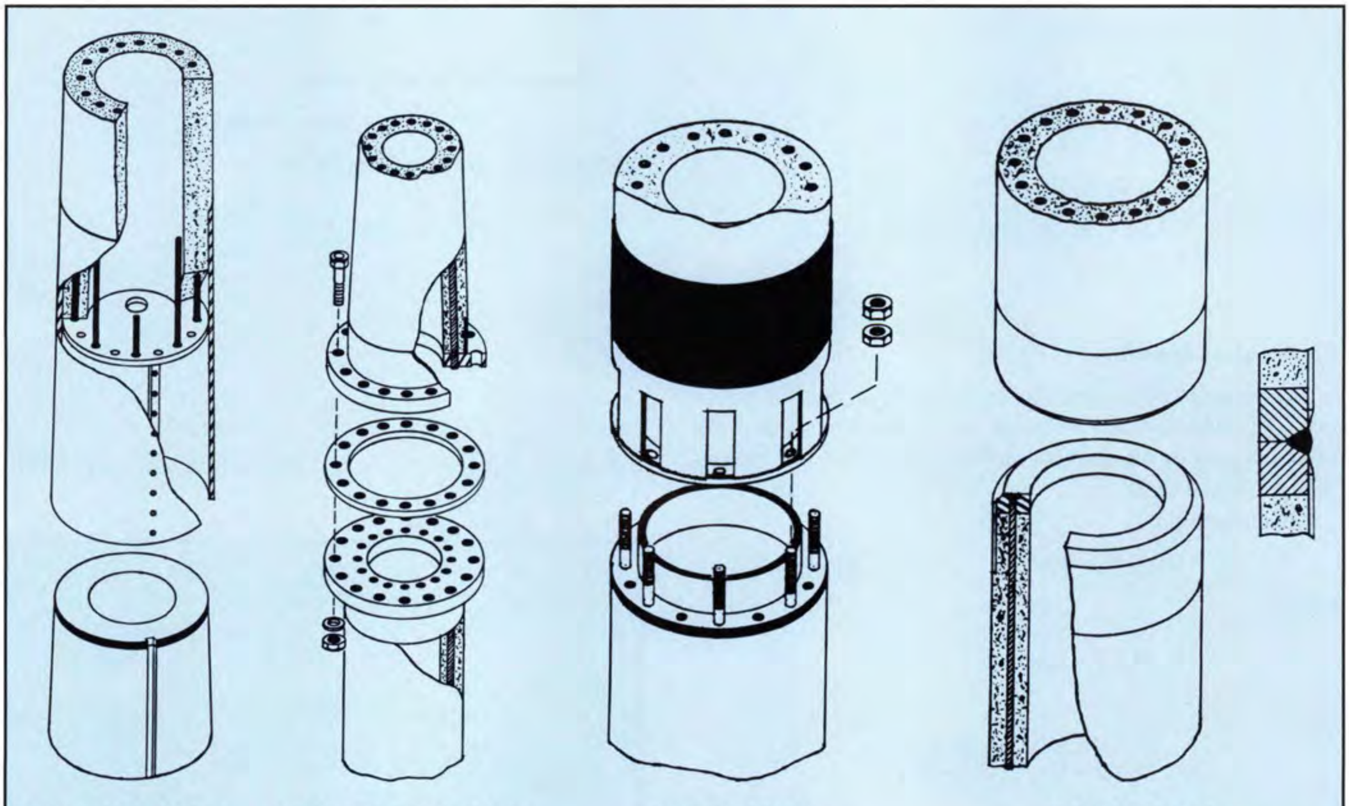
The quantity is positive in the tension concrete zone and negative in the compression concrete zone.

In summary, $\varepsilon_{su} = \varepsilon_{se} + \varepsilon_{ce} + \varepsilon_{sb}$

2. Strand stress f_{sei} of the ith strand:

$$f_{sei} = \varepsilon_{sei} \times \text{Modulus of elasticity of strand}$$

APPENDIX D — SPLICING



1. Slip Joint Splice 2. Flange Plate Splice 3. Bolted Splice 4. Welded Splice

APPENDIX E — SUGGESTED MANUFACTURING TOLERANCES

1. Length: ±2 in. (50.8 mm), or ±1 in. (25.4 mm) or ±1/8 in. (3.2 mm) per 10 ft (3.05 m) of length, whichever is greater.

2. Cross section:

(a) Outside dimension: ±0.25 in. (6.3 mm).

(b) Wall thickness — minus 12 percent with a minimum of 0.25 in. (6.3 mm).

(Note: This requirement may be waived provided that structural adequacy and durability are not impaired. Plus tolerance of wall thickness is limited by Tolerance No. 4, "Weight.")

3. Sweep: 0.25 in. (6.3 mm) per 10 ft (3.05 m) of length. Sweep tolerance is applicable to the overall length or any 10 ft (3.05 m) segment thereof. It is a measurement of the deviation of the pole's surface from a straight line joining two surfaces in the same plane.

4. Weight: Plus 20 percent and minus 10 percent of calculated value.

5. Reinforcement placement:

(a) Individual longitudinal elements: plus 0.25 in. (6.3 mm), and ±0.125 in. (3.2 mm) for the centroid of a group.

(b) Spiral reinforcement: spacing may vary ±25 percent, but the total required quantity per foot (0.305 m) of pole length should be maintained.

6. Bolt holes:

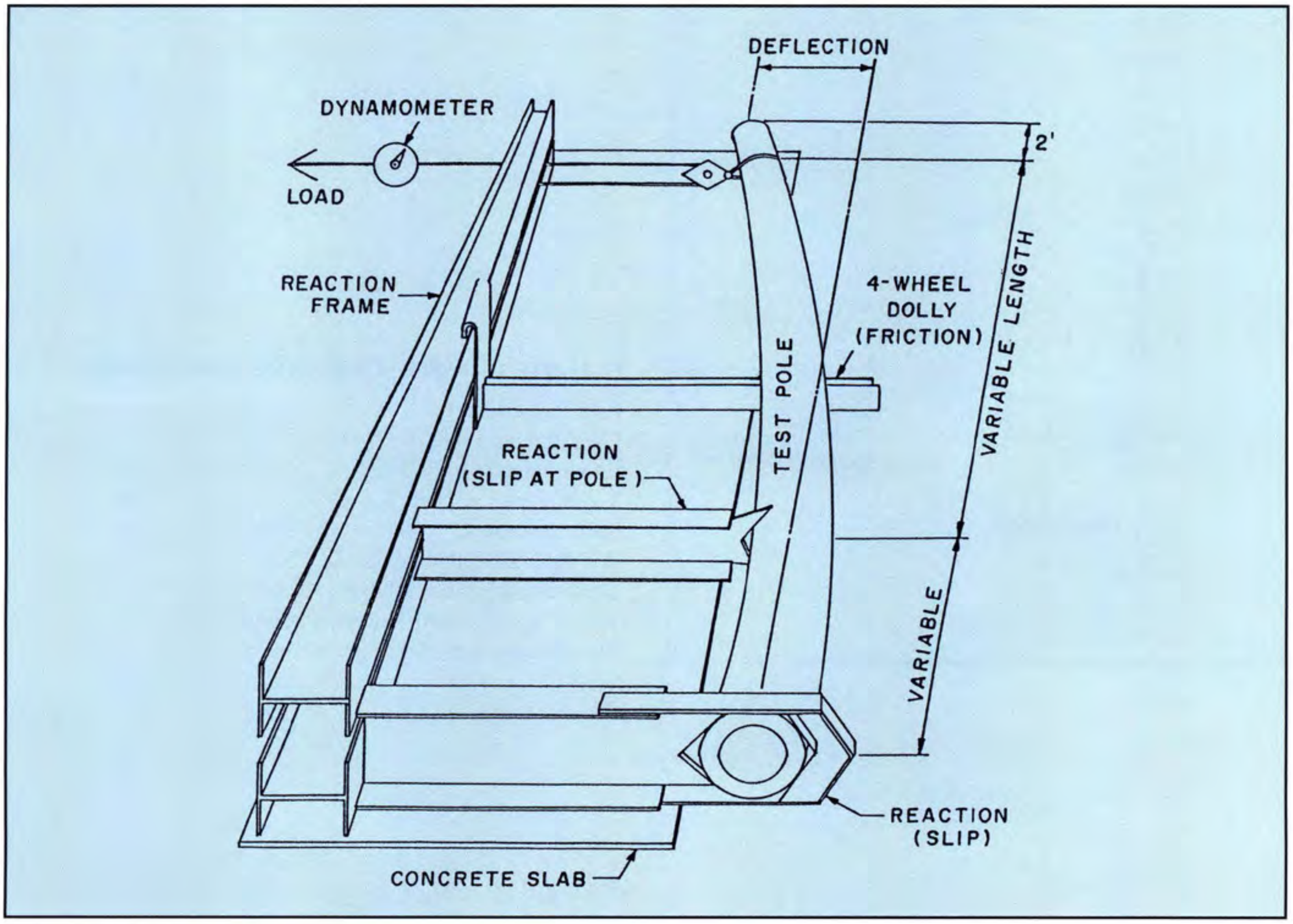
(a) Location: ±0.125 in. (3.2 mm) for holes within a group, ±1 in. (25.4 mm) for the centerline between groups and ±2 in. (50.8 mm) from the end of the pole.

(b) Hole diameter: ±0.0625 in. (1.6 mm) of the specified value. Bolt hole diameters should be specified at least 0.125 in. (3.2 mm) greater than the bolt diameter.

(c) Alignment: the alignment of holes within a group should not vary from the longitudinal pole centerline of that group by more than one-half of the hole diameter.

7. Aperture and hand-hole location: ±2 in. (50.8 mm) longitudinally and ± 1 in. (25.4 mm) transversely from the designated location.

APPENDIX F — POLE TEST SETUP



APPENDIX G — DESIGN EXAMPLES

The following examples illustrate how prestressed concrete poles may be used in typical transmission line structure applications. The examples include:

1. Self-supporting single pole

2. Guyed angle pole

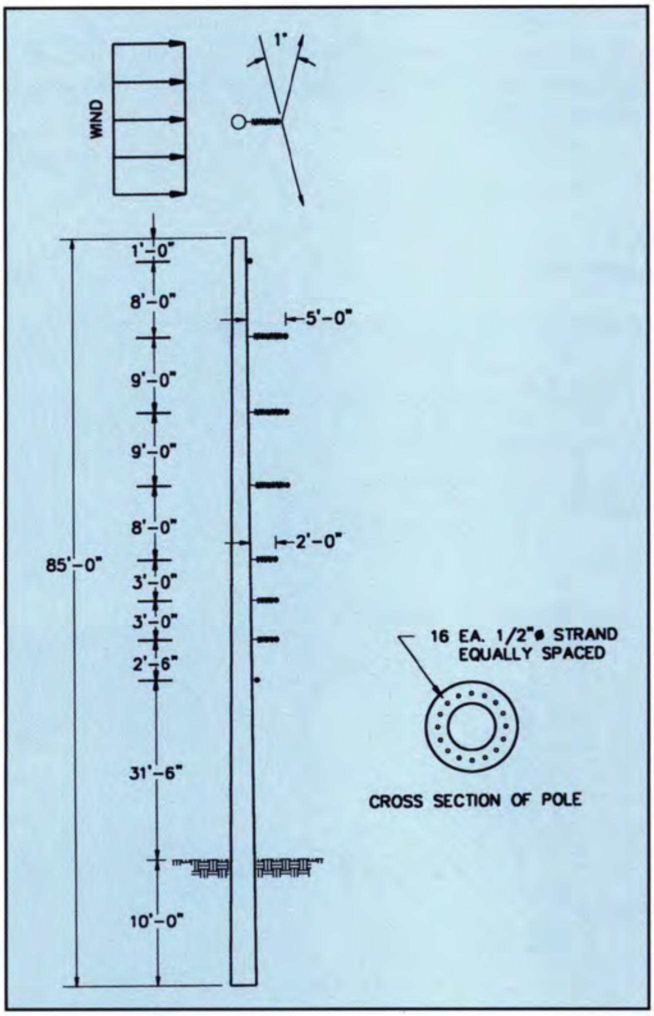
3. H-frame structure with X-bracing

Conductor information used for all examples is listed in the following table:

Conductor	Area (sq in.)	Diameter (in.)	Weight per foot (lbs)
$^3/_8$ in. 7 strand EHS steel	0.0792	0.375	0.273
795 kcmil 26/7 ACSR	0.7264	1.108	1.094
397.5 kcmil 18/1 ACSR	0.3295	0.743	0.432
#4/0 6/1 ACSR	0.1935	0.563	0.291

Note: 1 in. = 25.4 mm; 1 sq in. = 645.2 mm^2; 1 lb = 4.44 N.

The span lengths, line angles, and loading conditions are specified in each example.



EXAMPLE 1 — SELF SUPPORTING SINGLE POLE

Pole Properties:

Overall length = 85 ft (26 m)

Embedment = 10 ft (3 m)

Top diameter = 11 ft (3.3 m)

Bottom diameter = 29.4 in. (747 mm)

Weight = 17,000 lbs (7711 kg)

Concrete strength = 9000 psi (62 MPa)

16 - $^1/_2$ in. (12.7 mm) diameter strands (270K)

Effective prestress force per strand = 14,272 lbs (63482 N)

Round pole section properties.

Height above ground (ft)	Outside diameter (in.)	Inside diameter (in.)	Area (sq in.)	Moment of inertia (in.^{4})	Section modulus (cu in.)
75.0	11.0	6.0	77	754	137
74.0	11.2	6.2	79	806	144
66.0	12.9	7.7	95	1348	209
57.0	14.9	9.4	115	2235	300
48.0	16.8	11.1	136	3461	142
40.0	18.6	12.7	155	4929	530
37.0	19.2	13.2	163	5549	578
34.0	19.9	13.8	171	6269	630
31.5	20.4	14.3	177	6895	676
30.0	20.7	14.6	181	7292	704
20.0	22.9	16.5	209	10,413	910
10.0	25.0	18.4	238	14,410	1151
0.0	27.2	20.3	270	19,428	1428

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 sq in. = 645 mm^2; 1 cu in. = 16387 mm^3; 1 in.^{4} = 416231 mm^4.

Design loads for 400 ft (122 m) horizontal and vertical spans, 1-degree line angle.

	Case 1 – NESC Medium	Case 2 – High Wind	Case 3 – Broken Conductor
One shieldwire			
Vertical	390 lbs	170 lbs	170 lbs
Transverse	360 lbs	380 lbs	30 lbs
Each of three conductors			
Vertical	1260 lbs	670 lbs	340 lbs / 670 lbs
Transverse	720 lbs	1150 lbs	40 lbs / 80 lbs
Longitudinal			4290 lbs / 0 lbs
Each of three primary distribution cables			
Vertical	620 lbs	270 lbs	270 lbs
Transverse	550 lbs	770 lbs	60 lbs
One neutral wire			
Vertical	450 lbs	180 lbs	180 lbs
Transverse	450 lbs	580 lbs	50 lbs
Wind pressure on pole	10 psf	25 psf	0 psf

Note: 1 lb = 4.45 N; 1 psf = 4.88 kg/m^2.
All loads include NESC specified load factors.

Case 1 – NESC Medium.

Height (ft)	Deflection (in.)	Axial force (lbs)	Shear force (lbs)	Resultant moment (ft-kips)	Unfactored moment (ft-kips)	Cracking moment (ft-kips)	Ultimate moment (ft-kips)
75.0	30.47	0	0	0.0	0.0	0	0
74.0	29.88	505	360	0.0	0.0	43	76
66.0	25.21	2779	1170	9.7	5.2	53	100
57.0	20.11	5448	1994	27.7	13.8	68	142
48.0	15.32	8418	2833	53.9	25.8	83	184
40.0	11.04	10,826	3501	79.0	36.7	96	222
37.0	10.02	11,982	3915	93.5	42.1	103	245
34.0	8.68	13,379	4514	108.2	48.2	109	261
31.5	7.61	14,489	4964	121.3	53.6	114	276
30.0	6.98	15,129	5215	129.3	56.9	115	279
20.0	3.25	18,186	5396	183.4	80.3	140	347
10.0	0.87	21,717	5596	241.3	104.4	164	406
0.0	0.00	25,748	5814	302.6	129.3	191	466

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 lb = 4.45 N; 1 ft-kip = 1.36 kN-m.

Case 2 – High Wind.

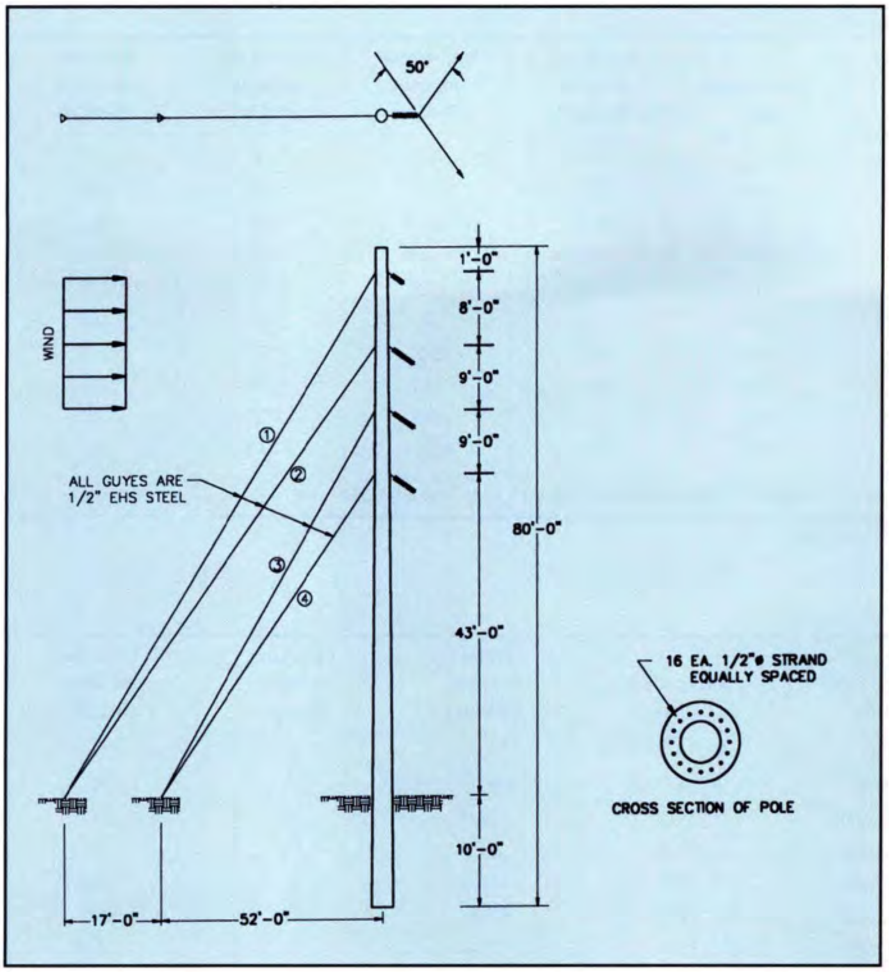
Height (ft)	Deflection (in.)	Axial force (lbs)	Shear force (lbs)	Resultant moment (ft-kips)	Cracking moment (ft-kips)	Ultimate moment (ft-kips)
75.0	67.89	0	0	0.0	0	0
74.0	66.67	247	380	0.0	43	76
66.0	56.94	1592	1754	7.5	53	100
57.0	46.13	3202	3165	28.5	68	142
48.0	35.66	5012	4613	63.7	83	184
40.0	25.47	6474	5678	102.7	96	222
37.0	23.56	7149	6309	126.1	103	245
34.0	20.45	7931	7201	148.4	109	261
31.5	17.97	8569	7853	168.9	114	276
30.0	16.50	8936	8207	181.7	115	279
20.0	7.64	10,974	8661	266.6	140	347
10.0	2.11	13,328	9160	360.9	164	406
0.0	0.00	16,015	9704	464.2	191	466

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 lb = 4.45 N; 1 ft-kip = 1.36 kN-m.

Case 3 – Broken Conductor.

Height (ft)	Deflection (in.)	Axial force (lbs)	Shear force (lbs)	Resultant moment (ft-kips)	Cracking moment (ft-kips)	Ultimate moment (ft-kips)
75.0	38.77	0	0	0.0	0	0
74.0	37.21	247	30	0.0	43	76
66.0	31.75	1262	4291	1.9	53	100
57.0	24.83	2872	4293	39.5	68	142
48.0	18.35	4682	4296	79.5	83	184
40.0	12.68	6144	4300	114.0	96	222
37.0	11.34	6819	4304	129.9	103	245
34.0	9.66	7607	4310	143.8	109	261
31.5	8.35	8240	4315	155.1	114	276
30.0	7.60	1606	4395	162.4	115	279
20.0	3.34	10,644	4315	205.4	140	347
10.0	0.85	12,998	4315	250.9	164	406
0.0	0.00	15,685	4315	298.6	191	466

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 lb = 4.45 N; 1 ft-kip = 1.36 kN-m.



EXAMPLE 2 — GUYED ANGLE POLE

Pole Properties:
Overall length = 80 ft (24.4 m)
Embedment = 10 ft (3 m)
Top diameter = 10 in. (254 mm)
Bottom diameter = 27.3 in. (693 mm)
Weight = 14,500 lbs (6577 kg)
Concrete strength = 10,000 psi (69 MPa)
12 - $^1/_2$ in. (12.7 mm) diameter strands (270K)
Effective prestress force per strand = 14,275 lbs (63495 N)

Round pole section properties.

Height above ground (ft)	Outside diameter (in.)	Inside diameter (in.)	Area (sq in.)	Moment of inertia (in.^{4})	Section modulus (cu in.)
70.0	10.0	5.0	67	518	104
69.0	10.2	5.2	69	561	110
61.0	11.9	6.7	85	990	166
52.0	13.9	8.4	104	1711	246
43.0	15.8	10.1	124	2747	347
40.0	16.5	10.7	131	3174	385
30.0	18.6	12.6	156	4951	531
20.0	20.8	14.5	183	7355	707
10.0	23.0	16.4	211	10,510	916
0.0	25.1	18.3	240	14,553	1159

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 sq in. = 645 mm^2; 1 cu in. = 16387 mm^3; 1 in.^{4} = 416231 mm^4.

Design loads for 400 ft (122 m) horizontal span, 550 ft (168 m) vertical span, 50-degree line angle.

	Case 1 – NESC Medium	Case 2 – High Wind	Case 3 – Broken Conductor
One shieldwire			
Vertical	200 lbs	90 lbs	90 lbs
Transverse	4640 lbs	2572 lbs	1390 lbs
Each of three conductors			
Vertical	850 lbs	500 lbs	250 lbs / 500 lbs
Transverse	11,165 lbs	7146 lbs	1810 lbs / 3620 lbs
Longitudinal			3885 lbs
Wind pressure on pole	10 psf	25 psf	0 psf

Note: 1 lb = 4.45 N; 1 psf = 4.88 kg/m^2.

Case 1 – NESC Medium.

Height (ft)	Deflection (in.)	Axial force (lbs)	Shear force (lbs)	Resultant moment (ft-kips)	Unfactored moment (ft-kips)	Cracking moment (ft-kips)	Ultimate moment (ft-kips)
70.0	5.6	0	0	0.0	0.0	0	0
69.0	5.6	8168	-2988	0.0	0.0	30	59
61.0	5.6	17,948	-709	-23.7	-12.2	38	85
52.0	4.9	28,957	1762	-30.0	-16.8	50	118
43.0	3.8	38,553	4282	-13.8	-7.8	62	156
40.0	3.3	39,392	4338	-1.0	-0.2	67	169
30.0	2.0	41,757	4489	42.8	25.4	83	214
20.0	0.9	44,557	4657	88.1	51.7	102	258
10.0	0.2	47,819	4844	135.1	78.7	122	302
0.0	0.0	49,632	4945	184.0	106.4	145	344
Guy tensions (lbs): (1) = 10,822 (2) = 11,965 (3) = 12,438 (4) = 11,354							

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 lb = 4.45 N; 1 ft-kip = 1.36 kN-m.

Case 2 – High Wind.

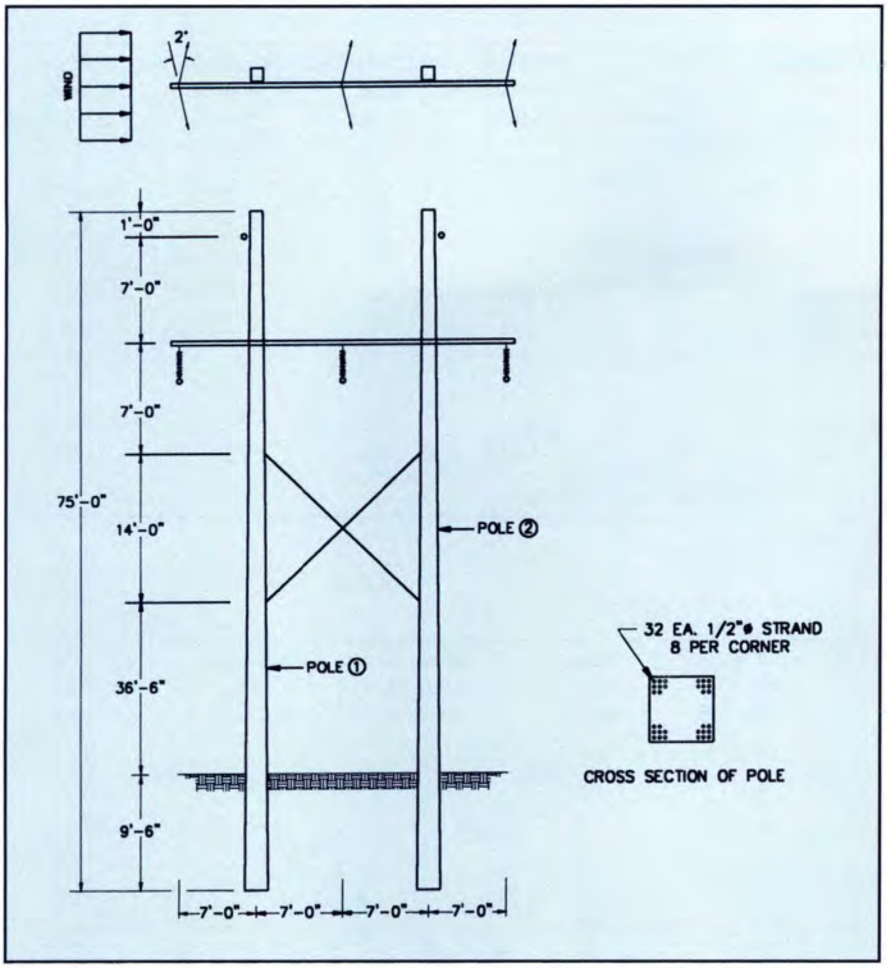
Height (ft)	Deflection (in.)	Axial force (lbs)	Shear force (lbs)	Resultant moment (ft-kips)	Cracking moment (ft-kips)	Ultimate moment (ft-kips)
70.0	3.5	0	0	0.0	0	0
69.0	3.5	5091	-2155	0.0	30	59
61.0	3.5	11,360	-578	-16.9	38	85
52.0	3.2	18,553	1036	-21.5	50	118
43.0	2.5	24,902	2587	-11.6	62	156
40.0	2.2	25,462	2726	-3.8	67	169
30.0	1.4	27,039	3104	24.4	83	214
20.0	0.6	28,905	3526	56.4	102	258
10.0	0.2	31,080	3993	92.8	122	302
0.0	0.0	32,288	4243	134.0	145	344
Guy tensions (lbs): (1) = 6775 (2) = 7699 (3) = 8178 (4) = 7600						

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 lb = 4.45 N; 1 ft-kip = 1.36 kN-m.

Case 3 – Broken Conductor.

Height (ft)	Deflection (in.)	Axial force (lbs)	Shear force (lbs)	Resultant moment (ft-kips)	Cracking moment (ft-kips)	Ultimate moment (ft-kips)
70.0	75.95	0	0	0.0	0	0
69.0	74.37	2157	-467	0.0	30	59
61.0	61.75	2462	3993	3.7	38	85
52.0	47.74	5505	3901	43.5	50	118
43.0	34.41	9383	4161	86.0	62	156
40.0	30.71	12,407	4162	100.6	67	169
30.0	17.72	13,720	4162	153.0	83	214
20.0	8.16	15,439	4162	204.5	102	258
10.0	2.1	17,485	4162	253.1	122	302
0.0	0.0	19,793	4162	297.1	145	344
Guy tensions (lbs): (1) = 2626 (2) = 3015 (3) = 3351 (4) = 3212						

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 lb = 4.45 N; 1 ft-kip = 1.36 kN-m.



EXAMPLE 3 — X-BRACED H-FRAME

Pole Properties:
Overall length = 75 ft (22.9 m)
Embedment = 9.5 ft (2.9 m)
Top diameter = 11 in. (279 mm)
Bottom diameter = 23.4 in. (594 mm)
Weight = 21,275 lbs (9650 kg)
Concrete strength = 6000 psi (41 MPa)
32 - 1/2 in. (12.7 mm) diameter strands (270K)
Effective prestress force per strand = 24,570 lbs (109287 N)
Inside hole formed by fiber tube void

Square pole section properties.

Height above ground (ft)	Outside diameter (in.)	Inside diameter (in.)	Area (sq in.)	Moment of inertia (in.^{4})	Section modulus (cu in.)
65.5	11.0	0.0	121	1220	222
64.5	11.2	0.0	125	1295	232
57.5	12.3	0.0	152	1920	312
50.5	13.5	0.0	182	2747	408
43.5	14.6	0.0	214	3817	522
36.5	15.8	0.0	249	5174	656
30.0	16.9	6.0	256	6666	791
20.0	18.5	8.0	292	9576	1035
10.0	20.2	10.0	328	13,267	1316
0.0	21.8	12.0	363	17,829	1635

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 sq in. = 645 mm^2; 1 cu in. = 16387 mm^3; 1 in.^{4} = 416231 mm^4.

Design loads for 900 ft (274 m) horizontal span, 1200 ft (366 m) vertical span, 2-degree line angle.

	Case 1 – NESC Medium	Case 2 – High Wind	Case 3 – Broken Conductor
Each of two shieldwires			
Vertical	840 lbs	500 lbs	360 lbs
Transverse	850 lbs	890 lbs	80 lbs
Each of three conductors			
Vertical	2960 lbs	1610 lbs	890 lbs / 1610 lbs
Transverse	1760 lbs	2720 lbs	130 lbs / 260 lbs
Longitudinal			3260 lbs / 0 lbs
Wind pressure on each pole	16 psf	44 psf	0 psf

Note: 1 lb = 4.45 N; 1 psf = 4.88 kg/m^2.

Case 1 – NESC Medium.

Height (ft)	Deflection (in.)	Axial force Pole 1 (kips)	Axial force Pole 2 (kips)	Shear force (kips)	Resultant moment (ft-kips)	Unfactored moment (ft-kips)	Cracking moment (ft-kips)	Ultimate moment (ft-kips)
65.5	1.2	0.0	0.0	0.0	0.0	0.0	0	0
64.5	1.2	1.4	1.4	0.9	0.0	0.0	0	0
57.5	1.0	7.4	7.4	3.6	7.3	3.2	25	52
50.5	0.8	-0.6	19.2	-6.1	33.3	14.9	55	113
43.5	0.8	1.7	21.5	-6.0	-9.7	-4.3	84	174
36.5	0.8	-5.1	34.5	4.1	-51.2	-23.1	113	235
30.0	0.7	-2.8	36.7	4.2	-24.8	-11.1	141	292
20.0	0.3	1.6	41.2	4.5	17.9	8.0	183	379
10.0	0.1	6.6	46.1	4.7	63.1	28.2	225	466
0.0	0.0	9.4	49.0	4.9	110.9	49.4	267	553
Cross-brace axial force = 14.0 kips								

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 kip = 4.45 kN; 1 ft-kip = 1.36 kN-m.

Case 2 – High Wind.

Height (ft)	Deflection (in.)	Axial force Pole 1 (kips)	Axial force Pole 2 (kips)	Shear force (kips)	Resultant moment (ft-kips)	Cracking moment (ft-kips)	Ultimate moment (ft-kips)
65.5	1.9	0.0	0.0	0.0	0.0	0	0
64.5	1.9	0.9	0.9	1.0	0.0	0	0
57.5	1.6	4.5	4.5	5.4	8.5	25	52
50.5	1.4	-9.5	21.3	-9.7	46.8	55	113
43.5	1.4	-7.9	22.9	-9.3	-20.2	84	174
36.5	1.4	-20.9	-20.9	6.6	-84.4	113	235
30.0	1.1	-19.3	-19.3	6.9	-41.6	141	292
20.0	0.6	-16.0	-16.0	7.6	29.4	183	379
10.0	0.2	-12.4	-12.4	8.3	107.2	225	466
0.0	0.0	-10.3	-10.3	8.7	192.6	267	553
Cross-brace axial force = 21.8 kips							

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 kip = 4.45 kN; 1 ft-kip = 1.36 kN-m.

Case 3 – Broken Conductor.

Height (ft)	Deflection		Axial force		Shear force		Resultant moment		Cracking moment (ft-kips)	Ultimate moment (ft-kips)
	Pole 1 (in.)	Pole 2 (in.)	Pole 1 (kips)	Pole 2 (kips)	Pole 1 (kips)	Pole 2 (kips)	Pole 1 (ft-kips)	Pole 2 (ft-kips)		
65.5	12.5	-4.1	0.0	0.0	0.0	0.0	0.0	0.0	0	0
64.5	12.2	-4.1	0.7	0.7	0.1	0.1	0.0	0.0	0	0
57.5	9.9	-3.3	3.7	3.9	4.9	-1.7	0.6	0.6	25	52
50.5	7.7	-2.6	4.0	6.2	4.9	-1.7	34.4	-11.7	55	113
43.5	5.7	-1.9	5.5	7.7	4.9	-1.7	68.5	-22.8	84	174
36.5	3.9	-1.3	6.6	10.8	4.9	-1.7	102.8	-34.5	113	235
30.0	2.6	-0.9	8.1	12.3	4.9	-1.7	134.5	-44.5	141	292
20.0	1.1	-0.4	11.0	15.3	4.9	-1.7	183.4	-61.1	183	379
10.0	0.3	-0.1	14.4	18.6	4.9	-1.7	232.4	-77.6	225	466
0.0	0.0	0.0	16.3	20.5	4.9	-1.7	278.3	-94.2	267	553
Cross-brace axial force = 1.4 kips										

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 kip = 4.45 kN; 1 ft-kip = 1.36 kN-m.