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CHAPTER C6
DESIGN OF CONNECTIONS

C6.1 INTRODUCTION

Transmission structures have traditionally been designed based on ultimate strength design methods using factored loads. The design stresses for connections in this specification are intended for limit state conditions, defined as the condition in which a component becomes unfit for service under factored loads. The design stresses of this specification were derived from research used to establish specifications for structural steel buildings published by AISC.

Resistance factors considered appropriate for the design of connections have been incorporated into the design stresses in this specification. The resistance factors for components vary depending on the manner and consequence of failure at the limit state condition and on the degree of certainty associated with the design methodology. The design stresses in this specification are applicable only to transmission structures and may deviate from AISC design stresses for building connections.

C6.2 BOLTED AND PINNED CONNECTIONS

Bolted connections for steel transmission pole structures are normally designed as shear or tension-type connections.

Pinned connections are those in which the attachments should be free to rotate about at least one axis while under load.

The minimum end and edge distances determined by the provisions of this section do not include allowances for fabrication tolerances.

Typical anchor bolt holes in base plates are 0.375 to 0.5 in. (10 to 13 mm) oversized.

C6.2.1 Material. Commonly used fastener specifications for steel transmission pole structures are ASTM A325, A354, A394, A449, and A490 for bolts, and A563 for nuts.

C6.2.2 Shear Stress in Bearing Connections. The nominal shear strength of a single high-strength bolt in a bearing connection has been found to be approximately 0.62 times the tensile strength of the bolt when threads are excluded from the shear plane. When there are two or more bolts in a line of force, nonuniform deformation of the connected material between the fasteners causes a nonuniform distribution of shear force to the bolts. Based on the number of bolts and joint lengths common to transmission structures, a reduction factor of approximately 0.95 has been applied to the 0.62 multiplier. Using a resistance factor of 0.75 results in a design stress equal to 0.45 F_u. A lower resistance factor may be appropriate for single-bolt connections; however, this would be offset by a joint length reduction factor of 1.0. Consequently, the design stress of 0.45 F_u is appropriate for single- and multiple-bolt connections typically used for transmission structures. When threads are included in the shear plane, it has been found that a reduction factor of 0.80 is appropriate, which results in a design stress approximately equal to 0.35 F_u. Both design stresses in the standard are based on the gross area of the bolt.

The shear strength used for testing A394 bolts may be used as the design strength when the bolts are ordered to include single shear lot testing. The length of typical joints using A394 bolts in transmission structures does not warrant the use of a joint length reduction factor.

C6.2.3 Bolts Subject to Tension. The nominal tensile strength of a bolt is equal to the tensile strength of the bolt material times the effective net area of the bolt. The design stress in the standard is based on applying a 0.75 resistance factor to nominal strength.

C6.2.5 Bearing Stress in Bolted Connections. Limiting the design bearing stress to 1.9 F_u will limit deformation of holes to an acceptable level for proper performance of transmission structures under service load conditions.

C6.2.6 Minimum Edge Distances and Bolt Spacing for Bolted Connections. The provisions of this section are applicable to sheared and mechanically guided flame-cut edges.

The requirement of 1.3d edge distance is a lower-bound requirement that has been used successfully for typical bolted connections for transmission structures. The requirement of $t + d/2$ is a requirement for thick members such that punching holes will not create a breakout condition. For other holes, this requirement is not necessary. Satisfactory punching of the holes in thick material depends on the ductility of the steel, the adequacy of the equipment (capability of the punching equipment and proper maintenance of punches and dies), the allowed tolerance between the punch and die, and the temperature of the steel. The following guidelines have been satisfactorily used:

- For 36 ksi (248 MPa) yield steel, the thickness of the material should not exceed the hole diameter.
- For 50 ksi (345 MPa) yield steel, the thickness of the material should not exceed the hole diameter minus 1/16 in. (1.6 mm).
- For 65 ksi (448 MPa) yield steel, the thickness of the material should not exceed the hole diameter minus 1/8 in. (3.2 mm).

The edge distance based on bolt force is based on a nominal tear-out stress equal to 0.60 F_u applied over two tear-out planes, one on each side of the bolt. The length of each tear-out plane is equal to the clear distance plus 1/4 bolt diameter. The minimum edge distance is based on a 0.80 resistance factor applied to the nominal tear-out stress.

C6.2.7 Bearing Stress in Pinned Connections. Clevis-type connections and insulator or guy shackle attachments are examples of pinned connections. The design stress is less than that for bolted connections to account for the lack of confinement, the use of oversize holes, the rotation, and the wear that is typical of a pinned connection.

The maximum bearing stress for a pinned connection is based on a nominal strength of 1.80 F_y times a 0.9 resistance factor rounded off to 1.65 F_y. Bearing stress limitations are based on F_y to limit material deformations and to satisfy joint rotation requirements. The maximum bearing stress in the previous edition of the standard was equal to 1.35 F_u, which for Grade 65 material resulted in a similar bearing stress limit compared to 1.65 F_y. Connections made with Grade 65 material based on this limit of bearing stress have performed well in service, and this performance is justification of the 0.90 resistance factor for bearing stress on pinned connections.

In addition, to avoid indentation and excessive wear of the material under everyday loading, the following should be met:

$$P \leq 0.6\,dtF_y \tag{C6.2-1}$$

where

P = Force transmitted by the pin,
d = Nominal diameter of the pin,
t = Member thickness, and
F_y = Specified minimum yield stress of the member.

Everyday loading can be defined as the sustained loading resulting from the bare wire weight at 60 °F (16 °C) final sag. If the location is subject to steady prevailing wind, the everyday loading can be considered to be the resultant load caused by the bare wire weight and the prevailing wind at 60 °F (16 °C) final sag.

C6.2.8 Minimum Edge Distances for Pinned Connections. The minimum edge distance requirement for a pinned connection is required to prevent a tension tear out across the net section perpendicular to the load. The minimum edge distance for hole diameters less than or equal to the pin diameter plus 1/2 in. (13 mm) is based on using a 0.75 resistance factor applied to a nominal strength of F_u times the effective net area. For larger hole diameters, a resistance factor equal to 0.65 is used to account for the associated additional bending stresses. The effective net area is based on the actual hole diameter plus 1/16 in. (2 mm).

Oversized holes are commonly used as load attachment points for insulator strings, overhead ground wires, and guys. These connections do not involve load reversal. The minimum edge distance requirement parallel to the load in Section 6.2.6 applies to oversized holes.

No adjustment is required to the minimum edge distances for slight chamfering. For attachment plates subject to bending, additional analysis is required to determine the plate thickness.

All connections should be investigated for tension rupture on the net section in accordance with Section 6.2.9. The net area is based on the actual hole diameter plus 1/16 in. (2 mm). For convenience, an equation for minimum edge distance perpendicular to the load is provided for pinned connection plates.

C6.2.9 Connection Elements and Members. Connecting elements and members should be proportioned to prevent yielding and rupture across their gross and net areas, respectively. Examples of connections that should be proportioned to limit the stresses specified are block shear ruptures at the ends of angles, coped members, and gusset plates; yielding or rupture through connection plates and connected members; and shear failures through the thickness of flange and base plates. The limiting design stresses are based on resistance factors equal to 0.90 for tension yielding, 1.00 for shear yielding, and 0.75 for rupture.

C6.3 WELDED CONNECTIONS

C6.3.3 Design Stresses. The design stresses in Tables 6.3, 6.4, 6.5, and 6.6 for welds are those of 1989 *AISC Specification* (Ninth Edition) *Allowable Bending Stress Design Aid* (AISC 1991), multiplied by 1.67. Punching shear stress should be considered in connection designs.

C6.3.3.1 Through-Thickness Stress. This restriction is applicable to plates welded perpendicular to or near perpendicular to the longitudinal axis of members (e.g., base plates, flange plates, and arm brackets) and takes into consideration the possible deficiencies in the tensile strength through the thickness of the plates, which may result in lamellar tearing. Lamellar tearing can occur in a plate of any thickness and is often caused by improper weld joint detailing and/or improper welding methods.

C6.4 FIELD CONNECTIONS OF MEMBERS

C6.4.1 Slip Joints. This common connection has been used on all types of structure applications. Whether a slip joint connection is suitable may vary depending on the following:

- Owner-specified critical dimensions such as minimum or maximum height of the assembled structure, minimum clearance to ground line, or minimum clearances between wire attachments,
- Structure designer's and/or fabricator's design/detailing practice,
- Construction assembly methods, and/or
- Magnitude and direction of axial load for H-frames and guyed poles.

Fabrication and erection tolerances should be accounted for when establishing the nominal or design lap length requirements. A commonly used practice is to define a nominal or design lap length that incorporates the minimum lap length required plus the fabrication tolerance, which can vary by fabricator and design practice. Experience has shown that the slip joint length performance is dependent on satisfying the following assembly steps:

1. Minimum jacking force has been applied.
2. Slip joint lap length after jacking is between the minimum and maximum specified values.
3. The joint is fully seated and no significant gaps between the mating sections are evident.
4. The application of additional jacking force does not result in additional movement of the joint.

If the pole has been assembled using the jacking force specified by the assembly documents and gaps exist between the sections before wires are strung and exceed either of the following, the condition should be referred to the structure designer for resolution:

- The sum of the lengths of gaps that exceed 1/8 in. (3 mm) is more than 30% of the slip joint's circumference.
- A gap extends across two full adjacent flats and the maximum gap exceeds 1/4 in. (6 mm).

Maximum lap should be restricted by practical factors, such as maintaining the minimum height of the assembled structure, maintaining minimum clearance to groundline, maintaining minimum clearances between wire attachments, and avoiding interference with climbing devices as defined in the contract specifications. For frame structures for which leg length tolerances are critical, the structure designer may consider using bolted flange connections as a substitute for slip joints. Bolted flange connections should also be considered for poles supporting switches where length tolerances are critical for attaching linkages for the operation of the switch.

Full-scale tests performed by Sumitomo Steel (in the 1970s in Japan) and Electric Power Research Institute (EPRI) (in the 1990s in the United States) have indicated that the full capacity of the slip joint is achieved with a minimum slip joint length of at least 1.5 times the largest inside diameter across the flats of the outer section. The EPRI tests were performed on joints assembled with 30,000 lb (130 kN) of force. The summary of test data of splice failures indicated that slightly more than a 1.5 length is required to achieve full strength (Figure C6-1).

ASCE MOP 72 and earlier versions of that document provided various minimum slip joint lap ratios, ranging from 1.35 minimum to 1.5. These past ratios largely depended on proprietary testing. The ratios were based on the largest outside diameter across the points of the outer section. A conversion of this ratio to the more common definition of "the largest inside diameter across the flats" is not straightforward because the conversion depends on the pole diameter and plate thickness. However, comparing inside (flat-to-flat) diameters ranging from 20 to 70 in. (500 to 1,800 mm) and plate thicknesses from 0.1875 to 1.00 in. (5 to 25 mm) (using those with $w/t \leq 40$) provides a range of values of 1.41 to 1.53. In addition, the majority of this testing was performed on sealed pole sections that likely provided additional strength compared with sections that are left open at the ends.

C6.4.2 Base and Flange Plate Connections. Theoretical methods of analysis for base plate design have not been published. It is recommended that details and practices proven through testing be used. Appendix F provides a proposed method to determine the plate thickness for a base plate supported by anchor bolts with leveling nuts. Traditionally, base plates are designed to be supported by anchor bolts with leveling nuts and without grout. Grouting of base plates is not recommended for reasons described in C11.5.1.

In certain types of structures (e.g., guyed poles or frame structures), the calculated design loads may be significantly less than the load capacity of the tubular member at the base plate or flange joint. It is not considered good engineering practice to size the base or flange plate connection for loads significantly lower than the tube capacity. Thus, 50% of tube capacity has been established as a minimum strength requirement for such welded joint connections.

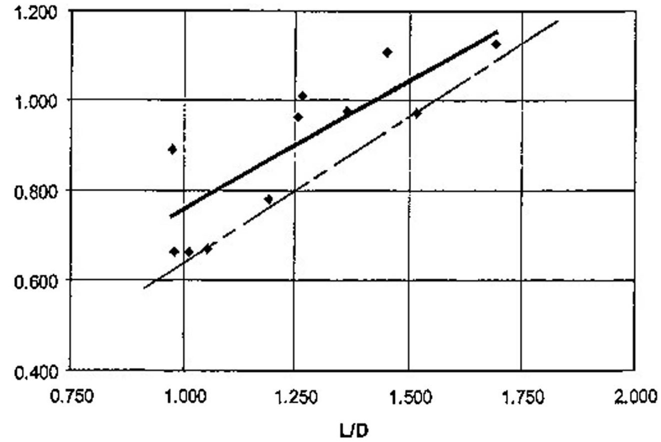

Figure C6-1. All EPRI splice failures.

Source: Courtesy of Electric Power Research Institute; reproduced with permission. EPRI makes no warranty or representations, expressed or implied, with respect to the accuracy, completeness, or usefulness of the information contained in the Material. Additionally, EPRI assumes no liability with respect to the use of, or for damages resulting from the use of the material.

Note: The solid line represents the least squares fit of the data, and the dashed line represents the line fitting the data for minimum strength.

C6.5 TEST VERIFICATION

Theoretical methods of analysis for arm connections have not been published. It is recommended that details and practices proven through testing be used.

CHAPTER C7
DETAILING AND FABRICATION

C7.1 DETAILING

C7.1.1 Drawings. The design of steel transmission poles, including the preparation of shop detail and erection drawings, is typically performed by the fabricator. Occasionally, the owner provides shop detail drawings as part of the contract documents, and their correctness is the responsibility of the owner. Differences between the owner's drawing requirements and the fabricator's shop practices need to be resolved before beginning fabrication.

C7.1.2 Drawing Review. The structure designer's review of drawings includes responsibility for the strength of members and connections. The correctness of dimensional detail calculations is the responsibility of the fabricator. Review of drawings does not include approval of means, methods, techniques, sequences, procedure of construction, or safety precautions and programs.

The owner's review is for determining conformance with the contract requirements. It does not relieve the fabricator of the responsibility for the accuracy of the structural detailing.

C7.1.3 Erection Drawings. The erection drawings are prepared as an aid in assembly and erection. They can be used with, but do not eliminate the need for, a construction specification. Erection drawings should show the position and lead of all guys.

C7.1.4 Shop Detail Drawings. The shop detail drawings are prepared as the communication, or link, between the design and the fabrication processes. As such, comprehensive detailing of fabrication requirements is very important. Sections 7.1.4.1 through 7.1.4.5 provide standard requirements of the shop detail drawings. Shop detail drawings facilitate quality assurance checks both before and after fabrication.

C7.1.4.2 Dimensions and Tolerances. Clearance and appearance requirements are normally established by the owner, whereas strength and assembly requirements are established by the structure designer. Foundation type, structure design, and construction methods are factors that should be considered when establishing tolerances.

The owner should coordinate dimensioning of mating parts obtained from different sources. The structure designer or the owner should either impose tolerances that ensure ease of assembly or require preassembly and match marking of mating parts by the fabricator. The structure designer should establish tolerances to control critical cross-sectional properties and to control the magnitude of the internal reactions. For example, a maximum variation of –5% for section modulus is recommended. This is within tolerances set for standard structural members covered by the ASTM A6 specification.

C7.1.4.4 Corrosion and Finish Considerations. Surface preparation should reference a Steel Structures Painting Council (SSPC) specification when possible. Drawings should show painting requirements, including the paint system, surface preparation, mil coverage, number of coats, and color. Paint manufacturer's application recommendations should be available.

Galvanizing should reference the applicable ASTM specification. ASTM A123 is typically referenced for plates and shapes. ASTM A153 is referenced for hardware. Venting and draining details should be indicated.

Metalizing requirements should be shown, including type of metalizing (e.g., zinc or aluminum), surface preparation, mil coverage, and sealing. Application instructions should be documented and available. AWS C2.18, *Guide for the Protection of Steel with Thermal Sprayed Coatings of Aluminum and Zinc and Their Alloys and Composites*, is a good reference.

C7.1.4.5 Other Requirements. Examples of specific requirements include the following:

- *Drilled hole* for holes specified as drilled and not punched.
- *Hot bend* when forming is to be done hot and not cold.
- *Acceptable welding processes* when one or more processes are unacceptable.

C7.2 FABRICATION

C7.2.1 Material. A wide variety of steels are used for steel pole structures. Therefore, the fabricator needs to carefully maintain the material identity throughout fabrication.

C7.2.2 Material Preparation. Material preparation includes cutting, bending, and machining. This standard defines the performance requirements but does not specify the methods to be used to accomplish these operations.

C7.2.2.1 Cutting. Cutting includes operations such as shearing, torch cutting, and sawing. Material that is to have straight edges can be cut to size with a shear; however, care should be taken to prevent cracks or other defects from forming at the sheared edge. Limitations of section size and length of the shear should be considered to ensure a good cut.

Any curved or straight edge can be cut with a burning torch. Care should be taken to prevent cracks or other notch defects from forming at the prepared edge, and all slag should be removed. Wherever practical, the torch should be mechanically guided. Edges prepared for welding or subject to high stresses should be free from sharp notches. Reentrant cuts should be rounded. Edges cut with a handheld torch may require grinding or other edge preparation to remove sharp notches. Steel can be cut with a reciprocating band saw–type blade, circular stone saw, or friction saw.

C7.2.2.2 Forming. Braking, rolling, stretch bending, and thermal bending are forming processes. Tubes of various cross sections, as well as open shapes (e.g., clips and brackets), can be produced by braking.

Roll forming is normally used for circular cross sections. In roll forming, the plate is either formed around an internal mandrel or rolled by forcing with external rolls. Either constant cross-sectional or tapered tubes can be made this way.

Tubes of various cross sections and tapers can be made by pressing plates in specifically profiled punch and die sets. Completed straight or tapered tubes can also be pressed into a die set to form curved crossarms.

Members may be straightened or cambered by mechanical means or by carefully supervised application of a limited amount of localized heat. The temperature of heated areas as measured by approved methods should not exceed 1,100 °F (593 °C) for quenched and tempered steel or 1,200 °F (649 °C) for other steels.

There are limits on the tightness of a bend that can be made in a piece of steel. They are usually expressed as a ratio of the inside radius of the bend to the material thickness. Some of the factors that affect the limits for a particular plate are the angle and the length of the bend to be made, the mechanical properties and direction of the final rolling of the plate, the preparation of the free edges at the bend line, and the temperature of the metal. Separation of the steel can occur during forming because of the method used, radii, temperature, and/or imperfections in the material.

Hot bending allows smaller bend radii to be used than does cold bending. Improper temperature during bending can adversely affect the material. Proper temperatures can be obtained from the steel producer, testing, or various AISC publications.

C7.2.2.3 Holes. Typically, holes may be punched in steel when the relationship between the material thickness and the hole diameter meets the recommendations of Section C6.2.6. If the steel is to be galvanized, precautions against steel embrittlement listed in ASTM A143 should be followed.

Holes can be drilled in plates of any thickness. Care should be taken to maintain accuracy when drilling stacks of plates. Holes can be torch cut. The torch should be machine-guided, and care should be taken that the cut edges are reasonably smooth and suitable for the stresses transmitted to them.

C7.2.2.4 Identification. Piece marks are typically at least 0.50 in. (13 mm) in height. They are usually made either by stamping or by a weld deposit before any finish application.

C7.2.3 Welding. Welding may be performed using many different processes and procedures but should be in conformance with AWS D1.1. Shielded metal arc welding (SMAW), flux cored arc welding (FCAW), gas metal arc welding (GMAW), submerged arc welding (SAW), and resistance seam welding (RSEW) are the weld processes most commonly used.

Workmanship and quality of welds are critical to the integrity of transmission pole structures. These structures often have large base and/or flange plate to shaft thickness ratios; thus, it is important that preheating be performed correctly. Improper preheating can result in significant base/flange plate distortion and premature weld failures.

If field welding is required, it should conform to the requirements of shop welding, except that the weld process may vary.

CHAPTER C8
TESTING

C8.1 INTRODUCTION

In a traditional proof test, the test setup is made to conform to the design conditions, that is, only static loads are applied; the prototype has level, well-designed foundations; and the resultants at the load points are the same as in the design model. This type of test verifies the adequacy of the main components of the prototype 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 actual stress distribution of unique configurations, fit-up verification, performance of the structure in a deflected position, and other benefits. The test cannot confirm how the structure will react in the transmission line where the loads may be more dynamic, the foundations may be less than ideal, and there is some restraint from intact wires at load points.

A full-scale structure test is commonly performed with the prototype structure erected in its natural upright/vertical position, as it will be when field erected. When tested in this position, the structure will be subjected to the full effects of loading, including all P-delta effects, that were considered in its design.

Horizontal tests are sometimes used for testing individual components, but can also be used for full-scale prototype testing, provided the effects of gravity are appropriately accounted for. Horizontal testing of full-scale structures is primarily reserved for free-standing, single poles for the simple purpose of assessing the ability of the poles to withstand their maximum design stress. In such cases, the bottom section of the pole is typically secured near the base plate to an uplift foundation while the other end rests on a compression pad. Because actual design loads (i.e., axial, shear, and torsional loads) cannot be directly applied to a structure in this test configuration, the test loads and their points of application on the prototype structure will need to be calculated to ensure that all critical points along the pole shaft are subjected to the same maximum stresses for which they were designed.

C8.2 FOUNDATIONS

The type, rigidity, strength, and moment reactions of the actual attachments of a prototype to a test bed may affect the ability of the members to resist the applied loads. Therefore, the restraint conditions of the test foundation should be as close as possible to the expected design conditions.

Pole structures that are designed to be attached to foundations through anchor bolts should be tested on an anchor bolt arrangement attached to the test facility foundation in a manner that best simulates the design conditions. Leveling nuts, if used, should be set at approximately the same height that is used during line construction.

Normally, for direct-embedded structures, only the aboveground portion of the structure is tested by having all of the controlling design load cases applied. The prototype should be furnished with special base sections that can be attached to the test facility foundation through anchor bolts or by direct welding. If the structure has been designed for a point of fixity below groundline, the length of the main shaft or shafts should be extended to ensure that the point of maximum moment on the shaft is tested.

Because soil properties at a test facility probably do not match the properties of the soil on the transmission line, foundation tests, when required, should be done at the line site. For most structures, a simplified, one-load case test that develops the critical overturning moment and associated vertical load is sufficient.

C8.3 MATERIAL

All prototype material should conform to the minimum requirements of the material specified in the design. Because of the alloying methods and rolling practices used by the steel mills, all steel plates have yield strength variations. Although desirable, it is impractical to limit the maximum yield strengths of the materials used for the fabrication of a prototype. Test loads should not be increased as a means of accounting for material yield strengths that are in excess of the specified minimum values.

C8.4 FABRICATION

Normally, the finish is not applied to the prototype for the test unless specified by the owner. Nonstructural hardware attachments, such as ladders or step bolts, are not normally installed on the prototype.

C8.5 STRAIN MEASUREMENTS

Stress determination methods, primarily strain gauging, may be used to monitor the loads in individual members during testing. Comparison of the measured unit stress is useful in validating the proof test and refining analysis methods. Care should be exercised when instrumenting with strain gauges, as to both location and number, to ensure valid correlation with design stress levels.

C8.6 ASSEMBLY AND ERECTION

It may be desirable to specify detailed methods or sequences for erecting the prototype to prove the acceptability of the proposed field erection method. Pick-up points designed into the structure should be used as part of the test procedure.

After the prototype has been assembled, erected, and rigged for testing, the owner, structure designer, or test engineer should review the testing arrangement for compliance with the contract specifications.

Safety guys or other safety features may be loosely attached to the prototype. They are used to minimize consequential damage to the prototype or to the testing equipment in the event of a failure, especially if a test-to-destruction is specified. Load effects of the safety guys should be minimized during the test.

C8.7 TEST LOADS

Destruction is defined as the inability of the prototype to withstand the application of additional load. The destruction load, when it occurs, should be referenced as a percentage of the maximum structure test load. Factored loads are typically applied when testing a structure to assess its load-carrying capacity (i.e., its ability to withstand its maximum design loads). However, when testing to assess structure deflections, load factors equal to 1.0 are typically used.

C8.8 LOAD APPLICATION

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 does not support compression, it is recommended that wire rope be used for simulated insulators in the test. If strut or post insulators are planned for the structures, members that simulate the insulators should be used.

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

C8.9 LOADING PROCEDURE

It is customary that load cases having the least influence on the results of successive tests be tested first. Another consideration should be to simplify the operations necessary to carry out the test program. Normally loads are applied to 50%, 75%, 90%, and 100% of the factored design loads. The 100% load for each load case should be held for 5 min. Unloading should be controlled to avoid possible damage or overload to the prototype.

Loads should be reduced to a minimum level between load cases except for noncritical load cases, where, with the structure designer's approval, the loads may be adjusted as required for the next load case.

C8.10 LOAD MEASUREMENT

All applied loads should be measured as close to the point of attachment to the prototype as practical. The effects of pulley friction should be minimized. Load measurement by monitoring the load in a single part of a multipart block and tackle should be avoided.

C8.11 DEFLECTIONS

Points to be monitored should be selected to verify the deflections predicted by the design analysis.

Also, it should be realized that measured and calculated deflections might not agree. There are two main reasons for this: First, the calculations for deflections usually do not include the effect of deflection and distortion within the joints and connections. Second, the actual stresses reached during testing often approach the yield strength of the material, which, by definition, includes some permanent set in the steel.

Upon release of test loads after a critical test case, a prototype normally does not return fully to its undeflected starting position.

C8.12 FAILURES

The prototype is normally considered acceptable if it is able to support the specified loads with no structural failure of prototype members or parts and does not exceed the specified deflection limits. If a retest is required, failed members affected by consequential damage should be replaced. The load case that caused the failure should then be repeated. After completion of testing, the prototype should be dismantled and inspected.

C8.13 POST-TEST INSPECTION

The owner should indicate any special inspection requirements in the contract documents.

C8.14 DISPOSITION OF PROTOTYPE

An undamaged prototype is usually accepted for use in the transmission line after all components are inspected in accordance with the test procedure and are found to be structurally sound and within the fabrication tolerances.

CHAPTER C9

STRUCTURAL MEMBERS AND CONNECTIONS USED IN FOUNDATIONS

C9.1 INTRODUCTION

The material in Chapter 9 covers structural members and connections normally supplied by the fabricator. Numerous factors enter into the selection of a foundation type, including but not limited to the following:

- Geotechnical considerations,
- Foundation loading,
- Base size of structure,
- Rotation and deflection limitations,
- Economics,
- Aesthetics,
- Contractor experience,
- Available equipment,
- Site accessibility, and
- Environmental concerns.

Many different foundation systems have been developed to meet the variety of steel pole support needs. The foundation types addressed in this standard are drilled shaft foundation with anchor bolts (Figure C9-1); direct-embedded foundation (Figure C9-2); embedded casing foundation (Figure C9-3); and base plate vibratory caisson foundation (Figure C9-4). Other types of foundations (spread, pile, rock anchor foundations, among others) may be considered for specific applications and should be designed according to an appropriate engineering standard.

C9.2 GENERAL CONSIDERATIONS

In selecting the type of foundation, the owner should consider the type of structure, importance of the structure, allowable foundation movement or rotation, and geotechnical conditions.

Foundation type, point of design fixity, rotation, deflection, and reveal have a significant effect on structure loading and cost and are of particular importance to the structure designer.

The following should be considered in foundation design:

- Soil characteristics: Adequate geotechnical exploration is necessary to determine the best type and size of foundation for the given soil or rock characteristics. The geotechnical report developed from the exploration should include design criteria for assessing the axial and lateral capacity as well as displacements. Chemical tests also are appropriate if corrosion is a problem. The cost of additional exploration should be compared against a more conservative foundation design. The savings realized from optimally designed foundations can more than offset the cost of the geotechnical evaluation.
- Displacements: Foundation displacement and rotation should be considered in the line and structure design. Excessive displacement or rotation can create an undesirable appearance, cause load redistribution, affect conductor sag adversely, and require future plumbing or adjustment of the structure.
- Loads: All foundation loads are to be supplied by the structure designer. Foundation designs should provide for all dead and live loads, horizontal shear, overturning moment, torsion, uplift, or compression loads. The owner has the responsibility for selecting minimum factors of safety used in the foundation design. Care should be taken to avoid combining load factors used in the structure design and additional factors of safety applied in the geotechnical analysis.
- Corrosion protection: Embedded steel shafts and/or casings may require special protection. In some cases, it may be

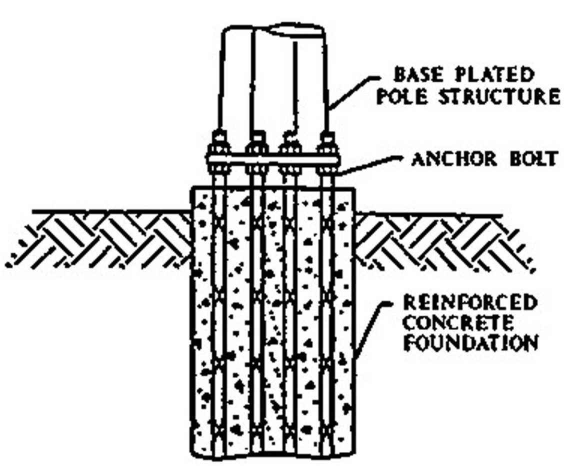

Figure C9-1. Drilled shaft foundation with anchor bolts.

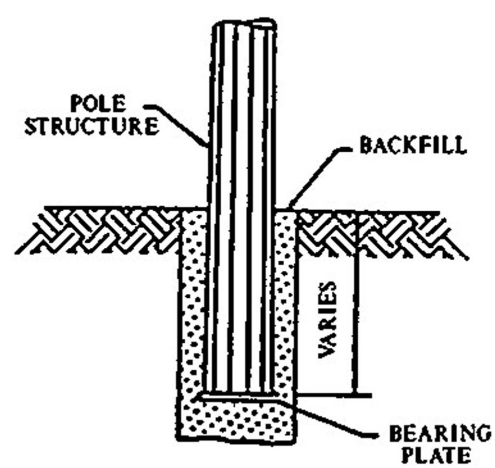

Figure C9-2. Direct-embedded pole.