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48-19

Design of Steel Transmission Pole Structures

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PREFACE

This standard includes informative commentaries and appendixes that are not a mandatory part of the standard. The commentary is numbered to correspond to the sections of the standard to which they refer. The appendixes provide additional information that is not necessarily related to specific sections of the standard.

Before the initial publication of this standard in 2005, most electric transmission design professionals used ASCE's Engineering Manual and Report on Engineering Practice No. 72, *Design of Steel Transmission Pole Structures*, as their primary reference for providing a uniform basis for designing, fabricating, testing, assembling, and erecting steel transmission pole structures. The second edition of MOP 72 served as the primary resource document for the development of the original version of Standard ASCE/SEI 48-05. The first revision was Standard ASCE/SEI 48-11. This document is the second revision to this standard and is intended to replace ASCE 48-11 in its entirety.

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CHAPTER 1
SCOPE

Design of Steel Transmission Pole Structures specifies requirements for the design, fabrication, testing, assembly, and erection of cold-formed tubular members and connections for steel electrical transmission pole structures. Structure components (members, connections, guys) are selected to resist factored design loads at stresses approaching yielding, buckling, fracture, or any other limiting condition specified in this standard. Communication and railroad electric traction structures are not included within the scope of this standard.

Overhead line structures, including steel transmission poles, are inherently different from buildings and most other structures. Overhead lines are a complex structural system composed of foundations, structures, conductors, and cables. A significant portion of the load on the structure is imposed by the conductors and other cables strung between the structures and is concentrated at specific points on the structure. The conductors and cables are subjected to extremes of temperature, wind, ice, and other phenomena not typical to other structures such as buildings and bridges. The tensions in these conductors and cables change with temperature, wind, and ice loads, and the temperatures of the conductors can be significantly higher than ambient temperature. Another significant difference between overhead line structures and other types of structures is that they are designed for a different life cycle than buildings, bridges, and other occupied structures. The structure designer shall clearly understand these differences and the unique behavior of overhead lines.

One advantage of this cable system is the combination of its ability to absorb dynamic energy and the very low relative mass of the steel pole structures. As a result, the design of overhead lines, including steel transmission poles, is not normally controlled by seismic loads. In general, large structure deflections are acceptable provided such deflections are accounted for in the analysis of the structures and their effects on the positions of the connected conductors and cables are considered.

Units of measurement herein are expressed first in US customary units followed by the Système International (SI) units in parentheses. Formulas are based on US customary units, and thus, some formulas require a conversion factor to use SI units. The appropriate conversion factor is given after each formula.

CHAPTER 2
APPLICABLE DOCUMENTS

The following standards are referenced in this standard:

AISC (American Institute of Steel Construction):
ANSI/AISC 360-05 *Specification for Structural Steel Buildings*

ANSI (American National Standards Institute):
ANSI O5.1-2008 *Wood Poles—Specifications and Dimensions*

ASCE:
ASCE 10-97 *Design of Latticed Steel Transmission Structures*
ASCE MOP 74 *Guidelines for Electrical Transmission Line Structural Loading*
ASCE MOP 91 *Design of Guyed Electrical Transmission Structures*

ASTM (ASTM International):
A6/A6M-09 *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*
A36/A36M-08 *Standard Specification for Carbon Structural Steel*
A123/A123M-09 *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*
A143/A143M-07 *Standard Practice for Safeguarding Against Embrittlement of Hot-Dip Galvanized Structural Steel Products and Procedure for Detecting Embrittlement*
A153/A153M-09 *Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware*
A193/A193M-09 *Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service*
A307-07b *Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength*
A325-09 *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*
A325M-09 *Standard Specification for Structural Bolts, Steel Heat Treated 830 MPa Minimum Tensile Strength [Metric]*
A354-07a *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*
A370-10 *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*
A385-09 *Standard Practice for Providing High-Quality Zinc Coatings (Hot-Dip)*
A394-08 *Standard Specification for Steel Transmission Tower Bolts, Zinc-Coated and Bare*
A449-07b *Standard Specification for Quenched and Tempered Steel Bolts and Studs*
A475-03 *Standard Specification for Zinc-Coated Steel Wire Strand*
A490-08b *Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength*
A490M-09 *Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)*
A529/A529M-05 *Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality*
A563-07a *Standard Specification for Carbon and Alloy Steel Nuts*
A563M-07 *Standard Specification for Carbon and Alloy Steel Nuts [Metric]*
A568/A568M-09 *Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled General Requirements for*
A572/A572M-07 *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*
A588/A588M-05 *Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4 in. [100 mm] Thick*
A595/A595M-06 *Standard Specification for Steel Tubes, Low-Carbon, Tapered for Structural Use*
A606/A606M-09 *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance*
A615/A615M-09 *Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement*
A633/A633M-01 (2006) *Standard Specification for Normalized High-Strength Low-Alloy Structural Steel Plates*
A673/A673M-07 *Standard Specification for Sampling Procedure for Impact Testing of Structural Steel*
A780/A780M-09 *Standard Practice for Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings*
A871/A871M-03 (2007) *Standard Specification for High-Strength Low-Alloy Structural Steel Plate with Atmospheric Corrosion Resistance*
A1011/A1011M-10 *Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength*
B416-98 (2007) *Standard Specification for Concentric-Lay-Stranded Aluminum-Clad Steel Conductors*
E165-09 *Standard Test Method for Liquid Penetrant Examination*
E709-08 *Standard Guide for Magnetic Particle Examination*

AWS (American Welding Society):
AWS B1.10 1999 *Guide for Nondestructive Inspection of Welds*
AWS C2.18-93R *Guide for the Protection of Steel with Thermal Sprayed Coatings of Aluminum and Zinc and Their Alloys and Composites*
AWS D1.1/D1.1M-2008 *Structural Welding Code—Steel*

AWS QC1-2007 *Standard for AWS Certification of Welding Inspectors*

CSA (Canadian Standards Association):
CSA-22.3 *Canadian Electrical Code*

IEEE (Institute of Electrical and Electronics Engineers):
IEEE C2 (2002) *National Electrical Safety Code*
IEEE 524 *Guide for the Installation of Overhead Transmission Line Conductors*
IEEE 1307 *IEEE Standard for Fall Protection for Utility Work*

RCSC (Research Council on Structural Connections):
Specification for Structural Joints Using ASTM A325 or A490 Bolts

CHAPTER 3
DEFINITIONS

Aeolian Vibration: High-frequency, low-amplitude vibration generated by a low-velocity steady wind blowing across the conductor or structural member.

Assembly Documents: Drawings, specifications, and/or instructions prepared by the structure designer, fabricator, and owner for the purpose of providing the requirements for the assembly and erection of the structures.

Blast Cleaning: Cleaning and descaling of a steel object using peening action of shot, sand, or abrasive powder under high pressure.

Camber (or **Precamber**): Pole curvature, induced in fabrication, used to counteract predetermined pole deflection, such that the pole will appear straight under a specified load condition.

Circumferential Weld: A weld joint directionally perpendicular to the long axis of a structural member. Commonly used to join two closed-section shapes of a common diameter.

Complete Fusion: Fusion that has occurred over the entire base metal surface intended for welding and between all adjoining weld beads.

Complete Joint Penetration: A penetration by weld metal for the full thickness of the base metal in a joint with a groove weld.

Contract Specifications: Written requirements prepared by the owner that are to be followed in the design, manufacture, and testing of the steel transmission structures. Items usually included in the contract specifications are structure design/framing drawings, structure loading requirements, installation considerations, fabrication tolerances, testing requirements, structure quantities, structure finish, and delivery requirements.

Corrosion Collar: *See* **Ground Sleeve.**

Design Stress: The maximum permitted stress in a given member.

Direct-Embedded Pole: A structure in which the lower section is extended below groundline a predetermined distance.

Edge Distance: The distance between the center of a connection hole and the edge of the plate or member.

Fabricator: The party responsible for the fabrication of the steel pole structure.

Factored Design Loads: Unfactored loads multiplied by a specified load factor to establish the design load on a structure.

Faying Surfaces: The contacting surfaces of two joined members.

Fusion: The melting together of filler metal and base metal (substrate), or of base metal only, to produce a weld.

Galloping Vibration: Low-frequency, large-amplitude vibration that occurs when a steady wind of moderate velocity blows over a conductor covered by a layer of ice deposited by freezing rain, mist, or sleet.

Ground Sleeve (or **Corrosion Collar**): A steel jacket that encapsulates a portion of a direct-embedded pole immediately above and below the groundline.

Lamellar Tearing: Separation in highly restrained base metal caused by through-thickness strains induced by shrinkage of the adjacent weld metal.

Line Designer: An agent of the owner who is responsible for the design of the proposed transmission line.

Load Factor (or **Overload Factor**): A multiplier used with the assumed loading condition or unfactored load to establish the factored design load.

Local Buckling: Introduction of a series of waves or wrinkles in one or more elements of a column section or on the compressive side of a beam section because of the inability of the section to resist the compressive stress in its current geometric shape.

Loosely Bolted: Bolted connections in which the nuts are drawn into contact with the mating surface without being tightened with tools.

Overload Factor: *See* **Load Factor**.

Owner: The owner of the proposed transmission line or the owner's designated representative, who may be a consulting engineer, general contractor, or other entity.

Precamber: *See* **Camber**.

Pre-Engineered Steel Pole: A steel pole provided by a fabricator that is designed to support a predefined loading. Among such poles are those that have been designed to support the ANSI O5.1 wood class loads that have been proportioned using the appropriate wood and steel NESC Rule 250B Grade B wind load factors.

Rake: The amount of horizontal pole top displacement created by installing a pole tilted out of plumb. It is used to counteract predetermined pole deflection such that the pole will appear plumb under a specified load condition.

Security Load: A design load used to decrease the risk of a cascading-type line failure. Loads that could cause cascading could be weather-related or accident-related resulting from broken conductors, components, or failed structures.

Shield Wire: Wire installed above the conductors for lightning protection and fault current return. Other terms used are overhead ground wire, static wire, and optical ground wire (OPGW).

Shop Detail Drawings: Drawings that are usually prepared by the fabricator and that contain complete and detailed information necessary for the fabrication of the structure and components.

Slip Joint (or **Slip Splice**): A telescoping-type connection of two tapered tubular pole sections.

Snug-Tight: Tightness obtained manually through the full effort of a worker using an ordinary spud wrench or as obtained through a few impacts of an impact wrench.

Stability: The ability of a structure or member to support a given load without experiencing a sudden change in configuration.

Structure Designer: The party responsible for the design of the structure, who may be an agent of the owner or fabricator.

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

Through-Thickness Stress: Tensile stresses through the thickness of the plate that can cause failure parallel to the plate or tube surface.

T-Joint: A joint between two members located approximately at right angles to each other in the form of a letter T.

Truss Member: A member designed to carry only axial force.

CHAPTER 4
LOADING, GEOMETRY, AND ANALYSIS

4.1 INTRODUCTION

This chapter details the minimum basic information that the owner shall provide in a written specification to enable the structure designer to design the structure. This chapter also details the methods of analysis that shall be used by the structure designer to design the structure.

4.2 LOADING

4.2.1 Factored Design Loads. Factored design loads shall be determined by the owner and included in the design specification, drawings, or documents.

4.2.2 Loading Considerations. The development of factored design loads shall consider the following:

- Conductor and shield wire properties,
- Minimum legislated loads,
- Historical climatic conditions,
- Structure orientation,
- Construction and maintenance operations,
- Line security provisions, and
- Unique loading situations.

4.2.3 Load Expression. Factored design loads shall be specified by the owner and shall be expressed in the form of load trees or in tabular form. Factored design loads shall include the magnitude, direction, and point of application with respect to a single orthogonal coordinate system.

4.3 GEOMETRIC CONFIGURATIONS

4.3.1 Configuration Considerations. Tubular steel pole structures shall be designed with geometric configurations that are based on electrical, economic, aesthetic, and safety requirements specified by the owner.

4.3.2 Structure Types. Tubular steel pole structures shall be designed as either self-supporting or guyed structures as specified by the owner.

4.4 METHODS OF ANALYSIS

The structure designer shall use established principles of structural analysis to determine the forces and moments caused by the factored design loads.

4.4.1 Structural Analysis Methods. The structure designer shall use geometrically nonlinear elastic stress analysis methods. The influences of guy tensions are of critical importance in determining the behavior of guyed structures; therefore, guy pretensions used in the design of such guyed structures shall be conveyed by the structure designer in the assembly documents.

4.4.2 Analysis of Connections. The structure designer shall be responsible for the analysis of all connections. This analysis shall be substantiated by stress calculations or by test results.

4.5 ADDITIONAL CONSIDERATIONS

4.5.1 Structural Support. The owner shall specify the type and degree of support provided by foundations or guys that will be used with the installed structure. Additional requirements regarding foundations appear in Section 9.2.

4.5.2 Design Restrictions. The owner shall specify applicable design restrictions, including shipping length, shipping weight, diameter, taper, deflection, finish, and critical dimensions that ensure the required heights, spacing, and clearances. The shaft-to-shaft connection type, foundation type, and guy attachment and anchor location shall be specified if applicable.

4.5.3 Climbing and Maintenance Provisions. The owner shall specify the types and positions of climbing and maintenance apparatus. This includes information concerning ladder or step attachment devices, grounding connection provisions, and *hot line* maintenance equipment attachment details, where applicable.

4.5.4 Pre-Engineered Steel Poles (Wood Pole Equivalents). The term *wood pole equivalents* shall not be used to specify pre-engineered steel poles. Pre-engineered steel poles shall be selected in accordance with the requirements of this standard and therefore shall not be selected solely based on wood pole classification. The owner and/or line designer shall be responsible for determining the applicable loads and loading criteria, geometric configuration, type and degree of structural support, and any design restrictions, as well as any other required design or performance characteristics for pre-engineered steel poles. When a pre-engineered steel pole is specified without providing loads, the owner and/or line designer shall be responsible for determining that the pole and all other structural components and attachments are adequate for the intended use and loads.

CHAPTER 5
DESIGN OF MEMBERS

5.1 INTRODUCTION

The design stresses for members shall be based on ultimate strength methods using factored design loads.

5.2 MEMBERS

This section contains criteria for determining design stress levels in tubular members and in truss members. Ground sleeves shall not be considered as structural members in the design.

5.2.1 Material

5.2.1.1 Specifications. Material conforming to the following ASTM specifications are suitable for use under this standard:

- ASTM A36/A36M *Standard Specification for Carbon Structural Steel*
- ASTM A529/A529M *Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality*
- ASTM A572/A572M *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*
- ASTM A588/A588M *Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4-in. (100 mm) Thick*
- ASTM A595 *Standard Specification for Steel Tubes, Low-Carbon, Tapered for Structural Use*
- ASTM A606 *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled, and Cold-Rolled, with Improved Atmospheric Corrosion Resistance*
- ASTM A633/A633M *Standard Specification for Normalized High-Strength Low-Alloy Structural Steel Plates*
- ASTM A871/A871M *Standard Specification for High-Strength Low-Alloy Structural Steel Plate with Atmospheric Corrosion Resistance*
- ASTM A1011/A1011M *Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low Alloy, and High-Strength Low Alloy with Improved Formability*

This list of suitable steels does not exclude the use of other steels that conform to the chemical and mechanical properties of one of the listed specifications or other published specifications, which establish the properties and suitability of the materials. As a minimum, material shall meet the requirements of ASTM A6 or ASTM A568, as applicable.

5.2.1.2 Material Properties. The yield stress, F_y, and the tensile stress, F_u, shall be the specified minimum values determined according to the appropriate ASTM specification. The modulus of elasticity, E, for steel is defined to be 29,000 ksi (200 GPa).

5.2.1.3 Energy-Impact Properties. Impact properties in the longitudinal direction of all structural plate or coil materials shall be determined in accordance with the Charpy V-notch test described in ASTM A370 and, at a minimum, shall meet the requirements of 15 ft-lb (20 J) absorbed energy at a temperature of −20 °F (−29 °C). Absorbed energy requirements for subsize test specimens shall be in accordance with ASTM A370 and A673.

For all plate and coil materials of any thickness, heat-lot testing shall be used unless specified differently by the owner.

5.2.2 Tension. The tensile stress shall not exceed either of the following:

$$\frac{P}{A_g} \le F_t \text{ where } F_t = F_y \tag{5.2-1}$$

or

$$\frac{P}{A_n} \le F_t \text{ where } F_t = 0.83F_u \tag{5.2-2}$$

where

P = Axial tension force on member,
A_g = Gross cross-sectional area,
F_t = Tensile stress permitted,
F_y = Specified minimum yield stress,
A_n = Net cross-sectional area, and
F_u = Specified minimum tensile stress.

5.2.3 Compression. Members subjected to compressive forces shall be checked for general stability and local buckling. The compressive stresses shall not exceed those permitted in the subsequent sections.

5.2.3.1 Truss Members. For truss members with a uniform closed cross section, the actual compressive stress, f_a, shall not exceed the compressive stress permitted, F_a, as determined by the following:

$$F_a = F_y\left[1-0.5\left(\frac{\frac{KL}{r}}{C_c}\right)^2\right] \text{ when } \frac{KL}{r} \le C_c \tag{5.2-3}$$

$$F_a = \frac{\pi^2 E}{(\frac{KL}{r})^2} \text{ when } \frac{KL}{r} > C_c \tag{5.2-4}$$

$$C_c = \pi\sqrt{\frac{2E}{F_y}} \tag{5.2-5}$$

where

F_a = Compressive stress permitted,
F_y = Specified minimum yield stress,

K = Effective length factor,
L = Unbraced length,
C_c = Column slenderness ratio, and
E = Modulus of elasticity.

KL/r is the largest slenderness ratio of any unbraced segment.
Truss members made of angles shall be designed in accordance with Section 3.7 of ASCE 10.

5.2.3.2 ***Beam Members.*** The limiting values of w/t and D_o/t specified in this section may be exceeded without requiring a reduction in extreme fiber stress if local buckling stability is demonstrated by an adequate program of tests.

5.2.3.2.1 Regular Polygonal Members. For formed, regular polygonal tubular members, the compressive stress, $P/A + Mc/I$, on the extreme fiber shall not exceed the following:

- Octagonal (8-sided), hexagonal (6-sided), or rectangular (4-sided) members (bend angle ≥ 45°)

$$F_a = F_y \quad \text{when} \quad \frac{w}{t}\sqrt{\frac{F_y}{E}} \leq 1.53 \tag{5.2-6}$$

$$F_a = 1.42F_y\left(1.0 - 0.194\frac{w}{t}\sqrt{\frac{F_y}{E}}\right) \quad \text{when} \quad 1.53 < \frac{w}{t}\sqrt{\frac{F_y}{E}} \leq 2.06 \tag{5.2-7}$$

- Dodecagonal (12-sided) members (bend angle = 30°)

$$F_a = F_y \quad \text{when} \quad \frac{w}{t}\sqrt{\frac{F_y}{E}} \leq 1.41 \tag{5.2-8}$$

$$F_a = 1.45F_y\left(1.0 - 0.220\frac{w}{t}\sqrt{\frac{F_y}{E}}\right) \quad \text{when} \quad 1.41 < \frac{w}{t}\sqrt{\frac{F_y}{E}} \leq 2.20 \tag{5.2-9}$$

- Hexadecagonal (16-sided) members (bend angle = 22.5°)

$$F_a = F_y \quad \text{when} \quad \frac{w}{t}\sqrt{\frac{F_y}{E}} \leq 1.26 \tag{5.2-10}$$

$$F_a = 1.42F_y\left(1.0 - 0.233\frac{w}{t}\sqrt{\frac{F_y}{E}}\right) \quad \text{when} \quad 1.26 < \frac{w}{t}\sqrt{\frac{F_y}{E}} \leq 2.42 \tag{5.2-11}$$

where

F_y = Specified minimum yield stress, ksi (MPa);
F_a = Compressive stress permitted, ksi (MPa);
E = Modulus of elasticity, 29×10^3 ksi (200×10^3 MPa);
w = Flat width of a side, in. (mm); and
t = Wall thickness, in. (mm).

In determining w, the actual inside bend radius shall be used unless it exceeds $4t$, in which case it shall be taken equal to $4t$. For sections with two or more plies, this criterion shall be satisfied for each ply.

Table 5-1. Compressive Stress Permitted Based on Bend Angle.

Bend Angle	f_a	Equation
≥45°	≤1 ksi (6.9 MPa)	(5.2-6), (5.2-7)
≥45°	>1 ksi (6.9 MPa)	(5.2-8), (5.2-9)
≥30° but <45°	N/A	(5.2-8), (5.2-9)
≥22.5° but <30°	N/A	(5.2-10), (5.2-11)
<22.5°	N/A	(5.2-12), (5.2-13), (5.2-14), (5.2-15), (5.2-16)

Note: N/A = not applicable.

Table 5-1 summarizes the equations that shall be used to determine the compressive stress permitted based on bend angle and axial stress.

5.2.3.2.2 Rectangular Members. Equations (5.2-6) and (5.2-7) shall be used for rectangular members. The flat width associated with each side shall be treated separately. If the axial stress, f_a, is greater than 1 ksi (6.9 MPa), Equations (5.2-8) and (5.2-9) shall be used.

5.2.3.2.3 Polygonal Elliptical Members. The bend angle and flat width associated with elliptical cross sections are not constant. The smallest bend angle associated with a particular flat width shall be used to determine the compressive stress permitted. See Table 5-1 to determine which equations shall be used based on this bend angle.

5.2.3.2.4 Round Members. For round members or regular polygonal members with more than 16 sides, the compressive stress shall be proportioned to satisfy the following equation:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \tag{5.2-12}$$

where

f_a = Compressive stress due to axial loads, ksi (MPa),
f_b = Compressive stress due to bending moments, ksi (MPa),
F_a = Compressive stress permitted, ksi (MPa), and
F_b = Bending stress permitted, ksi (MPa).

$$F_a = F_y \quad \text{when} \quad \frac{D_o}{t} \leq 0.131\frac{E}{F_y} \tag{5.2-13}$$

$$F_a = 0.75F_y + \frac{0.0328E}{\frac{D_o}{t}} \quad \text{when} \quad 0.131\frac{E}{F_y} < \frac{D_o}{t} \leq 0.414\frac{E}{F_y} \tag{5.2-14}$$

$$F_b = F_y \quad \text{when} \quad \frac{D_o}{t} \leq 0.207\frac{E}{F_y} \tag{5.2-15}$$

$$F_b = 0.70F_y + \frac{0.0621E}{\frac{D_o}{t}} \quad \text{when} \quad 0.207\frac{E}{F_y} < \frac{D_o}{t} \leq 0.414\frac{E}{F_y} \tag{5.2-16}$$

where

D_o = Outside diameter of the tubular section (flat-to-flat outside diameter for polygonal members) in. (mm);
t = Wall thickness in. (mm); and
E = Modulus of elasticity, 29×10^3 ksi (200×10^3 MPa).

5.2.4 Shear. The shear stress resulting from applied shear forces, torsional shear, or a combination of the two shall satisfy the following equation:

$$\frac{VQ}{\mathrm{Ib}} + \frac{Tc}{J} \le F_v \quad \text{when} \quad F_v = 0.58F_y \tag{5.2-17}$$

where

V = Shear force,
Q = Moment of section about the neutral axis,
I = Moment of inertia,
b = Twice the wall thickness ($2t$),
T = Torsional moment,
c = Distance from neutral axis to extreme fiber,
J = Torsional constant of cross section,
F_v = Shear stress permitted, and
F_y = Specified minimum yield stress.

5.2.5 Bending. The stress resulting from bending shall not exceed the following:

$$\frac{Mc}{I} \le F_y \tag{5.2-18}$$

$$\frac{Mc}{I} \le F_a \text{ for polygonal members} \tag{5.2-19a}$$

$$\frac{Mc}{I} \le F_b \text{ for round members} \tag{5.2-19b}$$

where

M = Bending moment,
c = Distance from neutral axis to extreme fiber,
I = Moment of inertia,
F_y = Specified minimum yield stress,
F_a = Compressive stress permitted for polygonal members, and
F_b = Bending stress permitted for round members.

5.2.6 Combined Stresses. For a polygonal member, the combined stress at any point on the cross section shall not exceed the following:

$$\left[\left(\frac{P}{A} + \frac{M_x c_y}{I_x} + \frac{M_y c_x}{I_y}\right)^2 + 3\left(\frac{VQ}{Ib} + \frac{Tc}{J}\right)^2\right]^{(1/2)} \le F_y \text{ or } F_a \tag{5.2-20}$$

For a round member, the combined stress at any point on the cross section shall not exceed the following:

$$\left[\left(\frac{P}{A} + \frac{M_x c_y}{I_x} + \frac{M_y c_x}{I_y}\right)^2 + 3\left(\frac{VQ}{Ib} + \frac{Tc}{J}\right)^2\right]^{(1/2)} \le F_y \text{ or } F_b \tag{5.2-21}$$

where

F_a = Compressive stress permitted by Section 5.2.3.2.1,
F_b = Bending stress permitted by Section 5.2.3.2.4,
F_y = Specified minimum yield stress,
P = Axial force on member,
A = Cross-sectional area,
M_x = Bending moment about the x–x axis,
M_y = Bending moment about the y–y axis,
I_x = Moment of inertia about the x–x axis,
I_y = Moment of inertia about the y–y axis,
c_x = Distance from the y–y axis to point where stress is checked,
c_y = Distance from the x–x axis to point where stress is checked,
V = Total resultant shear force,
Q = Moment of section about neutral axis,
I = Moment of inertia,
T = Torsional moment,
J = Torsional constant of cross section,
c = Distance from neutral axis to point where stress is checked, and
b = Twice the wall thickness ($2t$).

The bending stress (*Mc/I*) and shear stress portions of these equations shall be absolute values (i.e., always positive). The same equation shall be used to check tension and compression stresses. When checking tension, *P/A* is positive if the member is in tension and negative if the member is in compression. The converse is true when checking compression.

5.3 GUYS

5.3.1 Material Properties. The minimum rated breaking strength of guys shall be determined according to the appropriate ASTM specification or as specified by the owner. The modulus of elasticity, *E*, of a guy shall be as specified by the applicable ASTM specification or as specified by the owner. In the absence of a specified value, *E* shall be assumed to be 23,000 ksi (159 GPa).

5.3.2 Tension. The maximum design tension force in a guy shall not exceed the following:

$$P \le P_{\max} \quad \text{where} \quad P_{\max} = 0.65\,\mathrm{RBS} \tag{5.2-22}$$

where

P = Tension force in the guy,
$P_{\max}$ = Maximum tension force permitted in the guy, and
RBS = Minimum rated breaking strength of the guy.

5.4 TEST VERIFICATION

Design values other than those prescribed in this section are permitted, but they shall be substantiated by experimental or analytical investigations.

CHAPTER 6
DESIGN OF CONNECTIONS

6.1 INTRODUCTION

The design stresses for connections shall be based on ultimate strength methods using factored design loads.

6.2 BOLTED AND PINNED CONNECTIONS

For bolted connections, these provisions shall pertain to holes with diameters a maximum of 0.125 in. (3 mm) larger than the nominal bolt diameter (except for anchor bolt holes). For pinned connections, the ratio of the diameter of the hole to the diameter of the pin shall be less than 2.

6.2.1 Material. Materials conforming to the following standard specifications are suitable for use under this standard:

ASTM A307 *Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength*
ASTM A325 *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*
ASTM A325M *Standard Specification for Structural Bolts, Steel Heat Treated, 830 MPa Minimum Tensile Strength [Metric]*
ASTM A354 *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*
ASTM A394 *Standard Specification for Steel Transmission Tower Bolts, Zinc-Coated and Bare*
ASTM A449 *Standard Specification for Quenched and Tempered Steel Bolts and Studs*
ASTM A490 *Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength*
ASTM A490M *Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric]*
ASTM A563 *Standard Specification for Carbon and Alloy Steel Nuts*
ASTM A563M *Standard Specification for Carbon and Alloy Steel Nuts [Metric]*

This listing of suitable steels does not exclude the use of other steels that conform to the chemical and mechanical properties of one of the listed specifications or other published specifications, which establish the properties and suitability of the material.

6.2.2 Shear Stress in Bearing Connections. The shear stress for bolts and pins shall not exceed the following:

$$\frac{V}{A_g} \le F_v \qquad (6.2\text{-}1)$$

where

V = Shear force (bolt or pin),
A_g = Gross cross-sectional area of the shank (bolt or pin),
F_v = Shear stress permitted (bolt or pin),
F_v = 0.45 F_u when threads are excluded from the shear plane,
F_v = 0.35 F_u when shear plane passes through the threads, and
F_u = Specified minimum tensile stress (bolt or pin).

6.2.3 Bolts Subject to Tension. Bolts shall be proportioned so that the sum of the tensile stresses caused by the applied external load and any tensile stress resulting from prying action does not exceed the permitted tensile stress, F_t, as follows:

$$\frac{T_s}{A_s} = F_t \qquad (6.2\text{-}2)$$

where F_t = 0.75 F_u.

The stress area, A_s, is given by

$$A_s(\text{in.}^2) = \frac{\pi}{4}\left(d - \frac{0.9743}{n}\right)^2$$

or

$$A_s(\text{mm}^2) = \frac{\pi}{4}(d - 0.9382p)^2 \qquad (6.2\text{-}3)$$

where

T_s = Bolt tensile force,
d = Nominal diameter of the bolt,
n = Number of threads per inch, and
p = Pitch, mm per thread.

6.2.4 Bolts Subject to Combined Shear and Tension. For bolts subject to combined shear and tension, the permitted axial tensile stress in conjunction with shear stress, $F_{t(v)}$, shall be

$$F_{t(v)} = F_t\sqrt{1 - \left(\frac{f_v}{F_v}\right)^2} \qquad (6.2\text{-}4)$$

where

F_v = Shear stress permitted as defined in Section 6.2.2,
F_t = Tensile stress permitted as defined in Section 6.2.3, and
f_v = Shear stress on effective area.

The combined tensile and shear stresses shall be taken at the same cross section in the bolt.

6.2.5 Bearing Stress in Bolted Connections. The maximum bearing stress shall satisfy the following condition:

$$f_{br} = \frac{P}{dt} \leq 1.9F_u \qquad (6.2\text{-}5)$$

where

f_{br} = Bearing stress,
P = Force transmitted by the bolt,
d = Nominal diameter of the bolt,
t = Member thickness, and
F_u = Specified minimum tensile stress of the member.

6.2.6 Minimum Edge Distances and Bolt Spacing for Bolted Connections. Minimum edge distances shall satisfy the following conditions:

$$L_e = 1.3d \qquad (6.2\text{-}6)$$

$$L_e = t + \frac{d}{2} \qquad (6.2\text{-}7)$$

In addition, when $P > 0.24(F_u)(t)(d)$

$$L_c = \left[\frac{P}{0.96F_u t} - \frac{d}{4}\right] \qquad (6.2\text{-}8)$$

and minimum bolt spacing shall satisfy the following condition:

$$s \geq 2.67d \qquad (6.2\text{-}9)$$

where

P = Force transmitted by the bolt;
L_e = Minimum edge distance, parallel to the load, from the center of the hole to the edge of the member;
d = Nominal diameter of the bolt;
t = Member thickness;
F_u = Specified minimum tensile stress of the member;
L_c = Minimum clear distance, parallel to load, from the edge of the hole to the edge of an adjacent hole or edge of the member; and
s = Minimum center-to-center spacing between bolts.

The edge distance requirements of Equations (6.2-6) and (6.2-7) do not apply to base plates or flange plates that are detailed such that the nuts cannot extend over the edge of the plate. In addition, Equation (6.2-7) only applies to punched holes.

6.2.7 Bearing Stress in Pinned Connections. The maximum bearing stress for through-bolts, insulators, or guy shackle attachments bearing on connection plates or pole walls shall satisfy the following equation:

$$f_{br} = \frac{P}{dt} \leq 1.65F_y \qquad (6.2\text{-}10)$$

where

f_{br} = Bearing stress,
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.

6.2.8 Minimum Edge Distances for Pinned Connections. In addition to the edge distances specified in Section 6.2.6, the minimum edge distance for pinned connections shall also satisfy the following condition:

$$L_s = \frac{1}{2}\left[\frac{P}{\gamma F_u t} + d_h + x\right] \qquad (6.2\text{-}11)$$

where

L_s = Minimum edge distance, perpendicular to the load, from the center of the hole to the edge of the member;
P = Force transmitted by the pin;
F_u = Specified minimum tensile stress of the member;
t = Member thickness;
d_h = Diameter of the attachment hole;
x = 1/16 in. (2 mm); and
γ = 0.75 when hole diameter (d_h) is $\leq$ pin diameter plus 1/2 in. (13 mm), and 0.65 when hole diameter (d_h) is $>$ pin diameter plus 1/2 in. (13 mm).

6.2.9 Connection Elements and Members. In addition to the foregoing requirements, connection elements and the affected elements of members shall be proportioned to limit stresses to the following:

- Tension yielding on gross area: $0.90(F_y)$,
- Shear yielding on gross area: $0.60(F_y)$,
- Tension rupture on net area: $0.75(F_u)$, and
- Shear rupture on net area: $0.45(F_u)$.

The net area of a connection plate shall not be considered larger than 85% of the gross area.

6.3 WELDED CONNECTIONS

6.3.1 Material Properties. The nominal tensile strength of weld metals shall be based on the minimum values as established in AWS D1.1. Weld material shall be compatible with the base material as specified in AWS D1.1. Welding electrodes shall meet the same Charpy impact requirements as the base material.

6.3.2 Effective Area. Except for plug and slot welds, the effective area of a weld joint shall be equal to the effective length of the weld times the effective throat thickness. For plug and slot welds, the effective area shall be considered to be the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

The effective length of a groove weld shall be equal to the width of the connected part. The effective throat of a complete penetration groove weld shall be equal to the thickness of the thinner connected part. The effective throat thickness of partial penetration groove welds is listed in Table 6-1. The effective throat thickness for flare groove welds is listed in Table 6-2.

Except for welds in holes and slots, the effective length of a fillet weld shall be the overall length of a full-size fillet, including returns. For fillet welds in holes and slots, the effective length shall be the length of the centerline of the weld through the center of the plane through the throat. In the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

The effective throat thickness of a fillet weld shall be the shortest distance from the root to the face of the diagrammatic weld. For fillets made by the submerged arc process, the effective throat shall be equal to the leg size for 0.375 in. (9.5 mm) and smaller fillets and equal to the theoretical throat plus 0.11 in. (2.8 mm) for fillets larger than 0.375 in. (9.5 mm).

6.3.3 Design Stresses. Design stresses for welds shall conform to Tables 6-3 through 6-6.

Table 6-1. Effective Throat Thickness of Partial Penetration Groove Welds.

Welding Process	Welding Position	Included Angle at Root of Groove	Effective Throat Thickness
Shielded metal arc or submerged arc	All	<60° but ≥45° ≥60°	Depth of chamfer minus 1/8 in. (3.2 mm) Depth of chamfer
Gas metal arc or flux cored arc	All Horizontal or flat Vertical or overhead	≥60° <60° but ≥45° <60° but ≥45°	Depth of chamfer Depth of chamfer Depth of chamfer minus 1/8 in. (3.2 mm)
Electrogas	All	>60°	Depth of chamfer

Table 6-2. Effective Throat Thickness of Flare Groove Welds.

Type of Weld	Radius (*R*) of Bar or Bend	Effective Throat Thickness
Flare-bevel groove	All	5/16*R*
Flare-V groove	All	1/2*R**

* Use 3/8*R* for gas metal arc welding (except short-circuiting transfer process) when $R > 1/2$ in. (12.7 mm).

When the base metals are of different strengths, the lowest grade of base metal shall be used for the weld design.

6.3.3.1 Through-Thickness Stress. Maximum design through-thickness stress shall be 36 ksi (248 MPa) for all grades of steel.

6.3.4 Circumferential Welded Splices. Complete penetration (100%) welds shall be used for sections joined by circumferential welds. Longitudinal welds within 3 in. (76 mm) of circumferential welds shall have complete fusion through the section thickness and complete joint penetration for processes using weld metal.

Table 6-3. Complete Penetration Groove Welds.

Type of Weld and Stress[a]	Design Stress	Required Weld Strength Level[b,c]
Tension normal to effective area	Same as base metal	*Matching* weld metal must be used
Compression normal to effective area	Same as base metal	Weld metal with a strength level equal to or less than matching weld metal may be used
Tension or compression parallel to axis of weld	Same as base metal	
Shear on effective area	0.50*X* nominal tensile strength of weld metal, except shear stress on base metal shall not exceed 0.58*X* yield stress of base metal	

[a] For definition of effective area, see Section 6.3.2.
[b] For matching weld metal, see Table 4.1, AWS D1.1.
[c] Weld metal one strength level higher than matching weld metal will be permitted.

Table 6-4. Fillet Welds.

Type of Weld and Stress[a]	Design Stress	Required Weld Strength Level[b,c]
Shear on effective area	0.50*X* nominal tensile strength of weld metal, except shear stress on base metal shall not exceed 0.58*X* yield stress of base metal	Weld metal with a strength level equal to or less than matching weld metal may be used
Tension or compression parallel to axis of weld	Same as base metal	

[a] For definition of effective area, see Section 6.3.2.
[b] For matching weld metal, see Table 4.1, AWS D1.1.
[c] Weld metal one strength level higher than matching weld metal will be permitted.

Table 6-5. Partial Penetration Groove Welds.

Type of Weld and Stress[a]	Design Stress	Required Weld Strength Level[b,c]
Compression normal to effective area	Same as base metal	Weld metal with a strength level equal to or less than matching weld metal may be used
Tension or compression parallel to axis of weld	Same as base metal	
Shear parallel to axis of weld	0.50X nominal tensile strength of weld metal, except shear stress on base metal shall not exceed 0.58X yield stress of base metal	
Tension normal to effective area	0.50X nominal tensile strength of weld metal, except shear stress on base metal shall not exceed 1.0X yield stress of base metal or 36 ksi (248 MPa), whichever is less	

[a] For definition of effective area, see Section 6.3.2.
[b] For matching weld metal, see Table 4.1, AWS D1.1.
[c] Weld metal one strength level higher than matching weld metal will be permitted.

Table 6-6. Plug and Slot Welds.

Type of Weld and Stress[a]	Design Stress	Required Weld Strength Level[b,c]
Shear parallel to faying surfaces (on effective area)	0.50X nominal tensile strength of weld metal, except shear stress on base metal shall not exceed 0.58X yield stress of base metal	Weld metal with a strength level equal to or less than "matching" weld metal may be used

[a] For definition of effective area, see Section 6.3.2.
[b] For matching weld metal, see Table 4.1, AWS D1.1.
[c] Weld metal one strength level higher than matching weld metal will be permitted.

6.3.5 Flange and Base Plate to Pole Shaft Welds. Flange and base plate to pole shaft welds shall be complete penetration (100%) groove welds with reinforcing fillet to satisfy the requirements for through-thickness stresses in the flange or base plates. Longitudinal welds within 3 in. (76 mm) of a flange plate or base plate weld shall have complete fusion through the section thickness and complete joint penetration for processes using weld metal.

6.3.6 T-Joints. T-joints shall satisfy the requirements for through-thickness stresses.

6.4 FIELD CONNECTIONS OF MEMBERS

6.4.1 Slip Joints. Slip joints shall be designed to resist the maximum forces and moments at the connection. Combined stresses shall be evaluated at the top of the nominal splice location using the section properties of the upper section and at the bottom of the nominal splice location using the section properties of the lower section. Taper above and below the slip joint shall be the same. To develop the ultimate capacity of the section, the joint shall have a minimum lap length of 1.5 times the maximum inside diameter across the flats of the outer section (nominal to be dictated by manufacturing tolerances to ensure the minimum design lap length and critical dimensions defined in the contract specifications are maintained).

Supplemental locking devices shall be used if relative movement of the joint is critical or if the joint might be subjected to uplift forces. In resisting uplift forces, locking devices shall be designed to resist 100% of the maximum uplift load. The outer section longitudinal seam weld in the area of the splice shall have complete fusion through the section thickness and complete joint penetration for processes using weld metal for a length equal to the maximum lap dimension.

6.4.2 Base and Flange Plate Connections. Flexural stress in the base or flange plate shall not exceed the specified minimum yield stress, F_y, of the plate material. Base and flange plate connections shall be designed to resist the maximum forces and moments at the connection. As a minimum, base and flange plate connections shall be designed to resist 50% of the moment capacity of the lowest strength tube.

6.5 TEST VERIFICATION

Design values other than those prescribed in this section are permitted, but they shall be substantiated by experimental or analytical investigations.

CHAPTER 7
DETAILING AND FABRICATION

7.1 DETAILING

7.1.1 Drawings. Drawings consist of erection and shop detail drawings. If shop detail drawings are provided by the owner, the owner shall be responsible for the completeness and accuracy of these drawings. If shop detail drawings are prepared by the fabricator, the fabricator shall be responsible for conveying the dimensions and details from the design and contract documents, the correctness of dimensional calculations performed in preparing the drawings, and the general fit-up of parts to be assembled in the field.

7.1.2 Drawing Review. Drawings shall be reviewed by the structure designer regarding the strength requirements of the design and compliance with the owner's specification. Drawings prepared by the fabricator shall be submitted to the owner for review.

7.1.3 Erection Drawings. Erection drawings shall show the complete field assembly of the structure, clearly indicating the positioning of the components, including fasteners. The identification markings for each component shall be indicated on the drawing. Fasteners shall be identified by grade, length, and diameter for bolts and grade and diameter for nuts and washers.

The erection drawings shall include a bill of material of all components for the structure, including the weight of each component. The erection drawings shall provide instructions for slip joint assembly, bolt tightening, and field welding where applicable.

7.1.4 Shop Detail Drawings. Shop detail drawings shall show all fabrication requirements, including material, dimensions, welding, shop-applied finish, and any specific processing requirements, including those of the contract and applicable codes. They shall be shown either by assembled section or piece by piece. The drawings shall indicate the piece mark of each component.

7.1.4.1 Material. Shop detail drawings shall specify member and connection materials, such as ASTM specification and grade designation.

7.1.4.2 Dimensions and Tolerances. Dimensioning practices, including tolerances, shall ensure compliance with clearance, appearance, strength, and assembly requirements. Proper mating of components detailed and supplied by one fabricator shall be the responsibility of that fabricator.

7.1.4.3 Welding. Welding shall be detailed in accordance with the AWS D1.1 code, including weld symbols. Only weld details that are prequalified or qualified in accordance with the AWS D1.1 code shall be used. Appropriate detailing practices shall be used to ensure that the required penetration is achievable.

7.1.4.4 Corrosion and Finish Considerations. When shop finish is specified in the contract documents, the requirements and specifications for surface preparation, painting, galvanizing, and/or metalizing requirements shall be shown on the drawings.

Details for weathering steel structures shall be designed to avoid uncoated pockets, crevices, and faying surfaces that can collect and retain water, damp debris, and moisture. Weld backing for unsealed weathering steel structures shall be weathering steel.

7.1.4.5 Other Requirements. Specific requirements and limitations of the contract documents and applicable codes shall be shown on the drawings.

7.2 FABRICATION

Fabrication shall be performed in compliance with the shop detail drawings. The fabricator shall be responsible for the means, methods, techniques, sequences, and procedures of fabrication. Safety precautions and programs for fabrication shall be the responsibility of the fabricator.

7.2.1 Material. The fabricator shall maintain a system, including records that will verify that the structural steel furnished meets the specified requirements. Certified test reports from the plate or coil mills and from suppliers of bolts, welding electrodes, and other materials shall constitute sufficient evidence of conformity. The fabricator shall accurately identify all material to ensure proper usage.

7.2.2 Material Preparation

7.2.2.1 Cutting. Parts shall be cut in accordance with AWS D1.1. Burrs or sharp notches that are detrimental to the structure or that pose a safety hazard shall be removed. Reentry cuts shall be rounded.

7.2.2.2 Forming. Care shall be taken during forming to prevent separation of the outer surface and reduction of the cross-sectional properties below those required by design. If separation occurs during bending, it shall be repaired in accordance with AWS D1.1. Loosening of mill scale shall not be considered a separation.

When hot bending is required, heating shall be done evenly over the entire bend area and shall be of sufficient temperature to minimize separation and necking down of the cross section. The temperature used in hot bending shall be such that the physical properties of the steel are not diminished.

7.2.2.3 Holes. Bolt holes shall have the correct shape and alignment in accordance with connection details, be free of burrs, and be clean cut without torn or ragged edges.

7.2.2.4 Identification. All components shall be clearly marked.

7.2.3 Welding. All welding shall be performed by welders, welding operators, and tackers qualified for the type of welding to be performed. All welder performance qualification records (WPQRs), weld procedure specifications (WPSs), and weld procedure qualification records (PQRs) shall be developed and properly maintained in accordance with the applicable requirements of AWS D1.1 *Structural Welding Code—Steel*. If Charpy impact values are being specified for the base metal on the project (the minimum requirements according to Section 5.2.1.3 in this standard or other values), the resulting weld metal, including the heat-affected zone (HAZ) area, shall exhibit comparable energy impact resistance as determined in the qualifying PQR and WPS prepared according to the requirements of AWS D1.1, Clause 4, Part D. Preheat and interpass temperatures shall be in accordance with AWS D1.1 or the steel manufacturer's recommendations. Longitudinal seam welds shall have 60% minimum penetration (except as specified in Sections 6.3.4, 6.3.5, and 6.4.1 in this standard).

CHAPTER 8
TESTING

8.1 INTRODUCTION

The owner shall specify in the contract specifications which structures or components of a structure will be tested. If a proof test of a structure or structure component is specified, the test shall be performed on a full-size prototype of the structure or structure component in accordance with the following sections.

The structure designer is responsible for ensuring that the structural design meets the loading, deflection, clearance, and other design specifications set forth in the contract. The structure designer shall approve the proposed procedure for prototype testing. Also, the structure designer or designated representative shall be present at all times during the testing sequence and approve each decision made during the process. The owner shall review the testing arrangement for compliance with the contract documents and the intent of the test. The number, location, direction, holding time, sequence, and increments of the test loads and the number, location, and direction of deflection readings shall be specified by the owner. The method of attaching the test loads to the prototype, applying the test loads, measuring and recording the test loads, locations and type of strain gauges, and measuring and recording the deflections shall be approved by the owner before testing begins.

8.2 FOUNDATIONS

The structure designer shall approve the support conditions used for testing.

8.3 MATERIAL

The prototype shall be made of material that is representative of the material that will be used in production. Mill test reports shall be available for each major component in the test structure. When mill test reports are unavailable, coupon tests are required. Coupon tests shall be performed in accordance with ASTM A370.

8.4 FABRICATION

Fabrication of the prototype shall be done in the same manner as for the production structure.

8.5 STRAIN MEASUREMENTS

The owner shall specify if any special strain determination methods are required for the prototype and identify those components to be strain gauged.

8.6 ASSEMBLY AND ERECTION

The method to be used for assembling the prototype shall be approved by the owner and structure designer prior to beginning assembly. The completed test structure shall be erected within the tolerances established in the contract specifications and assembly documents.

8.7 TEST LOADS

The owner and the structure designer shall collaborate to establish the testing requirements for a prototype structure. The requirements shall be specified in the contract specifications and shall include the load cases to be tested and whether the structure shall be tested to destruction. The test loads shall be the factored design loads.

All test equipment (e.g., load cells, load cables, hardware, and other rigging) shall be sized to accommodate the maximum expected loads that will be applied during the testing, and the test equipment capacities shall be reduced by the applicable equipment safety factors.

8.8 LOAD APPLICATION

Load lines shall be attached to the load points on the prototype in a manner that simulates the in-service application as close as possible. The attachment hardware for the test shall have the same degrees of freedom as the in-service hardware. Wind-on-structure loads shall be applied as concentrated loads at selected points on the structure. Load application shall consider the deflected position of the structure.

8.9 LOADING PROCEDURE

The sequence of load cases tested shall be specified by the structure designer and approved by the owner. Loading shall be stopped at preselected load levels to allow time for reading deflections and to permit observation of the test to check for signs of structural distress.

8.10 LOAD MEASUREMENT

Loads shall be measured through a verifiable arrangement of strain devices or by predetermined dead weights. Load-measuring devices shall be used in accordance with manufacturer's recommendations and calibrated before and after testing.

8.11 DEFLECTIONS

At the locations specified by the structure designer and approved by the owner, deflections of prototypes under load shall be measured and recorded by the test engineer. Deflection readings shall be made for the *before* and *off* load conditions as well as at each intermediate hold during loading. All deflections shall be referenced to common base readings taken before the first test loads are applied.

8.12 FAILURES

When failure occurs before application of 100% of the factored design loads, the cause of the failure, the corrective measures to be taken, and the need for a retest shall be determined by the structure designer and approved by the owner.

8.13 POST-TEST INSPECTION

The prototype shall be inspected after testing. Welds shall be inspected in accordance with the normal fabrication procedures. Visual inspection for any signs of structural damage shall be conducted by the test engineer.

8.14 DISPOSITION OF PROTOTYPE

The contract document shall state the disposition of the prototype after the test is completed.

8.15 REPORT

The testing organization shall furnish the number of copies of the test report specified in the contract specifications. The test report shall describe the test procedure, test results, and any remedial action that was taken during or after the course of testing.

The following information shall be included in the test report, as applicable:

- Designation and description of the prototype tested;
- Name of the owner;
- Name of the person or organization, if different from the owner, that specified the loading, electrical clearances, technical requirements, and general arrangement of the prototype;
- Name of the test engineer;
- Name of the structure designer;
- Name of the fabricator;
- Brief description and the location of the test facility;
- Names and affiliations of the test witnesses;
- Dates of each test-load case;
- Design and detail drawings of the prototype, including any changes made during the testing program;
- Rigging diagram with details of the attachment points to the prototype;
- Calibration records of the load-measuring devices;
- Loading diagram for each load case tested;
- Tabulation of deflections for each load case tested;
- In case of failure:
  - Photographs of prototype and all failed members components,
  - Loads at the time of failure,
  - A brief description of the failure,
  - The remedial actions taken,
  - The measured dimensions of the failed members, and
  - Test coupon reports of failed members.
- Structure assembly documentation;
- Photographs of the overall testing arrangement and rigging;
- Air temperature, wind speed and direction, precipitation, and any other pertinent meteorological data;
- Mill test reports submitted in accordance with Section 8.3;
- Foundation condition information; and
- Additional information as specified by the owner.

CHAPTER 9
STRUCTURAL MEMBERS AND CONNECTIONS USED IN FOUNDATIONS

9.1 INTRODUCTION

This chapter specifies design procedures for steel members and connections embedded in concrete or other backfill material. This chapter is not intended to serve as a foundation design guide. It is the responsibility of the owner to ensure adequate geotechnical design.

9.2 GENERAL CONSIDERATIONS

As applicable, the owner shall include the following in the specifications:

- Foundation type,
- Depth to point of foundation fixity,
- Design limit for foundation rotation or deflection,
- Foundation reveal,
- Coating requirements,
- Grounding requirements,
- Concrete or backfill material strength,
- Corrosion protection, and
- Other special requirements.

9.3 ANCHOR BOLTS

Anchor bolts shall be designed to transfer the tensile, compressive, and shear loads to the concrete by adequate embedment length or by the end connection. Impact properties in the longitudinal direction of all anchor bolt materials shall be determined in accordance with the Charpy V-notch test described in ASTM A370 and, at a minimum, shall meet the requirements of 15 ft-lb (20 J) absorbed energy at a temperature of –20 °F (–29 °C).

9.3.1 Bolts Subject to Tension. Anchor bolts subject to tension shall be designed in accordance with the provisions of Section 6.2.3.

9.3.2 Shear Stress. The shear stress for anchor bolts shall be determined in accordance with the provisions of Section 6.2.2.

9.3.3 Combined Shear and Tension. Anchor bolts subject to combined shear and tension shall be designed in accordance with the provisions of Section 6.2.4.

9.3.4 Development Length. The owner shall provide a minimum of 3 in. (76 mm) clear concrete cover. The development length for the threaded reinforcing bar used as anchor bolts shall be calculated as follows:

$$L_d = l_d \alpha \beta \gamma \tag{9.3-1}$$

where

L_d = minimum development length (embedment) of anchor bolt and

l_d = basic development length of anchor bolt.

The basic development length for the bolt shall be as follows:

- For bars up to and including #11 (35M), use the larger of

$$l_d = \frac{1.27\,\Gamma A_g F_y}{\sqrt{f'_c}} \tag{9.3-2}$$

or

$$l_d = 0.400 \Phi d F_y \tag{9.3-3}$$

- For No. 14 and No. 14J (45M) bars

$$l_d = \frac{2.69\,\Theta F_y}{\sqrt{f'_c}} \tag{9.3-4}$$

- For No. 18 and No. 18J (55M) bars

$$l_d = \frac{3.52\,\Theta F_y}{\sqrt{f'_c}} \tag{9.3-5}$$

where

A_g = Gross area of anchor bolt;

$A_{s(\text{req'd})}$ = Required tensile stress area of bolt;

F_y = Specified minimum yield stress of anchor bolt;

f'_c = Specified compressive strength of concrete;

d = Anchor bolt diameter;

Γ = 1.00 for F_y and f'_c in ksi and A_g in in.^{2}, and 0.0150 for F_y and f'_c in MPa and A_g in mm^2;

Φ = 1.00 for F_y in ksi and d in in., and 0.145 for F_y in MPa and d in mm;

Θ = 1.00 for F_y and f'_c in ksi and 9.67 for F_y and f'_c in MPa;

α = 1.0 if F_y = 60 ksi (414 MPa), or 1.2 if F_y = 75 ksi (517 MPa);

β = 0.8 if the bolt spacing is equal to or greater than 6 in. (152 mm) on center, or 1.0 if the bolt spacing is less than 6 in. (152 mm) on center; and

$\gamma = A_{s(\text{req'd})}/A_g$.

9.4 DIRECT-EMBEDDED POLES

The embedded section shall be designed to resist the overturning moment, shear, and axial loads. The length of the section of the pole below the groundline shall be determined using a lateral resistance approach. The owner shall be responsible for supplying the structure designer information regarding the embedment depth, allowable foundation rotation, and design point of fixity of the embedded section.

9.5 EMBEDDED CASINGS

The casing shall be designed to resist all design loads. The length of the embedded casing below the groundline shall be determined using a lateral resistance approach. The owner shall be responsible for supplying the structure designer information regarding the embedment depth, allowable foundation rotation, design point of fixity of the embedded section, vibratory installation forces, vibratory device attachment, and method of steel pole attachment.

9.6 TEST VERIFICATION

Design values other than those prescribed in this section may be used if substantiated by experimental or analytical investigations.

CHAPTER 10
QUALITY ASSURANCE/QUALITY CONTROL

10.1 INTRODUCTION

The contract between the owner and the fabricator shall state the responsibility of each party and the conditions under which the work will be accepted or rejected.

10.2 QUALITY ASSURANCE

Quality assurance (QA) is the responsibility of the owner. The specifying and implementation of QA requirements by the owner shall not relieve the fabricator of responsibility in producing a product in accordance with this standard.

10.2.1 Design and Drawings. The QA specification shall indicate the procedure for review of the design concept, design calculations, stress analyses, and the fabricator's drawings.

10.2.2 Material. The QA specification shall specify the requirements for review and agreement on the fabricator's material specifications, supply sources, material identification, storage, traceability procedures, and acceptance of certified mill test reports.

10.2.3 Welding. The QA specification shall include requirements for the review of, and agreement on, welders' qualification and certification procedures, including a list of welders certified for the work to be performed. The QA specification shall establish the process for acceptance of welding procedures for each type of weld and the method used to determine that the procedure will be performed with satisfactory quality control.

10.2.4 Nondestructive Testing. The QA specification shall indicate the requirements for acceptance of the type and procedure of nondestructive testing and inspection programs used during each step in the fabrication processes.

10.2.5 Tolerances. Fabrication tolerances shall be specified and agreed on by the owner and the fabricator.

10.2.6 Surface Coatings. When painting is specified, the paint system, procedures, and methods of application shall be agreed on by both the owner and the fabricator. The selected paint system shall be suitable for both the product and its intended exposure.

When galvanizing is specified, the procedure and facilities shall be agreed on by the owner and the fabricator. Inspection rights and privileges, procedures, and acceptance or rejection of galvanized steel material shall conform to ASTM A123, A143, A153, and A385 as applicable. When metalizing is specified, the procedure and facilities shall be in accordance with the coating vendor's recommendations and shall be acceptable to both the owner and the fabricator. When bare weathering steel is specified, the need for and degree of blast cleaning the steel shall be agreed on by the owner and the fabricator.

The QA specification shall establish inspection rights, privileges, and procedures for evaluating the surface coating.

10.2.7 Shipping. When receiving material, all products shall be inspected for shipping damage before accepting delivery. If damage is apparent, the owner shall immediately notify both the delivering carrier and the fabricator.

10.3 QUALITY CONTROL

Quality control (QC) is the responsibility of the fabricator. The fabricator shall have a QC program consisting of a written document that establishes the procedures and methods of operation that affect the quality of the work. The QC functions shall be clearly defined and available for review and approval by the owner. The QC program shall verify that the product meets the level of quality established by the fabricator's standards and the owner's specification. The QC program shall establish procedures for maintaining records of all pertinent information on all components.

10.3.1 Material. The QC program shall specify the review requirements of all materials that are used in the fabricating and coating of the complete structure, all mill test reports for material compliance, all material suppliers for their manufacturing procedures and QC programs, and all welding electrodes.

The fabricator shall maintain a system, including records, that will allow verification that the structural steel meets the specified requirements. Certified test reports from the plate mills and from the supplier of bolts, welding electrodes, and other materials in accordance with the governing specification shall constitute evidence of conformity. Certified tests by the fabricator or a testing laboratory shall also constitute evidence of conformity.

10.3.2 Visual Inspection. Structural components and 100% of all welds shall be visually inspected to determine conformance to drawings, procedures, overall workmanship, weld contour, weld size, and any other pertinent items.

10.3.3 Dimensional Inspection. Structural components shall be inspected for dimensional compliance to determine conformance with detail drawings and established tolerances. When applicable, the owner shall specify shop assembly requirements.

10.3.4 Surface Coating Inspection. The fabricator shall check product preparation and coating thickness to ensure that the minimum dry film thickness requirements of the coating specification are met. Visual inspection shall be performed to detect pinholes, cracking, and other undesirable characteristics.

10.3.5 Weld Inspection. Quality control supervisory personnel shall be certified welding inspectors (CWIs) in accordance with the provisions of AWS QC1. Weld inspection shall be performed in accordance with the requirements of Section 6, Inspection, Part C, of AWS D1.1. Personnel qualification for nondestructive weld testing shall be in accordance with Section 6.14.6.1 of AWS D1.1.

Complete penetration welds shall be 100% inspected by either ultrasonic (UT) or radiographic (RT) methods. Appropriate inspection practices shall be used to ensure that required penetration is achieved.

For galvanized members with large T-joint connections, such as base plates and flange plates, ultrasonic nondestructive weld testing shall be performed on 100% of all such joints, not only before, but also after galvanizing to ensure that no cracks have developed. Any indications found with this test shall be ground smooth and inspected with magnetic particle methods. Any positive indications after this inspection shall be repaired and reinspected, and the finish shall be repaired in accordance with the requirements of the appropriate ASTM standard.

10.3.6 Shipment and Storage. The QC program shall provide procedures that minimize the potential for damage, loss, or deterioration to the structure during storage and shipment.

CHAPTER 11
ASSEMBLY AND ERECTION

11.1 INTRODUCTION

This chapter covers the assembly and erection requirements for steel transmission pole structures. Additional information on assembly and erection can be found in Appendix E.

11.2 HANDLING

Poles, pole sections, crossarms, and other structural elements shall be lifted and stored in such a manner as to prevent excessive deflection, stresses, and buckling. Sections that are distorted, buckled, or permanently deflected shall not be installed. The owner shall contact the structure designer to verify acceptability of members with suspected damage.

11.3 SINGLE-POLE STRUCTURES

Assembly shall be in accordance with the erection drawings and requirements of the owner.

11.3.1 Slip Joints. Slip joints shall be jacked together in accordance with the assembly documents, taking care to ensure that the assembled joint falls within the minimum and maximum lap length specified. The jacking force used to assemble the sections shall be the minimum jacking force within the limits specified in the assembly documents. The equipment and method used must have sufficient capability to exceed the required jacking force. Fabrication and erection tolerances shall be the minimum design lap length required plus the manufacturing tolerances. Slip joint acceptability shall be dependent on satisfying the following requirements:

1. The minimum jacking force has been applied.
2. The 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.

In the event an assembled slip joint is not within the specified tolerance for overlap length, the actual overlap shall be reported to the structure designer for review of acceptability.

11.3.2 Bolted Flange Joints. Mating surfaces shall be cleaned of all foreign matter before assembly. The bolts shall be installed to snug-tight condition in a sequence to ensure the proper alignment of the two pole sections. Following snug tightening, the bolts shall be tensioned in accordance with the structure designer's recommendations using a similar tightening sequence. In the absence of specific tightening recommendations, the *turn-of-nut* method as described in Research Council on Structural Connections *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Section 8.2.1, shall be used for fastener tensioning.

11.3.3 Attachments to Pole Sections. Installation of crossarms and other attachments to the pole structure shall be in accordance with the assembly documents.

11.3.4 Erection of Assembled Structures. Assembled structures with slip joints shall have the slip joints temporarily secured before lifting to prevent the pole sections from separating during the erection operation.

11.4 FRAME-TYPE STRUCTURES

The assembly procedure for frame structures shall be in accordance with the assembly documents and the requirements listed in Section 11.3 except as modified by this section.

11.4.1 Slip Joints in Frames. Slip joint connections within framed structures shall be assembled in accordance with all of the requirements of Section 11.3.1. On multiple-leg structures, the assembled leg-length differential shall not exceed the practical adjustment length of the foundation system. Slip joint locking devices shall be used if specified in the assembly documents. Locking devices, if specified, shall be installed on each applicable slip joint.

11.4.2 Erection. Erection of frame structures shall be in accordance with the assembly documents, including the use of temporary braces or members as required to prevent damaging or overstressing members and connections during the installation procedure. All slip joints shall be restrained to prevent separation of the joint during structure erection.

11.4.3 Bolted Frame Connections. Bolted frame joints shall be assembled with fasteners loosely bolted to permit movement in the joint during installation of additional framing. Bolted joints shall be tightened after completion of structure erection in accordance with the structure designer's recommendation.

11.5 INSTALLATION ON FOUNDATION

11.5.1 Anchor Bolt and Base Plate Installation. Installation of the pole on the foundation anchor bolts shall be made in such a manner as to ensure that both the leveling nuts and top anchor bolt nuts are tightened in accordance with the assembly documents. If welding is used as a postinstallation means of securing nuts from loosening while in service, the welds shall be between the nut and the base plate only. Welding to anchor bolts is not permitted.

The final installed distance between the top of the concrete foundation and the bottom of the structure base plate at each anchor bolt shall not exceed the maximum dimension specified in the assembly documents. For structures requiring raking, or in other cases where the distance between the concrete and base plate varies from anchor bolt to anchor bolt, this requirement shall apply to the anchor bolt with the greatest distance between the concrete and the bottom of the base plate.

11.5.2 Direct-Embedded Poles. The annular opening around the embedded pole shall be backfilled with material as specified by the line designer. Backfill material shall be compacted in accordance with the line designer's requirements.

11.5.3 Embedded Casings. Embedded casing installations are either direct embedded or vibrated in place. Direct-embedded casings shall be installed in accordance with Section 11.5.2. Once the superstructure is set inside the steel casing, it shall be plumbed in accordance with the assembly documents. The annular space between the structure and the steel casing shall then be filled in accordance with the line designer's requirements.

11.6 GUYING

11.6.1 Guy Anchor Location. Guy anchors shall be installed at the locations specified by the structure designer and approved by the line designer. If field conditions prevent the installation of any anchor at the specified location, the structure designer shall be consulted to provide an acceptable alternate location or other specific measures.

11.6.2 Guy Installation. Installation and tensioning of guys shall be in accordance with the assembly documents and shall include any pretension requirements for structures designed with pretensioned guys.

11.7 POST-ERECTION PROCEDURES

11.7.1 Inspection. Structures shall be inspected for proper tightening of all bolted joints, condition of protective coating, and vertical alignment (plumbness or rake). All field-assembled connections, including slip joints, shall be inspected prior to wire installation.

11.7.2 Grounding. Installation of all required structure grounding shall be completed promptly after structure erection.

11.7.3 Coating Repair. By the owner's approval, all damaged areas of protective coating shall be repaired in accordance with the coating manufacturer's recommendations.

11.7.4 Unloaded Arms. Unloaded arms shall be evaluated by the structure designer for susceptibility to damage from wind-induced oscillations. If arms are expected to be installed unloaded, remedial measures to reduce oscillation magnitudes shall be specified in the assembly documents along with the time frame within which such measures shall be installed.

11.7.5 Hardware Installation. Conductor dampening devices and/or spacers for bundled conductors, if required, shall be installed after conductor stringing is completed.

COMMENTARY

This commentary is not a mandatory part of the standard but is intended for informational purposes only. The information is provided as explanatory and supplementary material to assist in applying the recommended requirements.

Please note that the sections of this commentary are numbered to correspond to the sections of the standard to which they refer. Because it is not necessary to have supplementary material for every section in the standard, there are deliberate gaps in the numbering sequence of the commentary.

In addition, there is no commentary for Chapters 1, 2, or 3.

CHAPTER C4
LOADING, GEOMETRY, AND ANALYSIS

C4.2 LOADING

C4.2.1 Factored Design Loads. ASCE has developed information to aid in the selection of loads for transmission line structures. MOP 74, *Guidelines for Electrical Transmission Line Structural Loading*, presents a reliability-based methodology for developing transmission line structure loads. Other methods for selecting loads also may be acceptable where utility companies have established procedures that are based on years of successful operating experience.

C4.2.2 Loading Considerations. Prevailing practice and most state laws require that transmission lines be designed, as a minimum, to meet the requirements of past or current editions of the National Electrical Safety Code (NESC).

When evaluating potential structure loading criteria, the owner should consider the following sources of information:

- Individual utility planning criteria will usually dictate specific conductor and shield wire sizes.
- Minimum legislated loading conditions are specified in applicable national, state, and local codes [e.g., NESC, California GO-95, and Canadian Standards Association (CSA-22.3)].
- Historical climatic conditions in the utility's service area may indicate loads in excess of legislated loads. These may include wind or ice, or any combination thereof, at a specified temperature.
- Local terrain and line routing procedures will determine the individual structure orientation criteria.
- Individual utility policies and procedures will determine specific construction and maintenance requirements, such as structure stability before conductor/shield wire installation, the potential for unbalanced longitudinal loads during conductor/shield wire stringing, the need for attachment provisions for structure lifting and hoisting material such as insulators, stringing blocks, and so forth, or the need for hot line maintenance capability.
- Individual utility planning criteria and experience may require the need for a load condition to prevent progressive line failure (cascading).
- Utilities may need to consider unique loading situations that are applicable to their service areas or created by joint use of their structures. Examples of service area loading conditions include galloping and/or aeolian vibration of wires, as well as seismic events. Typical electrical transmission structure designs provide adequate strength to resist loadings from seismic acceleration forces. Tubular frame-type structures may need to consider the effects of foundation movement caused by earthquake ground motions. Examples of joint-use loads are telecommunication applications and other nonelectrical apparatuses (such as traffic signals). The owner should determine the applicable design code for joint-use applications.

C4.2.3 Load Expression. The owner should transmit specific loading criteria to the structure designer in the form of load trees, load tables, or electronic files using a single orthogonal coordinate system as shown in Figure C4-1. These loading criteria should express the magnitude, direction, and point of application for each load and load case. Conductor and shield wire loads should be shown at their appropriate attachment points. The weight of insulators and hardware should be included in these loads.

The magnitude and direction of wind on the structure should be defined by the owner. Shape and height coefficients should be included in the wind loading or listed separately in tabular form. One reference for these coefficients is MOP 74, *Guidelines for Electrical Transmission Line Structural Loading*. If listed

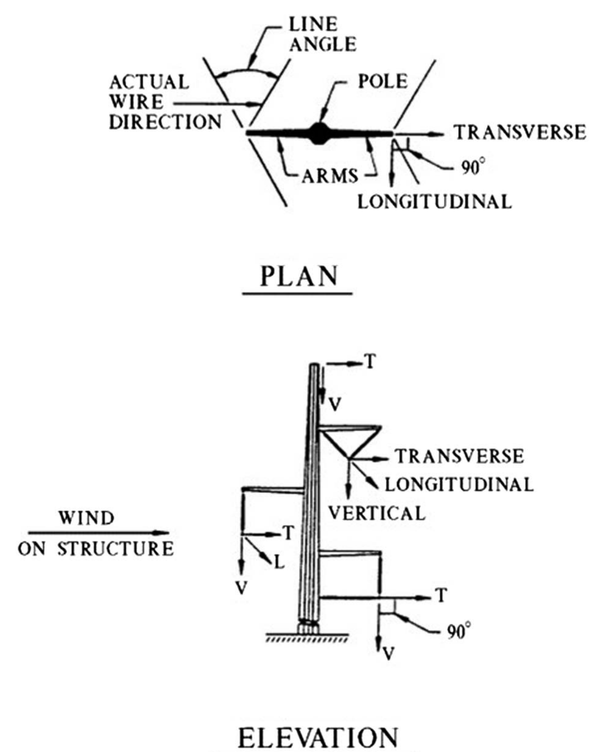

Figure C4-1. Recommended load tree format.

separately, the use of these coefficients should be defined by a formula.

When necessary, design loads for attachment plates should be shown separately.

All special loading considerations that may affect the design of the structure should be clearly communicated to the structure designer (e.g., reverse wind on bisector of a guyed small-angle structure, temporary construction guying, or installation of single-circuit conditions for double-circuit structures). Revisions to the design criteria by the owner should be well-documented and communicated to the structure designer because they may affect the structure design.

C4.3 GEOMETRIC CONFIGURATIONS

C4.3.1 Configuration Considerations. The owner should determine the preliminary structure geometry based on an appropriate technical and economic evaluation of the electrical and mechanical performance requirements.

In general, transmission structures may be classified as one of three types: suspension, strain, or dead end.

Suspension structures are those in which conductors and shield wires pass through and are suspended from support points.

Strain structures are those in which conductors and shield wires are attached to the structure by means of bolted or compression dead-end fittings. These structures are designed to support intact, basically equal, longitudinal loads on both sides of the structure.

Dead-end structures use similar conductor and shield wire attachment methods as used on strain structures. However, dead-end structures are designed to support intact unbalanced longitudinal loads because of differing conductor and shield wire tensions and/or sizes on opposite sides of the structure.

Additional nomenclature for the basic structure types is used to help identify structure orientation with respect to the centerline of the transmission line. The term *tangent* denotes a basic structure type with little or no line angle. The term *angle* denotes a basic structure type that is subjected to various degrees of line angle. Therefore, the following terminology is recommended: tangent suspension, angle suspension, tangent strain, angle strain, tangent dead-end, and angle dead-end.

The owner should consider the combined aesthetic effects of the overall structure configuration, pole taper limitations, pitch and curvature of the arms, insulator configurations, rake, pre-camber, additional appurtenances, brackets, vangs, type of finishes, field coating repair, and other items that may affect the installed appearance of the structure. Attention to these details can greatly improve the installed appearance of the structure.

C4.3.2 Structure Types. The previously described structure types may also be categorized as either self-supported or guyed. Self-supported structures have sufficient strength and stiffness to support the design loads without any guy support. Guyed structures rely on guys for load distribution, stiffness, and stability.

C4.4 METHODS OF ANALYSIS

The response of a tubular structure subjected to factored design loads is usually nonlinear. Geometric nonlinearity (also called second-order effects or P-delta effects) results from displacements that can be substantial. Material nonlinearity may occur in the behavior of the steel material, with localized yielding taking place. Localized yielding may even take place at load levels less than design loads because of stresses induced during manufacturing.

Three states of behavior can be described for a tubular structure: elastic state, inelastic or damage state, and ultimate or collapse state. A structure is in the elastic state if it does not sustain permanent deformation under loading conditions. A structure is in the inelastic or damage state if it can safely carry the loads but sustains permanent deformation. Repair or replacement may be required depending on the extent of damage sustained. A structure is in the ultimate or collapse state when the loads cannot be supported. Geometric nonlinearities are present in all three states. Material nonlinearity becomes significant only in the damage or ultimate states. It is not significant in the elastic state.

C4.4.1 Structural Analysis Methods. The philosophy adopted by this standard is to design a structure so that it will not be permanently damaged under the design climatic, construction, and maintenance loads. For security loads, the owner may allow permanent damage as long as ultimate collapse is prevented. Allowing permanent damage takes advantage of the fact that tubular structures can exhibit additional strength beyond the onset of ultimate loading or collapse state.

For design conditions in which permanent damage is prohibited, a geometrically nonlinear elastic analysis is required. For the rare design conditions in which damage is tolerated, a nonlinear analysis with both geometric and material nonlinearities is required. The elastic method of analysis, with geometric nonlinearity included, is the most common analysis method used by structure designers.

The analysis should have the ability to predict elastic instability phenomena (i.e., the analysis should indicate increasingly large deformations, even under small lateral loads, for vertical loads approaching the buckling load of the structure).

The analysis should include the axial and shear forces, as well as the bending and torsional moments at the critical locations of the structure. With geometric nonlinearity included, all forces and moments should be in equilibrium in the deformed state of the structure.

The structure should be modeled with a sufficient number of elements to ensure that locations of maximum stresses coincide with the origin or the end of an element, and the effects of deflection on amplification of moments are included. All structural members, including interpole ties, bracing, and guys, should be included as elements.

The inherent flexibility of most unguyed tubular structures can have a substantial effect on the magnitude of the loads and conductor and shield wire sags caused by an unbalanced longitudinal condition. For example, conductors and shield wires that remain intact may be able to provide some support for a structure adjacent to a single conductor break. When the loads on a flexible structure are considered to be affected by their connection to other structures through conductors and shield wires, a system analysis may be performed. As a minimum, the system should include several spans in either direction and their supporting structures. The conductors and shield wires in the spans should be modeled as catenary elements.

Cross-sectional properties of commonly used tubular members can be approximated by the formulas in Appendix B.

Guys may be modeled as straight tension-only bars, prestressed if desired, or as a cable element including the actual prestress in the guy. If the structure is designed using the reduced moments resulting from prestressing the guys, the owner's assembly and erection specifications should require compliance with these assumptions. For further information, see Chapter 5 of *Design of Guyed Electrical Transmission Structures*, MOP 91.

If foundation movement is of concern, the model should be able to accommodate specified displacement or rotation at the structure base or connection to foundation elements.

The effects of aeolian vibration of the structures or members and the resulting fatigue stress on the structure or members should be considered. Damping options or recommendations should be provided to the owner where appropriate. In lieu of a comprehensive engineering analysis of a structure or member, the installation of the conductor or shield wire can normally be considered an effective means of suppressing these vibrations.

C4.5 ADDITIONAL CONSIDERATIONS

C4.5.1 Structural Support. The degree of support provided by structure foundations can have a significant effect on the design of a structure because of foundation rotation or displacement. If foundation rotation or displacement allowances are specified, the owner should establish the performance requirements for the structure, guys, and foundations. In determining this value, aesthetics, electrical clearances, and the ability to replumb a structure should be considered.

C4.5.2 Design Restrictions. Structure shipping length and weight restrictions are usually influenced by construction site conditions and material handling limitations.

Structure diameter, taper, and deflection restrictions are usually influenced by the desired appearance of installed structures. Line angles and unbalanced phase arrangements can create loading situations that will cause a structure to deflect noticeably. Several methods can be used to minimize these effects. One method is to camber the structure during fabrication to offset the anticipated deflection under load so that it will appear straight and plumb after installation. Another method is to rake the structure during installation. The deflection at the top of the structure is determined, and the pole is tilted a corresponding amount so that the top of the structure is at a specified position in relation to the structure at groundline.

To camber or rake a structure, a special load case, usually the normal or everyday load on the structure, should be specified by the owner.

The structure finish is a factor that influences the design and fabrication of the structure. The most common structure finishes are hot-dip galvanized, weathered, painted, zinc silicate–coated, and metalized. The selection of a finish is normally influenced by environmental exposure, appearance, and regulatory requirements.

Critical dimensions should be based on an owner's design clearance buffers and specified construction tolerances and the structure designer's connection tolerances. Critical maximum and/or minimum dimensions are important to ensure required clearance to groundline, clearance between wire attachments, clearance to objects, and height requirements are met when slip joints are used because of their large assembly tolerance. Hardware fit-up may also be compromised if critical dimensions are not specifically identified. The use of "maximum" or "minimum" added to the dimension(s) in the contract specifications is a way to direct the structure designer's attention to critical dimensions.

The determination of the shaft-to-shaft connection is normally based on the type and magnitude of structure loading. Shafts loaded in bending or compression are normally designed using a slip joint connection, whereas shafts loaded in uplift, or guyed structures with axial loads greater than the structure designer's recommended jacking force, normally use a bolted flange connection.

The type of foundation is usually based on economic factors influenced by geotechnical conditions, construction material costs, and structure loads. The drilled shaft and anchor bolt, direct-embedded pole, and embedded casing foundations are the most common types used for tubular steel pole structures.

Guy attachment and anchor locations are usually determined by structural support, electrical clearance, and right-of-way considerations.

C4.5.3 Climbing and Maintenance Provisions. In general, provisions should be made so that all portions of structures and insulator and hardware assemblies are accessible for maintenance purposes. Where steps and/or ladders are required, they should be sufficiently strong so they do not deform permanently under the weight of maintenance personnel with tools and equipment.

All climbing devices should be oriented to provide adequate clearance between maintenance personnel and energized parts, allowing for conductor movement under specified climatic conditions. Detachable ladders should be fabricated in lengths that can be handled by maintenance personnel on the structure. Additional information on climbing can be obtained in IEEE 1307, *IEEE Standard for Fall Protection for Utility Work.*

C4.5.4 Pre-Engineered Steel Poles (Wood Pole Equivalents). Steel poles installed in applications in lieu of standard-class wood poles are often referred to as *wood equivalent* steel poles. However, this is a misnomer because it is impossible to equate the properties of a steel pole to those of a wood pole under all load conditions. Steel and wood poles have different properties, resulting in different performance. In addition, there is no standard within the industry for the dimensions of pre-engineered steel poles. Steel poles of the same equivalent class fabricated by different manufacturers may have different top diameters, tapers, and cross-sectional properties. The use of pre-engineered steel poles in lieu of individually engineered, site-specific, steel poles has become a common practice in the industry. The line designer should ensure that the properties of a pre-engineered pole are properly evaluated before specifying it for a specific design application.

Much has been written about the difference in load factors and strength factors between wood and steel poles. Although there is no national standard, often the manufacturer's standard-class steel poles are sized to carry the ANSI O5.1 classification load applied 2 ft (0.6 m) from the pole top with appropriate adjustment for the difference in NESC Rule 250B Grade B wind load factor between steel and wood. Historically, and through 2010, NESC has used a safety factor of 2.5 for wind on steel poles and 4 for wind on wood poles, leading to an equivalency factor of 0.625 (2.5/4). As such, the poles do not have equivalent moment capacities under other loading conditions, such as line tension, extreme wind, or under Grade C construction. The designer should recognize and specify the governing loading condition if other than NESC Rule 250B, Grade B. Differences in material and section properties of the wood pole versus the steel pole result in differences in buckling capacity, pole deflections, secondary moments, and applied wind forces, among others. Some computerized analysis programs allow the designer to check each individual structure for the various load cases, using the appropriate load and strength factors, to help ensure that each pole, whether wood or steel, is adequately sized for the application.

Other considerations for the design or selection of pre-engineered steel poles should include, but not be limited to, the following:

- The slip splice common on two-piece steel poles may not be adequate for large axial loads, either tension or compression. If a slip-spliced pole is used in a guyed structure, the compression load expected in the pole should be less than or equal to the jacking force during assembly. A slip-spliced pole subject to axial tension, as may occur in a braced H-frame below the X-brace, should be locked with some device to keep the joint from separating. Slip splices in H-frame structures may pose alignment problems for cross-arms and cross bracing if the holes are not field drilled. Pole slip joints cannot be fabricated to slip exactly the same amount and achieve full engagement, resulting in poles of different length and different attachment locations.

- NESC allows wood poles to be designed as struts with the guys holding the entire transverse load. Guyed steel poles should be designed as a complete structural system with the transverse load shared by the pole and guys. This difference in analysis methodology can result in different pole sizes for the same loading condition.
- Combining poles of different sizes and/or materials in multiple-pole structures should be closely checked because the loads are not equally distributed between the poles. The stiffer pole carries more than its share of the load; in addition, the steel pole might be significantly lighter than the wood pole, decreasing its uplift resistance.
- The customary wood pole embedment depth of 10% plus 2 ft (0.6 m) may not be adequate. A foundation analysis based on soil and pole information should be performed to ensure adequate embedment depths.
- Connections to steel poles are often overlooked in the design process. The fabricator of the pre-engineered steel poles may not have any design responsibility and may not be aware of what connections are to be made to the pole. The line designer is then responsible for the design of adequate connections. Connections to pre-engineered steel poles as a system have not been extensively tested or shared with the industry. Most testing was apparently performed to validate the design of the attachment rather than the connection to the pole. Some of the attachment methods used on wood poles cannot be used on steel poles. For example, typical attachments that induce heavy concentrated forces caused by davit arm or X-brace connections with traditional through-bolt details can introduce forces significant enough to cause a local buckling of the pole wall in the region of the bolting hardware. Reinforcement at these bolted connections can aid the pole section in resisting these induced forces. Often, spanning the bend lines of the pole section with a plate washer can add to the strength of the through-bolt connection.
- Drilling holes in steel poles reduces the strength of the poles. Field drilling of holes in welds or bend lines should be avoided. If the hole is on the neutral axis or located in an area of minor stress, the loss of moment capacity may be insignificant. If the hole is in an area of high stress, the loss of moment capacity could be significant and should be taken into consideration.
- Bolt shear strength is not given much consideration in most wood pole connections because of the large bearing area and the use of cleated washers to distribute loads. However, the bearing area may be a limiting factor for bolts in steel poles and should be checked.
- Bearing plates are necessary on steel poles to provide a bearing surface similar to that of a solid pole. Sealed poles may have a solid bottom plate, but galvanized poles may have openings to facilitate the galvanizing process. The intended use of the pole and the vertical load requirements determine the size of the bearing plate. The bearing plate can be extended beyond the pole shaft to provide uplift resistance if needed.
- Many steel poles are manufactured as regular polygons instead of being truly round. As such, the section modulus of the steel pole varies depending on the orientation of the flat sides. The structure designer needs to consider the orientation of the pole with respect to the loads. For example, when the resultant moment is across the points of the polygon, the section modulus is at its minimum.

CHAPTER C5
DESIGN OF MEMBERS

C5.1 INTRODUCTION

The American Institute of Steel Construction (AISC) and, to a lesser extent, the American Iron and Steel Institute (AISI) specifications (AISC 2005, AISI 1980) are the basis for the design requirements of this standard. Transmission structures have traditionally been designed based on ultimate strength methods using factored loads. The design stresses of this standard are derived from AISC *Specification for Structural Steel Buildings*. AISC allowable stresses are applicable to equations in which the member forces are the result of unfactored loads. The design stress values in this section are based on the allowable stresses in the AISC specification, with the values adjusted upward (by factors ranging from 1.5 to 2.0) for use in ultimate strength design and to compensate for the equivalent safety factors built into the AISC values.

These design requirements are applicable only to tubular members, truss members, and guys. ASCE (2000) or AISC (2005) provide for the design of other members, with appropriate conversions from allowable stress to ultimate strength design.

AISC (1986) includes published design criteria based on the load and resistance factor design (LRFD) methodology. Additional testing to determine probability-based factors for details unique to tubular transmission structures is required before adopting the LRFD method.

C5.2 MEMBERS

The formulas used in this section historically have been used in the industry to design members with cross-sectional shapes as shown in Appendix B and members with elliptical or rectangular cross sections that have maximum major to minor dimension ratios of 2 to 1.

C5.2.1 Material

C5.2.1.1 Specifications. Steel pole structures are typically manufactured from high-strength structural steel with a yield strength of 65 ksi (448 MPa). The suitable materials listed include some that are not specifically referenced in the AISC or AISI specifications (1980, 2005) but have been proven acceptable through in-service performance.

C5.2.1.2 Material Properties. Cold working in forming tubes increases the yield stress of the steel. However, increasing the design yield stress over the minimum yield stress specified in the applicable ASTM specification in Chapter 2 is not recommended. Because the difference between the yield stress (F_y) and the tensile stress (F_u) of the high-strength steels from which these structures are normally fabricated is relatively small, the beneficial effect of cold working would also be relatively small. Furthermore, this benefit would likely be offset to some extent by a reduction in the notch toughness.

C5.2.1.3 Energy-Impact Properties. In general, brittle fracture can occur in structural steel when there is a sufficiently adverse combination of tensile stress, temperature, strain rate, and geometrical discontinuities (notches). Other design and fabrication factors may also have an important influence. Because of the interrelation of these effects, the exact combination of stress, temperature, strain rate, notches, and other conditions that cause brittle fracture in a given structure cannot be readily calculated. Consequently, designing against brittle fracture primarily involves proper steel selection and minimizing geometric discontinuities.

The energy-impact property requirements of this section are based on historical experience of structure performance. These requirements exceed those listed in Table 3-1 of MOP 72 (ASCE 1990) for certain plate strengths and thicknesses, but the added cost of providing this testing is minimal. More stringent temperature requirements may be necessary in some areas because of the expected climatic conditions. Energy-impact requirements are applicable only to plate material, not to hot-rolled shapes. The energy-impact properties are supplemental requirements to the plate and coil specifications listed in Section 5.2.1.1.

This standard requires only heat-lot testing to be used. The requirement contained in MOP 72 (ASCE 1990) for plate testing of controlled-rolled or as-rolled plates greater than 0.5 in. (13 mm) thick was eliminated based on experience with the plate quality currently produced by steel manufacturers.

C5.2.2 Tension. For tension members with holes or slots, yielding of the net area may become a serviceability limit state warranting special consideration and exercise of engineering judgment. These conditions can result when the length of the hole or slot along the longitudinal axis of the member exceeds the member depth or constitutes an appreciable portion of the member length.

C5.2.3 Compression

C5.2.3.1 Truss Members. As discussed in Section C4.4.1, the elastic stability of beam elements in tubular structures is numerically checked during the nonlinear analysis and need not be checked by manual methods. However, when nonlinear analysis methods are used, the stability of truss elements, which by definition can carry only axial loads, is not checked. These members should be manually checked for stability using Equations (5.2-3) and (5.2-4).

Typically, truss members, made from round or polygonal tubes or angles, are used as bracing in a tubular transmission structure (cross braces in an H-frame, arm braces, and so forth).

The K factor for a truss member depends on the connection design for the member. Theoretically, $K = 1.0$ for a member pinned at both ends (AISC 2005). In practice, these members are attached to the structure with a single bolt installed perpendicular to the plane of the truss member. This connection may not act as a pin for loads out-of-plane with the member (such as longitudinal or torsional loads on an H-frame structure). This is especially important in the design of nonsymmetrical members, such as angles. Engineering judgment should be used in the selection of the K factor for this type of connection.

Truss members used for cross bracing are usually connected at the point of intersection by means of a U-bolt or through bolt. This connection changes the effective length of the compression brace based on the amount of rotational support provided by the connection. The effective length for tubular members was shown to depend on the relative load levels between the compression brace and the tension brace (Thevendran and Wang 1993, Khadivar 1990). Assuming no rotational support is provided by the bolt and the point of intersection is at the midpoint of the cross bracing, the K factor varies from 0.5 (tension load = 60% to 100% of the compression load) to 0.72 (no tension). This K factor applies to the overall length of the compression member. Thevendran and Wang (1993) provide suggested K factors for other bracing configurations. If the connection between the braces provides sufficient rotational support, $K = 0.8$, based on the length of the compression member from the intersection of the braces to the main support (AISC 2005).

KL/r values for tubular truss members should be limited to prevent potential vibration problems. Typically, these members are limited to values of 200 for compression members and 300 for tension members (AISC 2005).

C5.2.3.2 Beam Members. This section determines the design compressive stress based on what is commonly referred to as local buckling. When testing is performed to determine local buckling stability, actual yield strengths and dimensions of the test specimens should be used in the calculations.

C5.2.3.2.1 Regular Polygonal Members. Equations (5.2-6) through (5.2-11) are based on research conducted by the Electric Power Research Institute (EPRI) for tubes in bending and were published in a report (Cannon and LeMaster 1987). Full-scale testing (Cannon and LeMaster 1987, Currance 1974) demonstrates that regular polygonal-shaped tubes with different numbers of sides have different buckling capacities.

Thus, different equations are provided for octagonal, dodecagonal, and hexadecagonal tubes. These equations are summarized graphically in Figures C5-1 through C5-4.

Based on this testing, less conservative criteria than were previously used have been established for polygonal tubes with eight or fewer sides [Equations (5.2-6) and (5.2-7)]. However, these equations should be used only when the primary loading is bending. If the axial stress, *P/A*, is greater than 1 ksi (6.9 MPa), then Equations (5.2-8) and (5.2-9) should be used for tubes with eight or fewer sides.

Equations (C5.2-1), (C5.2-2), and (C5.2-3) are provided here for reference. These are the elastic local buckling stresses based on a plate buckling coefficient of 4.0. These are extensions to the equations contained in this section but are not validated by the same testing. The use of polygonal shapes in these ranges of *w/t* is uncommon. Sections in these ranges could be highly susceptible to shipping, handling, and construction damage.

- Octagonal, hexagonal, or rectangular members (bend angle ≥45°)

$$F_a = \frac{3.62E}{(\frac{w}{t})^2} \text{ when } \frac{w}{t}\sqrt{\frac{F_y}{E}} > 2.06 \qquad \text{(C5.2-1)}$$

- Dodecagonal members (bend angle = 30°)

$$F_a = \frac{3.62E}{(\frac{w}{t})^2} \text{ when } \frac{w}{t}\sqrt{\frac{F_y}{E}} > 2.20 \qquad \text{(C5.2-2)}$$

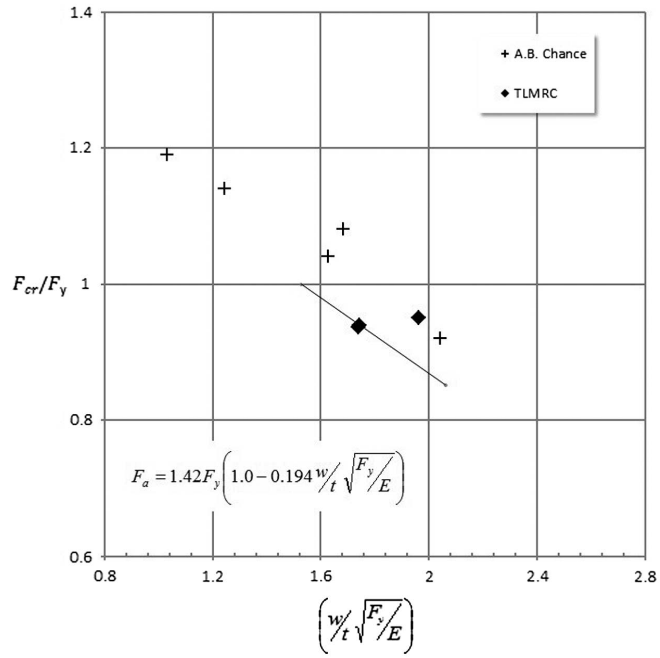

Figure C5-1. Local buckling test data for octagonal tubes.

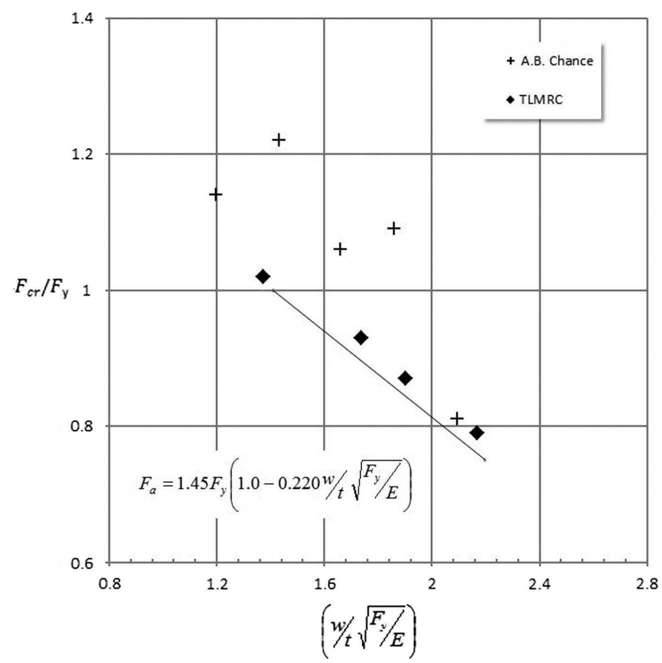

Figure C5-2. Local buckling test data for dodecagonal tubes.

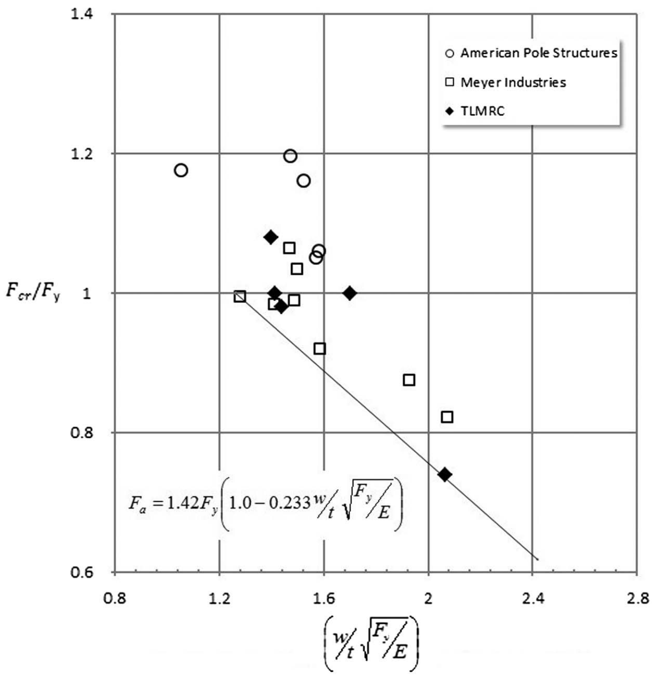

Figure C5-3. Local buckling test data for hexadecagonal tubes.

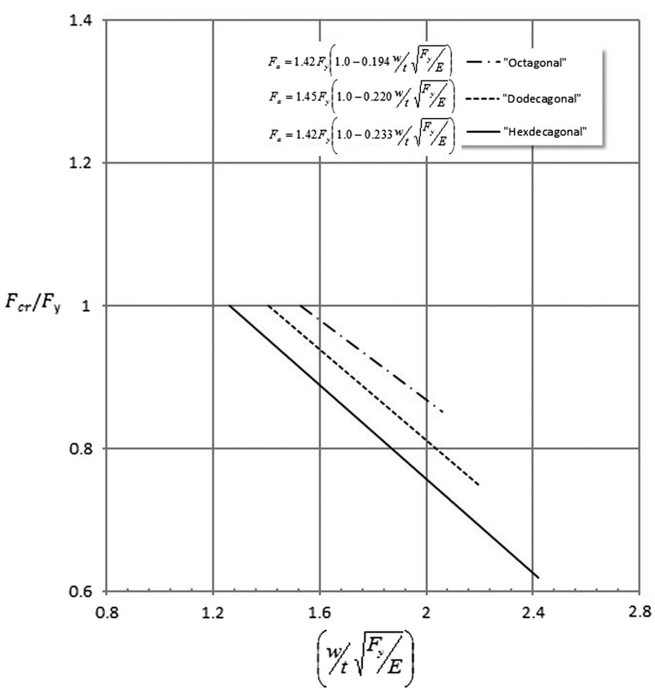

Figure C5-4. Comparison of local buckling equations.

- Hexadecagonal members (bend angle = 22.5°)

$$F_a = \frac{3.62E}{(\frac{w}{t})^2} \text{ when } \frac{w}{t}\sqrt{\frac{F_y}{E}} > 2.42 \quad \text{(C5.2-3)}$$

where

F_y = Specified minimum yield stress;
E = Modulus of elasticity, 29×10^3 ksi (200×10^3 MPa);
F_a = Compressive stress permitted;
w = Flat width of a side; and
t = Wall thickness.

C5.2.3.2.4 Round Members. Equation (5.2-14) is the Plantema formula (Plantema 1946, Schilling 1965). Figure C5-5 ("Tests on round tubes in bending," unpublished report, Union Metal Manufacturing Co., Canton, OH) shows it to be a good lower bound on the test results of axially compressed manufactured round tubes.

Manufactured tubes are classified as tubes produced by piercing, forming and welding, cupping, extruding, or other methods in a facility devoted specifically to the production of tubes.

Equation (5.2-16) is a modification of the Plantema formula derived from the test results shown in Figure C5-6 and provides a good lower bound. The tests shown herein are from Schilling (1965) and "Tests on round tubes in bending" (unpublished report, Union Metal Manufacturing Co., Canton, OH).

The use of circular tubes with D_o/t values exceeding the upper limits established by Equations (5.2-14) and (5.2-16) is uncommon, and no allowable stress equations are provided for them. To establish such equations, an adequate test program would be needed.

C5.2.4 Shear. Equation (5.2-20) is a rounded value of the yield stress in shear ($F_y/\sqrt{3}$) based on the distortion-energy criterion.

C5.2.5 Bending. A reduction in the design stress to account for lateral-torsional buckling is not necessary for tubular members because of their superior torsional stiffness.

The limiting bending stress is based on local buckling only because stability will have been verified by nonlinear analysis.

C5.2.6 Combined Stresses. Combinations of shear stresses and normal stresses have been evaluated by the distortion-energy (Hencky–Mises) yield criterion. More conservative criteria may be used. Stresses should be properly combined at a given point on the cross section. They are not necessarily the addition of the maximum stresses. For example, the maximum normal stress occurs at an extreme fiber, whereas the maximum shear stress occurs at the neutral axis. Normally, the highest stress results from combining the maximum normal stress with the shear stress occurring at the same point.

C5.3 GUYS

C5.3.1 Material Properties. Zinc-coated steel wire strand according to ASTM A475 and aluminum clad steel strand according to ASTM B416 are commonly used for guys. Capacities of these strands are stated as the minimum rated breaking strength.

Physical properties, such as minimum rated breaking strength and modulus of elasticity, for other types of wire strands or ropes should be specified by the owner.

C5.3.2 Tension. The level of force represented by 65% of the rated breaking strength of a guy is a reasonable measure of the point at which the deformation rate begins to become nonlinear. This is analogous to the yield strength of other steel members.

Forces greater than 65% of the rated breaking strength of a guy have been permitted by the *National Electrical Safety Code* (NESC) [Institute of Electrical and Electronic Engineers (IEEE 2002). However, when stressed above 65% of the rated breaking strength, inelastic stretching of the guys may occur, which is outside the scope of this standard. For further information, see Chapter 6 of *Design of Guyed Electrical Transmission Structures*

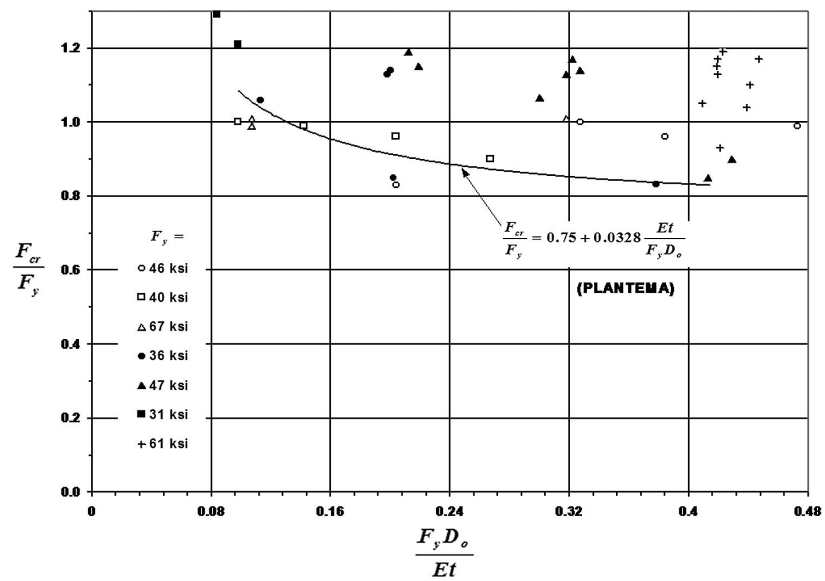

Figure C5-5. Local buckling test data for round tubes in compression.

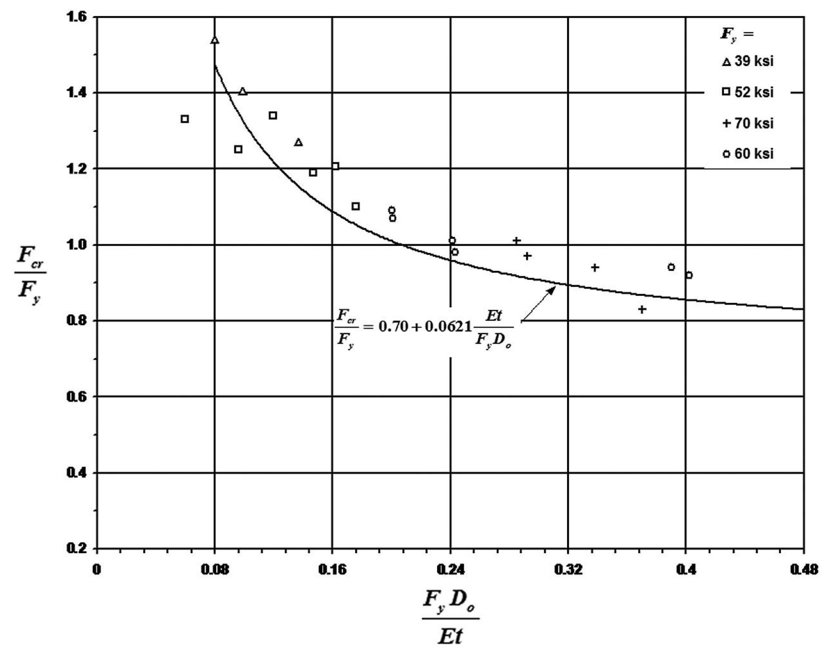

Figure C5-6. Local buckling test data for round tubes in bending.

(ASCE 1997). In addition, the guys should be retensioned or replaced after the occurrence of such an event to prevent excessive structure deflections or stresses as a result of the inelastic stretching of the guys.

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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:

$$P \leq 0.6\, dtF_y \qquad \text{(C6.2-1)}$$

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.

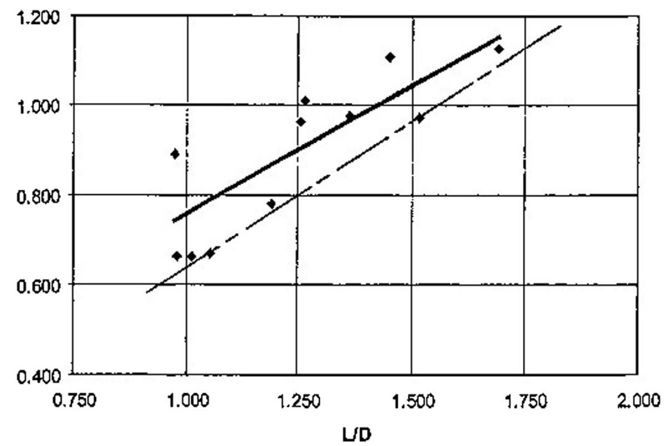


Figure C6-1. All EPRI splice failures.

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 above-ground 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

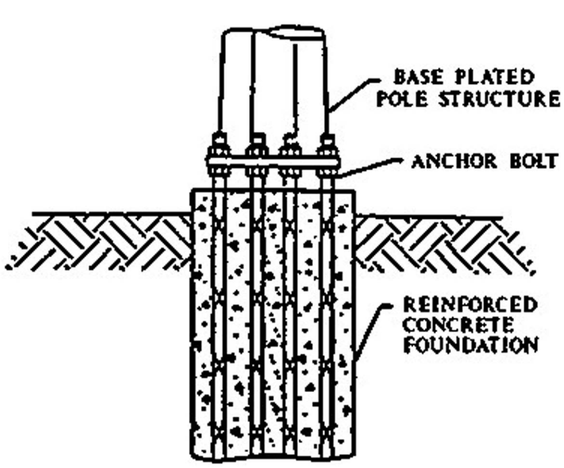


Figure C9-1. Drilled shaft foundation with anchor bolts.

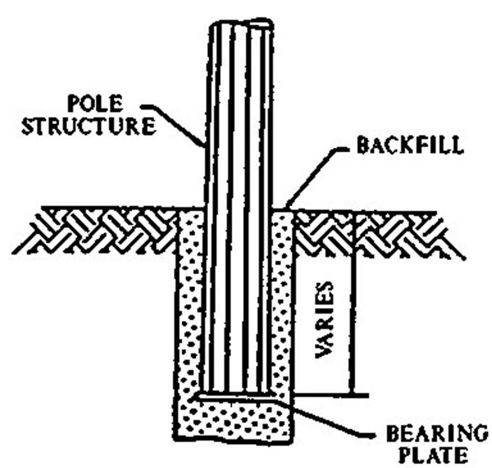


Figure C9-2. Direct-embedded pole.

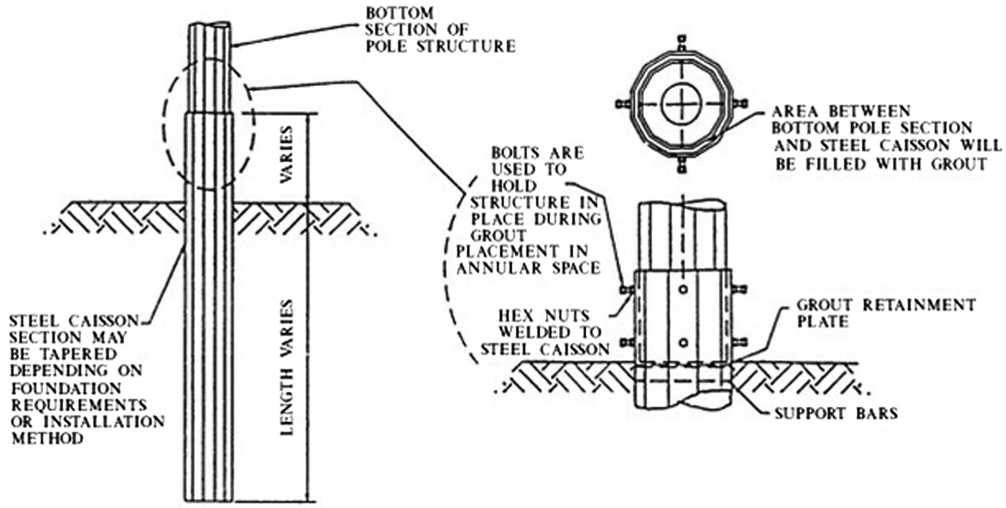


Figure C9-3. Embedded casing foundation.

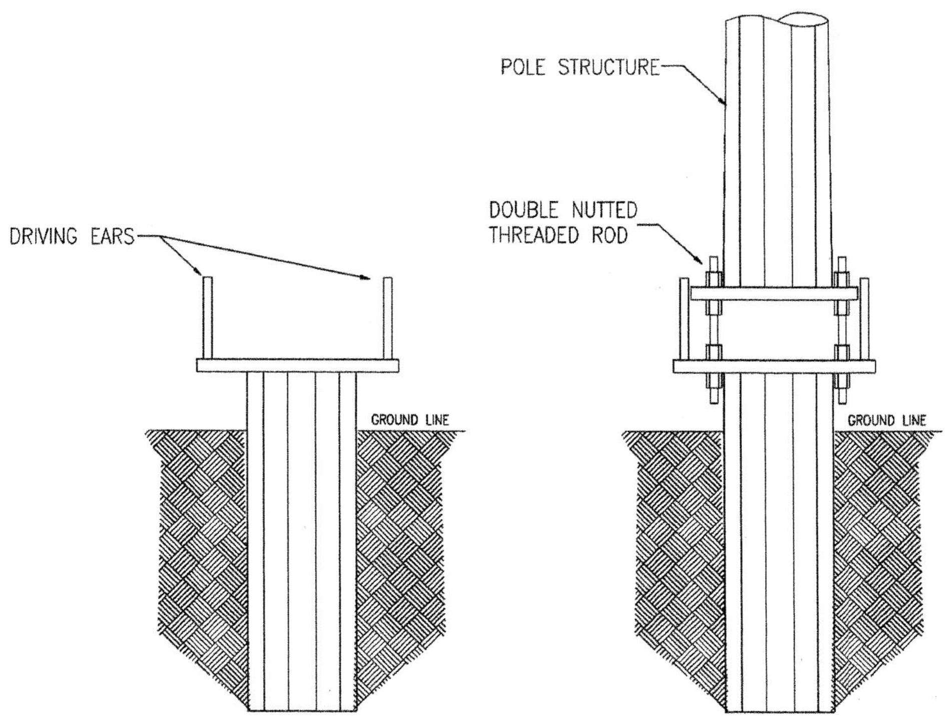


Figure C9-4. Base plate vibratory caisson foundation.

necessary to apply an additional protective coating, such as a bitumastic compound, polyurethane coating, galvanizing, or paint, to the steel. Cathodic protection can be used to inhibit corrosion. Consideration may also be given to adding a ground sleeve or to increasing the thickness of steel members exposed to corrosive ground conditions. Concrete encasement or reinforced concrete foundations are often used for poles located in highly corrosive environments, such as ash pits, industrial drainage areas, and oil refineries.

C9.3 ANCHOR BOLTS

The drilled shaft is a type of foundation that is used extensively with anchor bolts. The minimum foundation diameter is determined by the diameter of the bolt circle, bolt diameter, and the necessary concrete clear cover. The minimum length of the anchor bolts should be determined by the structure designer in accordance with the number and size of bolts used. Typical reinforcing methods include development-length anchor bolts plus reinforcing steel as well as full-length anchor bolt cages, either with or without additional reinforcing steel. Depending on the geotechnical conditions and the foundation loads, the use of full-length anchor bolts can provide cost savings.

Typically, the structure designer designs the anchor bolt to resist the groundline forces. The foundation engineer designs the foundation for the same groundline forces plus any additional loads that would produce a maximum foundation bending moment.

Threaded reinforcing bar is the most common type of anchor bolt used for connecting steel transmission pole structures to concrete foundations. For threaded reinforcing bar, the anchor bolt material should be limited to ASTM A615, Grade 60 for bars No. 5 through No. 18 and Grade 75 for bars No. 11 through No. 18 and 18J (ASTM A615M, Grade 400 for bars 10M to 55M and Grade 500 for bars 35M to 55M).

C9.3.3 Combined Shear and Tension. For steel transmission poles, anchor bolts are bearing-type connections that should include threads in the shear plane when sizing the bolt. The literature presents various equations for approximating the shear and tension interaction. For the interaction equation to be valid, the anchor bolts should have no more than two bolt diameters separating the bottom of the base plate and the top of the concrete. If the distance is greater than two bolt diameters, then bending in the bolt should be included when sizing the anchor bolt.

Bending in combination with shear and axial load may be considered by satisfying the following interaction equation:

$$\left[\left(\frac{f_t}{F_t}\right) + \left(\frac{f_b}{F_b}\right)\right]^2 + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0 \qquad \text{(C9-1)}$$

where

$f_t = |T|A_s$,
$f_b = (0.65\ x_b V)/S_b$,
$f_v = V/A_g$,
$F_t = 0.75\ F_u$,
$F_b = 0.90\ F_y$,
$F_v = 0.35\ F_u$,
$T =$ Maximum anchor bolt axial load (tension or compression),
$V =$ Shear per anchor bolt (shear and torsion components),
$A_s =$ Stress area per Equation (6.23),
$A_g =$ Gross area of anchor bolt,
$x_b =$ Clear distance from top of concrete to bottom of leveling nut,
$S_b =$ Equivalent section modulus of anchor bolt,
$S_b = (\pi d_e^3)/32$, and
$d_e =$ Equivalent anchor bolt diameter based on stress area $d_e = \sqrt{(4A_s)/\pi}$.

C9.3.4 Development Length. The No. 18J (55M) reinforcing bar meeting ASTM A615 Grade 75 (500) has been successfully used for anchor bolts by the industry for many years. Until 1989, anchor bolt embedment length calculations have been based on the ACI 318 development-length provisions for the deformed reinforcing bar. Revisions to ACI 318 in 1989, 1995, and 1999 first increased and then decreased the development-length requirements. The committee decided to continue the industry practice of using the development-length provisions of ACI 318-83 for determining embedment length of threaded, deformed reinforcing bars used as anchor bolts.

For reinforcing bars used as anchor bolts, it is recommended that the development length determined in accordance with Section 9.3.4 be subject to a minimum of 25 bar diameters for all bar sizes to safeguard against the use of unusually short anchor bolts. The development-length calculations are applicable only to uncoated reinforcing bars to safeguard against the use of unusually short anchor bolts. Development length and anchorage value calculations for headed anchor bolts are shown in Appendix D.

C9.4 DIRECT-EMBEDDED POLES

Direct-embedded pole foundations use the bottom portion of the steel pole as the foundation member reacting against the soil, rock, and/or backfill.

A direct-embedded pole foundation typically is designed to transfer overturning moments to the in situ soil, rock, or backfill by means of lateral resistance. Axial loads can be resisted by a bearing plate installed on the base of the pole. Additional bearing capacity can be realized by installing base-expanding devices. The quality of backfill, method of placement, and degree of compaction greatly affect the strength and rotation of the foundation system and, thereby, the design of the embedded pole. Direct-embedded pole foundations have become popular because of their relatively low installation cost. When using direct-embedded poles where there is a high water table, buoyancy of the pole should be considered.

C9.5 EMBEDDED CASINGS

Embedded steel casing foundations are round or regular polygonal tubular steel members that serve as the foundation to which the bottom of the steel pole is attached.

The bottom of the steel pole structure is attached to the casing by either a *socket* or *base plate*–type connection. In a socket-type connection, the aboveground structure is set inside the steel casing. The annular space, usually from 3 in. to 9 in. (76 mm to 229 mm) between the structure and the steel casing, is then filled with either grout or concrete.

In a base plate connection, the flange of the aboveground structure is bolted to a flange on the steel casing. Bolting can be done on either the inside or the outside of the casing. The structure can be plumbed by adjusting leveling nuts.

Vibratory steel caisson foundations have been used successfully to support steel structures in locations with suitable soil conditions. Depending on installation techniques and layer depths, stiff cohesive soils and those with rock may not be well-suited for the use of steel caissons. The steel caisson is vibrated into the ground by the use of a vibratory hammer. The steel caisson is commonly fitted with reinforcing plates or *driving ears* to attach the vibratory hammer. A minimum wall thickness of 3/8 in. (10 mm) is common. Damage to the caisson and its welds is possible if the progress of driving the caisson into the ground is halted or is minimal. If the driving ears develop fatigue weld cracks during driving, they should not be rewelded or reinforced without first contacting the structure designer to discuss the situation.

The geotechnical design parameters for the design of vibratory steel caissons in looser soils are improved because of densification caused by the vibratory installation.

CHAPTER C10
QUALITY ASSURANCE/QUALITY CONTROL

C10.1 INTRODUCTION

A well-planned and executed quality assurance (QA)–quality control (QC) program is necessary to ensure delivery of acceptable material in a timely manner. The objective of the program is to establish that materials are in conformance with the specifications of the purchase contract. A clear and concise contract between the owner and the fabricator is an important part of the procedure necessary to obtain acceptable steel transmission pole structures. The responsibilities of the owner and the fabricator should be defined in the contract so that no part of the process used to purchase, design, manufacture, inspect, test, construct, or deliver material is omitted.

C10.2 QUALITY ASSURANCE

The owner's bid documents should outline the QA methods, types of inspections, and records that will be required to determine the acceptability of the product at each stage of the design, manufacturing, structure testing, and field construction process.

Quality assurance is responsible for the methods followed to establish appropriate review and interface with the fabricator's QC procedures. This will ensure that the contract can proceed smoothly, that proper communication channels are established with the responsible personnel to minimize confusion, that the owner's requirements are properly met, and that proper guidance and adequate technical support is provided throughout the period of the contract.

The owner should determine, by site visit if required, that the fabricator's equipment and process facilities are adequate to meet the requirements of the QA specification, that fabrication procedures are satisfactory, that tolerances are within specified limits, and that the existing QC program is satisfactory.

C10.2.1 Design and Drawings. The QA specification should specify the procedure for reviewing the stress analyses of the main structure and all component parts, including attachments and connections. The fabricator's drawings should be checked to ensure that they contain proper and sufficient information for fabrication and erection in accordance with the requirements of the owner's specification.

C10.2.4 Nondestructive Testing. The owner may specify that the fabricator furnish copies of testing and inspection reports. The owner may also perform independent random sample testing to verify results of the fabricator's testing.

C10.2.5 Tolerances. Dimensional variations can affect the structural performance, ease of assembly, electrical clearances, and structure appearance. The fabricator and the owner should agree on the fabrication tolerances that will achieve the specified performance.

C10.2.6 Surface Coatings. Blast cleaning of weathering steel structures may be specified if a clean and uniformly weathered appearance is important in the structure's initial years of exposure. In time, even a nonblasted cleaned steel structure will usually develop a uniform oxide coating.

C10.2.7 Shipping. Before the start of fabrication, the owner should review the fabricator's methods and procedures for packing and shipping and agree on the mode of transportation.

When receiving materials, the owner is responsible for checking to see that all materials listed on the accompanying packing lists are accounted for. When a discrepancy is detected, both the carrier and the fabricator should be notified.

C10.3 QUALITY CONTROL

A QC program should be established in a manner that provides open avenues of communication throughout the fabricator's plant. It should be headed by a manager with the overall authority and responsibility to establish, review, maintain, and enforce the program. As a minimum, the QC program should identify key personnel who are responsible for planning and scheduling, engineering, drafting, purchasing, production, testing, shipping, and appropriate quality control checks. The QC inspectors are responsible for determining that the product meets the level of quality established by the fabricator's standards and the specific requirements of the owner.

C10.3.1 Material. The fabricator's records should show all pertinent information on all component parts. This may take the form of a *traveler* on major components. The traveler usually contains pertinent information on items such as materials, welding procedures, welder's identification, type of inspection, inspector's test results, records of all visual and nondestructive testing, inspector's identification, and other items agreed on by the owner and the fabricator.

C10.3.3 Dimensional Inspection. Any structure that is of unique and/or complex design should be shop assembled before shipment. Mating parts should be match-marked.

C10.3.4 Surface Coating Inspection. The surface of structural steel prepared for spraying should be inspected visually. The metalized coating should be inspected for thickness by a magnetic thickness gauge. Any metalized surface that exhibits visible moisture, rust, scale, or other contamination should be reblasted before spraying. Defective areas should be sandblasted clean before respraying, except where the rejection results from insufficient thickness.

C10.3.5 Weld Inspection. Not all inspection personnel need to be qualified to AWS QCI. Visual inspection may be performed by noncertified inspectors under the supervision of a certified welding inspector (CWI).

After galvanizing, nondestructive weld testing should be considered to ensure that there have been no adverse effects to the finished product. This is especially important for large T-type weld joints, such as base plate welds.

The fabricator should establish written nondestructive testing procedures and train nondestructive testing personnel in accordance with the guidelines of the American Society for Nondestructive Testing (ASNT) *Recommended Practice No. SNT-TC-1A*.

Nondestructive testing can be used to detect material and welding flaws. Present methods include visual, magnetic particle, dye penetrant, ultrasonic, radiographic, and eddy current. Each of these methods has inherent limitations.

Magnetic particle testing (MT) is a practical method for detecting tight surface cracks. MT inspection should be in accordance with ASTM E709.

Dye penetrant testing (PT) is a very reliable method for detecting any cracks or porosity that are open to the test surface. PT inspection should be in accordance with ASTM E165.

Ultrasonic testing (UT) is the only practical method of determining weld quality in the base and flange connection welds. It is also very reliable in detecting small cracks and internal flaws in other complete penetration welds. It should be noted that AWS D1.1/D1.1M does not provide any specific guidelines for UT of plate less than 5/16 in. (8 mm) thick or for welds using backing bars. It is recommended that the fabricator follow the procedure established by AWS D1.1/D1.1M, Section 6.27.1, in developing a specific inspection procedure.

Radiographic testing (RT) is a method that provides a permanent record of the test results. However, its use is limited on many types of weldments (e.g., base and flange connections) where it is difficult, if not impossible, to position the film to record the entire weld joint. It is also possible to miss tight cracks that lie normal to the RT source and film.

Eddy current testing (ET) techniques have limited application in the determination of weld penetration and the detection of cracks.

Additional information on the limitations and complementary use of each method is explained in ANSI/AWS B1.10, *Guide for Nondestructive Inspection of Welds*.

C10.3.6 Shipment and Storage. The QC program should establish procedures that specify the methods, materials, documentation, and owner's special requirements for handling, storing, preserving, packaging, packing, marking, material releasing, and shipment.

All members in storage should be placed on wood blocking or other suitable material to ensure that the structures are not in direct contact with the ground. Dunnage materials should be suitable for direct contact with the supported materials.

CHAPTER C11
ASSEMBLY AND ERECTION

C11.1 INTRODUCTION

This commentary provides information supporting and explaining the requirements of Chapter 11. Additional relevant information on the assembly and erection of steel transmission pole structures can be found in Appendix E. Chapter 11 identifies assembly and erection practices that are required to ensure adherence to structure design assumptions and to prevent actions that could compromise structural integrity but does not address all aspects of structure installation. The formulation of a complete installation specification is the responsibility of the line designer. Additional information and recommendations for stringing and grounding can be found in IEEE 524, *Guide for the Installation of Overhead Transmission Line Conductors*.

C11.2 HANDLING

The contract specifications, design, and detailing of pole structures should consider limits to length, size, and weight of individual members because of shipping, handling, terrain, and equipment restrictions. In setting weight limits, the owner should consider that actual structure weights can deviate by as much as 15% from the fabricator's calculated weights because of mill and fabrication tolerances.

Pole sections can be stored at centralized storage/staging yards or at the installation site. Creating a process for receiving materials can help in organizing the storage/staging yards and minimize delays owing to possible shortages. Poles stored at storage/staging yards can be partially or fully assembled before transportation to the installation site.

Care should be taken when shipping and handling pole sections to prevent any damage to the protective coating and finish and to prevent puncture, denting, or other types of damage to the steel.

To minimize the impact the environment will have on a member's finish or protective coating, storage/staging yards should be set up so that the materials that are received first can be issued first, where practical. Pole sections should be stored on blocks and cribbing to prevent contact with the ground. The amount and spacing of the cribbing should be arranged to prevent excessive deflection during storage.

C11.3 SINGLE-POLE STRUCTURES

The decision to use aerial or ground assembly depends on individual site considerations and the owner's preference. To minimize the need to install equipment in the air, davit arms, line hardware, insulators, stringing blocks, ropes, and grounds as applicable are usually installed on the structure before erection.

C11.3.1 Slip Joints. The following are recommendations to help ensure proper field assembly of slip joint connections:

- Poles assembled on the ground should have sections blocked level before joint assembly.
- Care should be taken to properly align the sections using marks specified by the fabricator as shown in the assembly documents.
- Proper orientation of jacking nuts, arms, brackets, and climbing accessories should be verified before the jacking process begins.
- The maximum and minimum lap lengths should be marked on the lower section using a water-soluble marker.
- The sections should be overlapped as far as possible before application of jacks or other mechanical assembly equipment. The use of heavy construction equipment to apply force at the ends of the pole sections is not recommended because damage to the sections could occur.
- Final assembly should be made using jacking equipment as specified in the assembly documents. Methods used should be capable of (1) developing the required jacking force and (2) measuring the actual force applied during assembly.
- Applying an up-and-down motion to the upper section during final assembly can often aid in the successful assembly of the slip joint.
- Lubricants should only be used in accordance with the assembly documents.

When slip joints are assembled in the air, the bottom pole section is set on the foundation or embedded in the excavation and then plumbed or raked as required. The section being added should be as plumb as possible during its lowering so as to prevent binding of the sections during overlapping. This upper section should be slowly lowered onto the lower section. The weight of the upper section alone can produce a slip joint that is difficult to disassemble, so it is especially important to ensure proper alignment of sections when assembling in the air.

Sections should be carefully aligned to produce a tight, even joint without major gaps between the two sections. Slip joints transfer load through friction between the section surfaces, thus the use of shims is not recommended, because they can reduce the load transfer capacity of the joint.

If the pole has been assembled in accordance with the assembly documents and gaps that exceed any of the following conditions exist between the sections, the structure designer should be contacted to determine joint acceptability before wires are strung: the sum of the lengths of gaps that exceed 1/8 in. (3 mm) is more than 30% of the slip joint's circumference, or a gap extends across two full adjacent flats and the maximum gap exceeds 1/4 in. (6 mm).

Assembly records are recommended and, as a minimum, should include the following:

- Actual jacking force applied,
- Actual slip joint lap length, and
- Type of jacking equipment and its calibration records.

C11.3.2 Bolted Flange Joints. The flange bolts should be brought to a snug-tight condition. As flange bolts are brought to a snug-tight condition, all of the faying surface may not be in contact.

Final bolt tensioning should follow a sequence to provide for even tensioning of all bolts and to ensure section alignment. A pair of bolts on opposite sides of the joint should be tensioned followed by a similar pair until all bolts are tensioned.

Proper bolt tensioning in flange joints is required because of the cyclic nature of the loading. High-strength bolts are susceptible to cracking when subjected to high-stress fluctuations, and pre-tensioning of the bolts by the turn-of-nut or other approved method ensures a more constant bolt stress and prevents bolt failure.

Flange plates used for pole joints are relatively thick compared with material common to joints in other types of structures, and acceptable fabrication misalignments or plate distortion can result in small gaps between the flanges, even after final bolt tensioning. These gaps, within permitted limits, are not injurious to the load transfer capability of the joint. Larger gaps may be filled by shims.

Alignment marks on mating sections, when provided, should be used to facilitate proper orientation during assembly. The mark can be a weld bead or other permanent mark.

C11.3.3 Attachments to Pole Sections. Some attachments to pole sections, such as arms that use box or bracket type connections in which the bolts act as pin connectors, do not have true faying surfaces and are intentionally loose fitting.

C11.3.4 Erection of Assembled Structures. The structure should be laid out at the installation site in such a way as to minimize erection effort and ensure safety. Structure members should be inspected for any dents, damage to the finish, excessive twist, and proper alignment of jacking nuts, and so on, prior to assembly. If any flaws are detected, corrective measures should be taken before structure assembly is completed. If specified by the owner, insulators, hardware, travelers, and climbing devices may also be added to the structure prior to erection. Once the structure is totally assembled, it should be thoroughly inspected to ensure all members are properly aligned and connections are properly made. A temporary link between slip-jointed sections should be installed to prevent loosening or separation of the sections during lifting. The jacking attachment nuts can often be used for attachment of the link.

The owner may specify that the structure designer provide lifting attachments, such as vangs, slots around the top of pole sections, or through holes to be used in conjunction with lifting rods. The owner may also specify that the structure designer identify the ultimate capacity of each attachment. The erection contractor can then use these capacities, reduced by appropriate safety factors, to efficiently and safely perform the lifting operations with the equipment available.

Poles may be erected using lifting lugs (if installed) or a choker. The lift point for the choker will be field-determined and depends on the assembled arrangement of the pole, including accessories such as line hardware. Tall, slender poles, such as guyed structures, can require two-point lifting or other special rigging to prevent excessive deflection and/or stress during the lift. As the structure is being lifted, tag lines can be used to guide the structure to its foundation.

Correct pole orientation, including considerations for rake and/or precamber, should be identified prior to installation according to the assembly documents. Once the structure is in place, it should be checked for plumbness with a transit. Deflection caused by uneven solar heating of tubular steel poles is a known phenomenon that should be considered during assembly and erection of the structure. Steel poles are in their most natural state of straightness on cloudy days or in the very early morning hours when the temperature of the steel is the same around the full circumference of the pole.

C11.4 FRAME-TYPE STRUCTURES

The most common type of frame structure is the H-frame. Another typical frame structure is the four-legged A-frame, commonly used as a substation termination structure. The assembly process for frame structures is similar to that used for single-pole structures, and the same discussion and recommendations given in Section C11.3 apply.

C11.4.1 Slip Joints in Frames. When slip joints are used with direct embedment foundations, slip-jointed legs of frame structures should be assembled on the ground as described in Section C11.3.1 to allow variations in leg lengths caused by slip tolerance to be accommodated by adjusting embedment length. Structures with slip joints that are subject to uplift should utilize some type of locking device at each connection. These permanently installed locking devices may vary in design because of material finish and construction capabilities. Due to slip joint tolerances, field drilling may be required to accommodate these locking devices.

C11.4.2 Erection. Crossarms and cross bracing, if used, can be installed on the ground and the structure erected as a unit. Special care should be taken to maintain structure geometry. The correct distance between the legs should be ensured before tightening the connections.

A spreader bar or yoke should be used between the two legs of an H-frame-type structure during lifting. Tag lines or equipment such as bulldozers, trucks, or tractors can be used to guide the structure to the foundation.

Installation on anchor bolt foundations could require installing one leg on its foundation, and then moving the second leg to position for installation on its foundation. Chain hoists, winches, or other means may be used for this alignment as permitted by the owner. Care should be taken to protect the anchor bolt threads from damage during erection and alignment of the structure. Once placed on the foundation, the structure should be plumbed and the anchor bolts tightened.

Frame structures with single-piece legs, or flange-bolted leg joints, are recommended for aerial assembly applications. H-frame structures can be erected one leg at a time, followed by the top section consisting of the cross arm and static masts either assembled and lifted as a unit or lifted and assembled individually. Cross bracing can be added as the final pieces in the structure assembly.

Routine fabrication and construction tolerances require adjustment and alignment of members during assembly. Assembly of larger frames can require mechanical aid to deflect or rotate members to align connections. To facilitate the erection and assembly process, maximum adjustability should be maintained in a frame structure during assembly by leaving all connections loosely bolted. When assembly is complete, connection bolts should be tightened, and the structure should be checked for vertical alignment (either plumbed or raked as required by the line designer).

C11.4.3 Bolted Frame Connections. Frame structures usually have flange- and/or pin-type connections, each of which require their own installation technique. Single-bolt pin connections for H-frame crossarms, where the bolts are primarily used as shear connectors, may not have the same pretensioning requirements as flange bolts. Pretensioning these connections could be detrimental to the performance of the frame. For pretensioning of flange joint bolts in frame structures, refer to Section 11.3.2. Flange connection bolts should be pretensioned according to the structure designer's requirements. The installation of all connections should conform to the assembly documents.

C11.5 INSTALLATION ON FOUNDATION

For related information concerning helicopter erection, see Appendix E.

C11.5.1 Anchor Bolt and Base Plate Installation. Anchor bolts can be shipped from the fabricator either in preassembled clusters or loose bolts with templates. To ensure that a structure's base plate will fit on its anchor bolts, the following precautions should be taken:

- Handle the anchor bolt assemblies carefully during shipping and installation.
- Provide adequate support/stiffness to the anchor bolt assembly during handling and concrete placement.
- Perform dimensional checks including orientation, spacing, and projection of the anchor bolt assembly before and after the placement of concrete.

The anchor bolt and base plate foundation system typically uses two nuts to attach each anchor bolt to the pole base plate. A leveling nut is set below the base plate and tightened to the bottom of the base plate, and a top nut is set above the base plate and tightened to the top of the base plate. The base plate does not directly bear on the foundation surface, and bearing is not considered in the structure design.

Before structure erection, a leveling nut is first installed on each anchor bolt and turned down on the bolt to allow installation of the pole base plate at its approximate final elevation. During erection, the structure base plate is lowered onto the anchor bolts and set on the leveling nuts. The top nuts are then placed on each bolt, and all (top and bottom) nuts are then tightened to a snug-tight condition against the base plate. Final tightening should be in accordance with the assembly documents.

Industry practice as set forth by Section 9.3.3 permits the design of anchor bolts to ignore bending stress caused by shear when the distance between the concrete and base plate at each anchor bolt is not greater than twice the diameter of the bolt. To ensure the validity of this design assumption, the structure installation should provide for the clear distance between the concrete and the bottom of the base plate at each anchor bolt to be no greater than twice the diameter of the bolt. For example, when using 2.25 in. (57 mm) diameter anchor bolts, the bottom of the base plate should be no higher than 4.50 in. (114 mm) above the concrete at all anchor bolt locations.

Raking of poles, sloping of concrete surfaces, and construction variations can result in varying distances between the concrete and the bottom of the base plate at different anchor bolts. Nevertheless, the maximum permissible distance at the location with the greatest distance between the concrete and the bottom of base plate is twice the diameter of the anchor bolt. In the event that the distance between the concrete and base plate exceeds the dimension specified in the assembly documents, the distance should be reported to the structure designer for evaluation.

Once set on the anchor bolts, the structure should be checked for alignment, plumbness, or, in the case of a raked pole, proper structure rake. Adjustments can be made to top and bottom anchor bolt nuts as needed to provide the proper pole alignment. Rake should be checked prior to the installation of any conductors. If the pole is part of a frame structure, the anchor bolt nuts should be left in a snug-fit condition while the remainder of the structure is assembled and erected.

Once pole or structure assembly and erection are complete, all top and bottom anchor bolt nuts should be tightened in accordance with the assembly documents. Each nut should be checked to ensure it is in solid contact with the base plate. Tightness of all nuts should also be checked after line stringing.

Anchor bolt nuts can be secured to prevent loosening during service, if desired. The nuts may be secured by mechanically damaging the bolt threads, using a mechanical locking system, using a jam nut (a third nut set above the top nut and tightened onto the top nut), or applying a tack weld between the anchor bolt nut and the base plate. Because of the risk of heat damage to high-strength bolt material, welds should not be applied directly to the anchor bolt.

It is recommended that the gap area between the top of the concrete foundation and the bottom of the base plate be left ungrouted to provide adequate drainage and reduce maintenance. When grout is used it could potentially deteriorate over time, allowing crevices to form and moisture to penetrate, which could cause corrosion of the anchor bolts, leveling nuts, and/or underside of the base plate. If grouting is deemed necessary for any reason, then these installations should be inspected on a regular basis and appropriate measures should be taken to maintain the integrity of the grout and prevent moisture penetration.

C11.5.2 Direct-Embedded Poles. Specific recommendations and requirements should be made by the line designer as to the type of material and method of placement of backfill. This helps ensure that the in-service behavior of the pole is in accordance with design assumptions regarding pole rotation at groundline.

Typical backfill material consists of soil, crushed rock, or concrete. Other backfill materials, such as urethane foams, might also be used. The type of backfill material will typically be determined by the line designer based on the structure groundline reactions, soil conditions at the pole site, and other site- and project-specific parameters.

When setting the pole, care should be taken to match the excavation depth and diameter to the setting depth and hole diameter specified by the line designer. When setting the pole in the hole and prior to placement of backfill, the pole should be carefully checked to ensure the alignment and orientation of crossarms, insulators, conductors, and other pole attachments and features conform to the requirements of the line design as specified in the assembly documents.

Backfill should meet the requirements for type of material, placement procedure (maximum thickness of lifts), and compaction as set forth by the line designer. Care should be taken during the backfilling and compaction process to prevent damage to the protective coating of the embedded pole section. Any damage to the protective coating should be repaired in accordance with the assembly documents.

The owner should maintain a log detailing the placement of direct-embedded poles. For each pole, the log should list the actual excavated depth and diameter of the hole, any unusual conditions encountered with the excavation (cave-in, water, rock, and so forth), the backfill material used, the backfill placement procedure (lift thicknesses), and the compaction method.

C11.5.3 Embedded Casings. Direct-embedded casings should be installed in accordance with Section C11.5.2. Vibratory steel casings should be installed as discussed in Section C9.5. Before and after installation, the casing should be checked for plumbness and proper orientation. Depending on site access, either a crane or helicopter can be used with the vibratory equipment (see Appendix E for information on the use of helicopters). When the base plate connection detail is used, it should be installed in accordance with Section C11.5.1.

C11.6 GUYING

C11.6.1 Guy Anchor Location. The accurate location of guy anchors is critical to the proper distribution of loads in the structure. Changes in guy angles, either horizontal or vertical, and guy lengths can cause dramatic changes in structure forces from those predicted in design. It is vital that any field conditions that require a change in guy geometry be referred to the structure designer for review.

C11.6.2 Guy Installation. The structure designer should identify to the line designer the need for timely guy installation and any temporary guying required to provide structure stability before line completion. Some designs require immediate guy installation to resist even routine wind loading, whereas other designs use guys only to resist applied conductor and ground wire loads. Additional information for installation of guys can be found in Chapter 7 of *Design of Guyed Electrical Transmission Structures*, MOP 91.

Care should be taken to ensure proper installation of guy anchors in accordance with the owner's specification and recommendations from geotechnical reports. The owner may specify additional cathodic protection measures to minimize below-grade corrosion. A soil resistivity analysis should be included in the geotechnical report in conductive soils. Additional cathodic protection for the anchors should be considered in areas that are adjacent to underground pipelines.

The owner should provide installation directions for proper tensioning of guys in accordance with the structure designer's requirement for guy tensions. The quantity and size of guy wires, the corresponding location on the structure, and anchor locations should be identified in the assembly documents.

The assembly documents should include the guy installation procedure, the guy pretension if specified by the structure designer, and the means of measuring the force in each guy. The installation procedure may vary depending on the structure configuration, its application, and the number of guys on the structure. As a single guy is pretensioned, any changes in the previously pretensioned guys should be checked. A pull test of the guy anchor prior to guy pretensioning may be appropriate depending on the soil properties and the type of guy anchor used.

To ensure proper load distribution, guy-to-structure attachment details should be included in the assembly documents.

C11.7 POST-ERECTION PROCEDURES

For additional information concerning recommended maintenance practices, refer to Appendix E.

C11.7.1 Inspection. Structures assembled in the air, where joints were initially assembled loosely bolted, should have all joints tightened and inspected for conformance with the fabricator's requirements. Damage to protective coatings should be noted, and touch-up repairs should be made. Final checks of alignment and plumbness should be made.

Structure assembly and installation should be inspected to ensure conformance with the assembly documents. Such inspection should be performed prior to wire installation and include, as a minimum, the following:

- Connection fit and installation;
- Structure alignment, straightness, and direction and magnitude of rake/precamber;
- Crossarm and insulator alignment and hardware installation;
- Anchor bolt installation;
- Direct buried pole embedment depths; and
- Permanent or temporary guy installations.

C11.7.2 Grounding. In some cases, bonding jumpers may be required to provide electrical continuity across structural joints to ensure a continuous ground path through the structure.

C11.7.3 Coating Repair. The damaged area of a galvanized coating should be cleaned using a wire brush and solvent, if necessary, to remove rust, grease, and other foreign matter. When dry, the area should be coated with a cold galvanizing product, as approved by the owner, with as many coats applied as necessary to reach the required dry film thickness. Refer to ASTM A780 for additional information.

The damaged areas of paint coatings or below-grade coatings should be cleaned using a wire brush, scraper, and/or solvent as necessary to remove rust, grease, and other foreign matter. It may be desirable to lightly sand the edges of the repair area to feather the touch-up material into the existing coating. The damaged areas should be dry before coating. If damage is limited to the finish or topcoat, apply one coat of properly mixed coating to the required dry film thickness. If damage includes the primer, the appropriate touch-up primer should be applied to the required dry film thickness and allowed to cure properly before application of the topcoat. Care should be taken to ensure that the coating manufacturer's recommendations are followed during field application.

The damage areas of below-grade coating should be cleaned using a wire brush, scraper, or solvent to remove rust, grease, and other foreign matter. It may be desirable to lightly sand the edges of the area to be repaired to feather the touch-up area into the existing coating. The damaged areas should dry prior to coating. The contract specifications should specify the quantity of touch-up material that is to be provided. This touch-up material should be readily field applied and compatible with the factory-applied coating.

C11.7.4 Unloaded Arms. Conductors or ground wires attached to arms typically provide a vibration damping effect to the arms. When conductors or ground wires are not installed on the arms and/or crossarms concurrently with the line construction, such arms and/or crossarms become susceptible to damage from wind-induced oscillations in their unloaded state. The symmetrical shape of the arms and the absence of the vibration damping effect of attached conductors, ground wires, and assemblies can result in damaging oscillatory movements, even in relatively low wind velocities. The tension–compression cycling of the arms can cause fatigue cracking and arm failure. Stresses from wind-induced vibrations of the arm at its fixed end can exceed the allowable stress range in short-duration time periods. Special attention during construction is necessary to identify and document any vibration conditions.

When arms and/or crossarms are installed without the planned conductor or ground wires, it may be necessary to provide remedial measures, either in the original design and fabrication or after installation in the field, to minimize the oscillations. Examples of these measures may include installing internal or external damping devices, internal cables, weights, temporary tiebacks to fixed points, and insulator assemblies. Erecting structures with insulators and stringing travelers may also provide the necessary damping. The remedial action may require an iterative process because of the complexity of the phenomena. The structure designer should be consulted to determine what measures, if any, should be used for each specific circumstance.

For additional information concerning wind-induced vibration and oscillation of structures and members, see Appendix E.

C11.7.5 Hardware Installation. Aeolian vibration of conductors and static wires is a common occurrence. The severity of the vibration depends on the tension-to-strength ratio of the installed conductors, span lengths, wind speed, and average annual minimum temperature. It is common practice to require installation of vibration dampers within two weeks of conductor installation. Without damper installation, wind-induced oscillations could be transmitted from the conductors and cause vibration of the structure components.

APPENDIX A
NOTATION

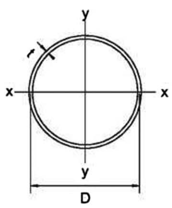
The following symbols are used in this standard:

A = Cross-sectional area, in.^{2} (mm^2)
A_{BC} = Total anchor bolt cage net cross-sectional area, in.^{2} (mm^2)
A_{eff} = Effective projected stress area of concrete foundation, in.^{2} (mm^2)
A_g = Gross cross-sectional area, in.^{2} (mm^2)
A_n = Net cross-sectional area, in.^{2} (mm^2)
A_r = Cross-sectional area at root of the threads, in.^{2} (mm^2)
A_s = Tensile stress area of bolt, in.^{2} (mm^2)
$A_{s(\text{req'd})}$ = Required tensile stress area of bolt, in.^{2} (mm^2)
A_t = Tensile stress area, in.^{2} (mm^2)
BR = Effective bend radius, in. (mm)
c = Distance from neutral axis to point where stress is checked, in. (mm)
C_c = Column slenderness ratio separating elastic and inelastic buckling
c_x = Distance from y–y axis to point where stress is checked, in. (mm)
c_y = Distance from x–x axis to point where stress is checked, in. (mm)
d = Diameter of bolt, in. (mm)
d_h = Diameter of hole, in. (mm)
D_o = Outside diameter of tubular section, in. (mm)
E = Modulus of elasticity, 29,000 ksi (200 GPa)
F_a = Permitted compressive stress, ksi (MPa)
f_a = Stress, in tension or compression, on a member, ksi (MPa)
F_b = Permitted bending stress, ksi (MPa)
f_b = Bending stress on a member, ksi (MPa)
f_{br} = Bearing stress, ksi (MPa)
F_c = Effective concrete tensile capacity, ksi (MPa)
F_{cr} = Critical stress for local buckling, ksi (MPa)
F_t = Permitted tensile stress, ksi (MPa)
$F_{t(v)}$ = Permitted axial tensile stress in conjunction with shear stress, ksi (MPa)
F_u = Specified minimum tensile stress, ksi (MPa)
F_v = Permitted shear stress, ksi (MPa)
f_v = Shear stress, ksi (MPa)
F_y = Specified minimum yield stress, ksi (MPa)
f'_c = Specified compressive strength of concrete at 28 days, ksi (MPa)
I = Moment of inertia, in.^{4} (mm^4)
I_{BCx} = Anchor bolt cage moment of inertia about the x–x axis, in.^{4} (mm^4)
I_{BCy} = Anchor bolt cage moment of inertia about the y–y axis, in.^{4} (mm^4)
I_x = Moment of inertia about the x–x axis, in.^{4} (mm^4)
I_y = Moment of inertia about the y–y axis, in.^{4} (mm^4)
J = Torsional constant of cross section, in.^{4} (mm^4)
K = Effective length factor
KL/r = Slenderness ratio
L = Unbraced length, in. (mm)
L_c = Minimum clear distance, parallel to load, from the edge of the hole to the edge of an adjacent hole or edge of the member, in. (mm)
L_d = Minimum development length (embedment) of anchor bolt, in. (mm)
l_d = Basic development length of anchor bolt, in. (mm)
L_e = Minimum distance, parallel to the load, from center of hole to edge of the member, in. (mm)
L_s = Minimum distance, perpendicular to the load, from center of hole to edge of the member, in. (mm)
M = Bending moment, in.-kip (mm-N)
M_t = Resultant groundline moment, in.-kip (mm-N)
M_x = Bending moment about the x–x axis, in.-kip (mm-N)
M_y = Bending moment about the y–y axis, in.-kip (mm-N)
n = Number of threads per unit length, in. (mm) total number of bolts
P = Axial load, tension, or compression, on member or guy, kips (N) actual force transmitted by bolt or pin, kips (N)
$P_{\max}$ = Maximum tension force permitted in the guy, kips (N)
Q = Moment of section about neutral axis, in.^{3} (mm^3)
r = Governing radius of gyration, in. (mm)
RBS = Minimum rated breaking strength of guy, kips (N)
s = Center-to-center spacing between bolt holes, in. (mm)
T = Torsional moment, in.-kip (mm-N)
t = Thickness of element, in. (mm)
T_s = Bolt tensile force, kips (N)
V = Shear force, kips (N)
w = Flat width of element, in. (mm)
x_i = Distance of bolt from the y–y axis, in. (mm)
y_i = Distance of bolt from the x–x axis, in. (mm)
α = Unit factor as specified in the text
β = Unit factor as specified in the text
Γ = Unit factor as specified in the text
γ = Ratio of required tensile area to the gross area of the anchor bolt
Θ = Unit factor as specified in the text
Φ = Unit factor as specified in the text
Ω = Unit factor as specified in the text

APPENDIX B
PROPERTIES OF VARIOUS TUBULAR SECTIONS

Approximate equations for commonly used section properties are shown in Figures B-1 through B-6. Figure B-7 illustrates the dimensions used in the section property equations.

Note: For polygon sections of Figures B-1 to B-6, all properties except flat width (w) assume sharp-cornered sections.



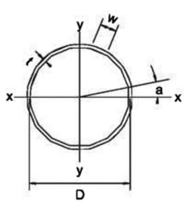
$A_g = 3.14 \bullet D \bullet t$ $\quad$ $r = 0.354 \bullet D$

$I_x = I_y = 0.393 \bullet D^3 \bullet t$ $\quad$ $\max \dfrac{Q}{It} = \dfrac{0.637}{D \bullet t}$

$C_x = 0.5 \bullet (D+t) \bullet \cos(a)$

$C_y = 0.5 \bullet (D+t) \bullet \sin(a)$ $\quad$ $\max \dfrac{C}{J} = \dfrac{0.637 \bullet (D+t)}{(D^3 \bullet t)}$

Figure B-1. Properties of round sections.



$A_g = 3.19 \bullet D \bullet t$ $\quad$ $r = 0.356 \bullet D$

$I_x = I_y = 0.403 \bullet D^3 \bullet t$ $\quad$ $\max \dfrac{Q}{It} = \dfrac{0.634}{D \bullet t}$

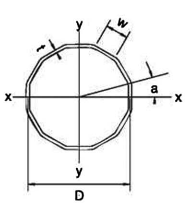
$C_x = 0.510 \bullet (D+t) \bullet \cos(a)$

$C_y = 0.510 \bullet (D+t) \bullet \sin(a)$ $\quad$ $\max \dfrac{C}{J} = \dfrac{0.628 \bullet (D+t)}{(D^3 \bullet t)}$

$w = 0.199 \bullet (D - t - 2 \bullet \mathrm{BR})$

$a = 11.25°$

Figure B-2. Properties of hexadecagonal (16-sided polygon) sections.



$A_g = 3.22 \bullet D \bullet t$ $\quad$ $r = 0.358 \bullet D$

$I_x = I_y = 0.411 \bullet D^3 \bullet t$ $\quad$ $\max \dfrac{Q}{It} = \dfrac{0.631}{D \bullet t}$

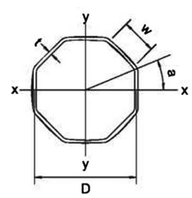
$C_x = 0.518 \bullet (D+t) \bullet \cos(a)$

$C_y = 0.518 \bullet (D+t) \bullet \sin(a)$ $\quad$ $\max \dfrac{C}{J} = \dfrac{0.622 \bullet (D+t)}{(D^3 \bullet t)}$

$w = 0.268 \bullet (D - t - 2 \bullet \mathrm{BR})$

$a = 15°$

Figure B-3. Properties of dodecagonal (12-sided polygon) sections.



$A_g = 3.32 \bullet D \bullet t$ $\quad$ $r = 0.364 \bullet D$

$I_x = I_y = 0.438 \bullet D^3 \bullet t$ $\quad$ $\max \dfrac{Q}{It} = \dfrac{0.618}{D \bullet t}$

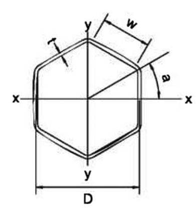
$C_x = 0.541 \bullet (D+t) \bullet \cos(a)$

$C_y = 0.541 \bullet (D+t) \bullet \sin(a)$ $\quad$ $\max \dfrac{C}{J} = \dfrac{0.603 \bullet (D+t)}{(D^3 \bullet t)}$

$w = 0.414 \bullet (D - t - 2 \bullet \mathrm{BR})$

$a = 22.5°$

Figure B-4. Properties of octagonal (8-sided polygon) sections.



$A_g = 3.46 \bullet D \bullet t$ $\quad$ $r = 0.373 \bullet D$

$I_x = I_y = 0.481 \bullet D^3 \bullet t$ $\quad$ $\max \dfrac{Q}{It} = \dfrac{0.606}{D \bullet t}$

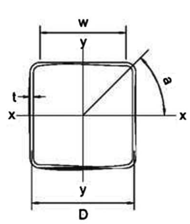
$C_x = 0.577 \bullet (D+t) \bullet \cos(a)$

$C_y = 0.577 \bullet (D+t) \bullet \sin(a)$ $\quad$ $\max \dfrac{C}{J} = \dfrac{0.577 \bullet (D+t)}{(D^3 \bullet t)}$

$w = 0.577 \bullet (D - t - 2 \bullet \mathrm{BR})$

$a = 30°$

Figure B-5. Properties of hexagonal (6-sided polygon) sections.



$A_g = 4.00 \bullet D \bullet t$ $\quad$ $r = 0.408 \bullet D$

$I_x = I_y = 0.666 \bullet D^3 \bullet t$ $\quad$ $\max \dfrac{Q}{It_t} = \dfrac{0.563}{D \bullet t}$

$C_x = 0.707 \bullet (D+t) \bullet \cos(a)$

$C_y = 0.707 \bullet (D+t) \bullet \sin(a)$ $\quad$ $\max \dfrac{C}{J} = \dfrac{0.500 \bullet (D+t)}{(D^3 \bullet t)}$

$w = (D - t - 2 \bullet \mathrm{BR})$

$a = 45°$

Figure B-6. Properties of square sections.

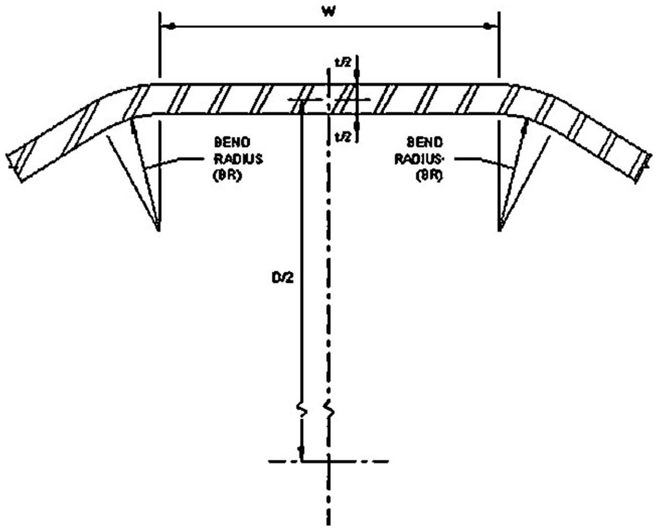


Figure B-7. Typical dimensions of polygon sections.

NOTATION

The following symbols are used in this appendix:

a = Angle between the x-axis and the corner of the polygon
A_g = Gross area
BR = Effective bend radius (actual or $4 \times t$, whichever is smaller)
C/J(max) = Value for determining maximum torsional shear stress
C_x = Distance from y-axis to point
C_y = Distance from x-axis to point
D = $D_o - t$ =Mean diameter (measured to midpoint of thickness across flats on polygonal sections)
D_o = Outside diameter (measured across flats on polygonal sections)
I = Moment of inertia
J = Polar moment of inertia
Q/It(max) = Value for determining maximum flexural shear stress
r = Radius of gyration
t = Thickness
w = Flat width of a side of a polygon

APPENDIX C
RESERVED FOR FUTURE EDITIONS

This appendix formerly contained information related to horizontal testing. Much of this information has been moved to Chapter 8 and the commentary for Chapter 8, eliminating the need for an appendix on horizontal testing. This appendix is reserved for future use.

APPENDIX D
HEADED ANCHOR BOLTS

Headed anchor bolts have headed or threaded and nutted ends embedded as cast-in-place anchors. The use of J- or L-hooked smooth bolts is not recommended for applications with significant tension loads due to less predictable behavior in tension tests. Anchorage of deformed bars with threaded and nutted ends may be conservatively considered as smooth-headed anchor bolts in resisting tension, ignoring any strength contribution from the bar deformations.

MATERIAL SPECIFICATIONS

Materials conforming to the following standard specifications are considered suitable for use under this standard:

- ASTM F1554 *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength*
- ASTM A615 *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*
- ASTM A563 *Standard Specification for Carbon and Alloy Steel Nuts*

All anchor bolt material should meet the energy impact requirements of Section 9.3.

DESIGN CONSIDERATIONS

The embedment depth, edge distance, and spacing should be in accordance with ACI 318.

ACI 318 identifies the potential modes of failure related to a headed anchor bolt subjected to tension. Similar modes of failure may apply to anchor bolts subjected to compression. Two failure modes have to do with steel strength and are within the scope of this document. These failure modes are identical to the failure modes for straight deformed anchor bolts.

The other failure modes, which have to do with concrete strength, are the responsibility of the foundation engineer and not within the scope of this standard. Some of these failure modes depend on the embedment depth of the anchor bolts. Because the anchor bolt length is often required to be determined prior to foundation design, the failure modes are discussed as follows:

- Steel strength in tension: Refer to Section 6.2.3 of this standard. In general, this failure mode controls anchor size.
- Steel strength in shear: Refer to Section 6.2.2 of this standard. In general, shear does not govern anchor bolt size for transmission poles, but combined shear and tension may govern in accordance with Section 6.2.4 of this standard.
- Concrete breakout strength in tension: This failure mode involves the concrete surrounding the anchor bolts separating from the foundation. For pier or caisson foundations, as with straight deformed anchor bolts, concrete breakout strength is limited by the diameter or width of the foundation. Therefore, it is often necessary to develop the vertical foundation reinforcing on each side of a 35° theoretical cone as shown in Figure D-1. The development length (l_d) of the vertical reinforcement is provided in ACI 318. Sufficient anchor bolt embedment length (l_e) must be provided to allow the foundation designer to develop the necessary reinforcement. The required embedment length depends on the development length of the vertical reinforcing and the distance between the anchor bolts and the vertical reinforcing. Unless the owner provides specific anchor bolt requirements or foundation parameters, anchor bolts should be provided with a minimum embedment length of 25 bolt diameters ($l_e = 25d$). For high-strength anchors, a longer development length may be required for development with the foundation reinforcing in piers or caissons where concrete breakout strength is limited by the diameter or width of the foundation.
- Concrete breakout strength in shear: Concrete breakout occurs when a deformed or headed anchor bolt is loaded in shear and is close to the edge of a foundation. For multiple anchor bolt patterns, concrete breakout due to shear is rarely a governing failure mode, especially if transverse reinforcing is provided.
- Concrete pullout strength in tension: Pullout strength is limited by the concrete bearing stress above the head or nut of the anchor bolt. The net area of the bolt head or nut is usually sufficient to develop the strength of the bolt for bolts with yield strengths below 60 ksi (414 MPa) when concrete strengths are 4 ksi (28MPa) or more. Higher-strength bolts may require plate washers to develop the required pullout

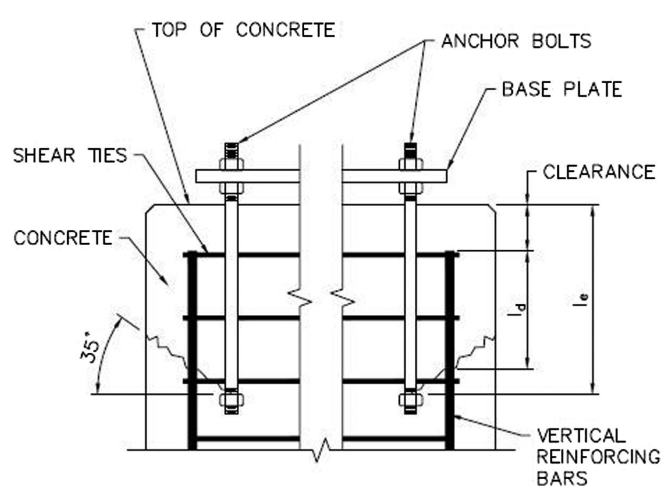


Figure D-1. Concrete breakout strength.

strength. The contributing width of plate washers should be limited to their thickness.

- Concrete side-face blowout in tension: Limited side cover and high bearing stresses on the net area of a bolt head or nut may result in spalling or blowout on the free concrete surface adjacent to the embedded head. This failure mode may be avoided by increasing the nut bearing area to result in the side-cone blowout strength matching the strength of the anchor bolt.
- Concrete pryout strength in shear: Pryout occurs with short, stiff anchor bolts when concrete spalling occurs at the bottom of the anchor as it kicks back in the direction opposite the loading. This failure mode is only applicable for very short anchors. Concrete pryout is not a consideration for anchor bolts meeting the minimum $25d$ embedment length recommended above when the anchor bolts are placed within a reinforcing cage in a pier or caisson.

OTHER CONSIDERATIONS

- The minimum edge distance and center-to-center spacing for anchor bolts should meet the ACI 318 requirements for reinforcing. For headed anchor bolts, the recommended minimum center-to-center spacing of anchor bolts is four bolt diameters. This spacing minimizes splitting and facilitates concrete placement and consolidation around the anchor bolts.
- The embedded portion of an anchor bolt may be subjected to torque when a properly tightened leveling nut is not used. Under these conditions, the minimum center-to-center spacing and minimum edge distance should be six bolt diameters.
- A ductile failure mode, if required, may be obtained by providing a concrete strength greater than the ductile anchor bolt capacity or by fully developing the reinforcing bars on both sides of the 35° cone shown in Figure D-1.
- Concrete punch-out below headed anchor bolts can occur when base plates are supported on leveling nuts only (without grout) and the thickness of concrete below the embedded bolt head or nut is relatively thin. The thickness of the slab below the lower anchor nut should be sized to prevent punching shear.
- Templates are often used to maintain the position of anchor bolts during concrete placement. Templates can be provided for the top and bottom of the anchor bolts to form an anchor bolt cage. The templates should have a large center opening in order not to hinder the placement of concrete. Embedded templates can be ignored in concrete strength calculations; however, templates can be used to increase the bearing area of a head or nut increasing concrete breakout, pullout, and side-face blowout strengths.

APPENDIX E
ASSEMBLY AND ERECTION

INTRODUCTION

This appendix provides supplemental information to Chapter 11 concerning the assembly and erection of steel transmission pole structures.

STRUCTURE ASSEMBLY

The assembly should conform to the assembly documents, using methods and equipment that will not cause damage or distortion of any part of the structure. Methods of assembly and erection may be subject to review by the owner.

Whether the pole is assembled on the ground or in the air depends on right-of-way considerations and thus is usually left to the discretion of the line contractor. Most often, line contractors elect to assemble the structures on the ground. Ground assembly involves assembling the pole sections, arms, crossarms, cross bracing, and other miscellaneous hardware prior to erection. Such assembly is usually performed using blocking material so that all attachment points are accessible and all attachments can be added without the need to rotate the structure. H-frame structures should be framed with the connection bolts left loose until all the members are joined. For multipole structures, a leg spacing dimensional check should be made after final tightening of bolts and prior to erection.

When assembly is performed in the air by crane, it is usually accomplished by assembling a series of partially assembled components. Structures could be assembled member by member, but this technique would be quite time consuming. To ensure proper fit, inspection and dimensional checks of all mating components/subassemblies should be made prior to the lifting operation. With this type of assembly, it is critical that the orientation and plumbness of each subassembly be checked with a transit after every step of the assembly. Aerial assembly of all slip joints should be performed using the appropriate jacking device as discussed in Sections 11.3.1/C11.3.1 and 11.4.1/C11.4.1 of this standard. Aerial jacking of slip joints will usually require the use of a large aerial basket or other form of manlift equipment.

All finish touch-up should be done prior to erection, whether the structures are assembled on the ground or in the air. Then, once the structure is erected, it should be thoroughly inspected again prior to stringing.

ARM CONNECTION ASSEMBLY

Through-plate arm connections are one of the most common types of arm connections. Whenever the structure site is suitable, arm connection assembly is performed on the ground prior to structure erection. An example procedure for attaching the arms to the structure is as follows:

1. Center the arm mount channel on the arm connection such that the gaps are approximately equal. This is typically done with the aid of either a boom truck or a forklift, using nylon slings to protect the finish. Without centering, proper bolt tightening is difficult to achieve, and the arm tips might become offset from the center of the structure, requiring arm reassembly after erection.
2. Install all of the connection bolts.
3. Finger tighten all the nuts onto all the bolts.
4. Tighten all the nuts in an alternating fashion between each side of the connection and position of the bolts. For example, move from the top nut on the right side of the connection to the bottom nut on the left side, then to the top nut on the left side, then to the bottom nut on the right side, and so on until all the nuts are evenly tightened.
5. Perform final tightening of the nuts as specified in the assembly documents using the same alternating sequences as described in Step 4. If, according to the assembly documents, the plates are to be in direct contact and gaps are present, consult the design engineer to determine acceptable methods to address these gaps, which could possibly include washers or shims.
6. Repeat Step 5 as the sequential tightening of the nuts may loosen nuts that were previously tightened.

HELICOPTER ERECTION

The availability of helicopters with larger load capacities, innovations in helicopter construction and maintenance techniques, and the increasing need to construct and maintain transmission lines with the least possible environmental impact have led to an increased use of helicopters for both line construction and maintenance. When assessing the practicality of using helicopter erection, it is important to consult with helicopter specialists who are experienced in the transport and setting of transmission structures.

Helicopter erection requires coordinated efforts between the owner, fabricator, and helicopter contractor regarding the techniques to be used, weight limitations, and other similar items unique to helicopter lifts. Incorporating appropriate guides and lifting details into the structure design and ensuring liftable components requires the coordinated effort of all parties.

The decision of whether to use helicopters for erection needs to be determined as early as possible, but only after a feasibility study has been conducted that has given proper consideration to the economics of such construction, the planning time that will be required, and the availability of the necessary equipment. In general, such studies involve developing some preliminary structure designs. There are significant differences in the lifting capability of different helicopter models, so the preliminary designs need to reflect the limitations of the particular helicopter that is planned for use. Also, if there is flexibility in the spotting of structures within the line design, the structure designs can often be optimized around varying loading and structure height combinations, which could result in the use of more structures that are shorter in height, but at a lower overall installed cost. The following items are some of the factors that need to be considered when assessing the practicality of using helicopter construction:

- Number and size of available staging/storage yards: Larger and/or multiple yards are advantageous for staging assembled structures.
- Number of lifts required to assemble a complete structure: A single lift of a completely assembled structure is ideal. However, multiple lifts are commonplace for the higher-voltage structures and, when using multiple lifts, special guides will be needed to facilitate the assembly.
- Published rated lift capacities for helicopters: These are typically based on their operation at sea level and an ambient air temperature, 60 °F (15 °C). These capacities need to be adjusted when construction is performed at higher elevations and/or higher ambient temperatures. When such is the case, more lifts may be required and/or restrictions may need to be placed on the time of day for the lifts.
- Weight: When planning lifts, include the structure and all of the attachments (i.e., insulators and rigging).
- Weights provided by the fabricator's erection drawings are theoretical. Actual material thicknesses and the finish can significantly increase the actual weight by as much as 15% to 20% over the calculated weights shown on the drawings.
- Actual flying distances between the staging yard(s) and the structure sites.
- Project location and the associated cost of helicopter mobilization and transportation to and from the construction site: This may make such construction prohibitively expensive for small projects.

Extensive planning time is necessary to avoid even the smallest delay, because delays tend to be expensive when helicopters are involved. Special permits might be required, which could impact schedules. However, helicopters could eliminate the need for building special access roads to accommodate heavy construction equipment but could also require additional planning for structural inspection and maintenance during the life of the line.

Erection time is usually significantly shortened by the use of helicopters. This means that material shipments and deliveries must be closely monitored to avoid negatively impacting the construction schedule. Helicopter construction also requires crews to have structures assembled and ready for erection prior to the helicopter arriving at the site. The priority must be erecting structures in the most cost-effective manner.

In addition to erecting steel transmission structures, these helicopters are used for logging, fighting forest fires, and the lifting of materials when conventional cranes are impractical. Thus, their availability is somewhat dependent on the demands of these other uses. When only helicopters with smaller-than-planned lift capacities are available, a complete review of structure designs and lifting plans should be performed.

Structure and component placement techniques vary when multisection tubular steel structures are involved. This is true for both slip joint and flange-connected structures. The owner, the helicopter contractor, and the structure designer should all be consulted when developing a placement plan.

When lifting structures and components, a sling is typically attached to an electrically operated hook on the underside of the helicopter, and a load cell is used to monitor the effective weight of the payload, including the effects of the aerodynamic drag and rotor downwash. The hook is controlled by the helicopter pilot, who can release the load should it become necessary to ensure safety. The slings should be attached to the structure in a manner that prevents overstress, excessive deflection, or distortion of the structure.

Guyed structures can be flown with guys already connected to the structure, but the guys should be coiled and attached to the structure to prevent contact with the ground, trees, fences, or other objects during flight. Coils should be reachable from the ground once the structure is set. Each guy should be marked to identify the ground anchor to which it corresponds.

Unguyed structures should be secured on their foundations before their release by the helicopter. Guyed structures should have all guys required for stability, as specified in the assembly documents, secured to their anchors before being released by the helicopter. For structures set on anchor bolt foundations, the bottom leveling nuts should be properly set in advance of structure placement. To facilitate structure placement on anchor bolts, it is common to include a single anchor bolt with a longer projection length than the other anchor bolts that can then be used to help guide the base plate into place. In addition, structures should be grounded to allow discharge of any static buildup prior to workers touching them. Typically, workers with tag lines are used to assist the helicopter pilot in guiding structures onto their anchor bolts. Before the helicopter releases the structure, a sufficient number of top anchor bolt nuts should be installed and hand tightened to secure the structure. When setting a framed structure, one leg is first secured in a similar fashion as to that used for a single pole structure; then the remaining leg, or legs, are set and secured.

MAINTENANCE AND INSPECTION

After installation, a routine program of structure inspection is recommended for all steel transmission pole structures. This program should be designed to guard against structural degradation resulting from environmental wear, corrosion, accidental damage, and vandalism. Included among the items in a typical routine inspection program are the following:

- Inspection of protective coatings and touch-up of damaged areas;
- Inspection of self-weathering structures for localized areas of continual moisture and excessive corrosion;
- Visual inspection of bolted connections and spot check of bolt tightness in selected connections;
- Visual inspection of welded connections and seams to detect cracks;
- Visual inspection of the groundline area to ensure that soil and vegetation are not in contact with or otherwise creating a corrosive environment for the steel structure;
- Inspection of climbing hardware and attachments for continued integrity and to ensure inaccessibility to unauthorized climbing; and
- Visual inspection of grounding and cathodic protection connections.

WIND-INDUCED VIBRATION

Structures and Members. Vortex shedding of wind forces by structures and structure members can result in oscillation of slender arms, poles, or other elements. Tall steel pole structures used for substation lightning masts are examples of a structure type commonly subjected to noticeable wind oscillations. Where such movement is predicted or observed with a pole or a structure member, the installation of vibration damping devices might be considered to prevent fatigue damage. When the potential for such movement is anticipated by the structure designer, changing the bending stiffness of the member (*EI*) can be considered as an alternative to the installation of damping devices.

Attached Conductors and Static Wires. Aeolian vibration of aerial conductors and static wires is a common occurrence. The severity of the vibration depends on the tension-to-strength ratio of the installed conductors, span lengths, wind speed, and average annual minimum temperature. Where such vibrations are calculated to be of such a magnitude range that wire damage could occur, it is common practice to install vibration dampers. The use of these dampers should also be considered to prevent structural damage from vibrations transmitted from the wires and to prevent audible structure noise that can result from conductor and static wire vibration.

APPENDIX F
SHAFT-TO-FOUNDATION CONNECTION

BASE PLATES: ANALYSIS CONSIDERATIONS

Currently, there are no industry standards that provide specific requirements for the analysis of base plates for tubular steel transmission pole structures. Most fabricators have developed proprietary analysis procedures. Because these procedures account for a fabricator's own specific detailing and manufacturing practices, no two are the same.

In an effort to meet its long-term goal to develop a complete base plate analysis methodology for this standard, the Standards Committee decided to provide a method that is generally conservative and can be used in lieu of other proprietary and confidential design methods. The following discussion describes the primary issues that need to be considered when developing an appropriate methodology for analyzing base plates. The common bend-line design method assumes that the base plate is sufficiently rigid to resist full bending. Holes that are used in some designs to accommodate galvanizing drainage may be so large as to alter the stress distribution in the base plate. Therefore, when a drainage hole exists in the center of a base plate, its effect should be considered in the base plate analysis. These are basic guidelines only and should not be construed as a complete methodology for the design of base plates.

To accomplish this, the Standards Committee solicited reference designs from all represented fabricators on the Standards Committee. Twenty-three unique designs from three fabricators were submitted. Although these submittals represented a wide variety of the most common types of base plate configurations, including evenly spaced anchor bolts, clustered anchor bolts, double-ring anchor bolts, 4-anchor bolts, 12-sided base plates, square base plates, round base plates, rectangular base plates, drainage holes, and no drainage holes with thicknesses ranging from 1 1/4 to 6 in. (32 to 152 mm), they do not necessarily cover all applications. Because of the confidentiality requested by the fabricators, the sources of the designs were not revealed, and the supporting data are held confidential by an independent subcommittee.

Base plate thickness analyses were made using the method outlined in ASCE 48-05. The Standards Committee agreed that the effective bend-line lengths in ASCE 48-05 should be limited, and a 45° limit was used (Figure F-1). A comparison of these results to the thicknesses provided in the fabricator's designs is provided in Table F-1.

Review of the comparative results shows that the thicknesses for the 4-bolt base plates are significantly different than the manufacturer's results, and 11 of the 23 designs were as much as 1.5 in. (38 mm) thinner than the fabricator's results. The Standards Committee determined that the ASCE 48-05 method does not produce results that are representative of various fabricators' designs and that further investigation of this issue is warranted.

The Standards Committee researched many documents in an effort to find an appropriate method, including

- AISC Design Guide 1, *Column Base Plates*, 1990 and 2006;
- AISC 360-05, *Specification for Structural Steel Buildings*;
- TIA-222-G, *Structural Standard for Antenna Supporting Structures*;
- *Design of Welded Structures*, O. W. Blodget; and
- Technical Manual 1, *Design of Monopole Bases*, D. Horn.

Of these five documents, only one provided a method applicable for a tubular shape to base plate welded connection as commonly used for tubular steel transmission poles, Horn (2004). The conclusion of that document suggests that the ASCE 48-05 method with the 45° effective bend-line limitation is appropriate. Because this method does not represent the submitted designs, the following additional modifications to developing an effective bend line were considered:

- Removal of all vertex effective bend lines,
- Varying the 45° effective bend-line limitation,
- Extending the effective bend line by 1.5 × bolt diameter to account for the bolt and body of the nut,
- Reducing the effective bend-line moment arm by 0.75 × bolt diameter to account for the body of the nut, and
- Various options with the 4-bolt base plates.

None of these options provided a better method of correlation with the various fabricators' methods.

A simple methodology was developed that correlated reasonably well with the fabricators' designs. This methodology, which is referred to as the wedge method, is only one method for analyzing base plates and should not be construed as being the only appropriate method for the design of base plates. The following elements are used in this methodology for analyzing base plates:

- Effective bend lines only occur on faces of the pole.
- Effective bend lines are the length of each face.
- Bolts that act on each face are those that fall within a wedge created by extending the vertexes from the center of the shaft radially.
- Bolt holes that fall on a vertex act half on one face and half on the other face.
- This method assumes that the base plate is sufficiently rigid to resist full bending. Large-diameter holes in the base plate may alter the stress distribution and should be considered in the base plate analysis.

Resultant loads for each of the anchor bolts are needed to calculate the stresses in a base plate. All load cases need to be considered. It is normally assumed that these anchor bolt loads produce a uniform bending stress, f_b, along the effective portion of each of the bend lines analyzed. To calculate the stress along

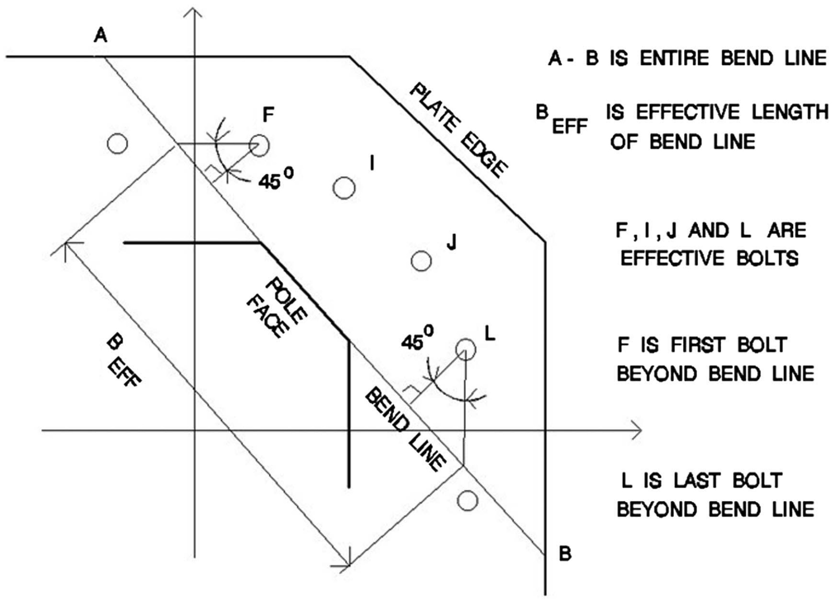


Figure F-1. Example of possible bend lines.

Table F-1. Manufacturer Designs versus ASCE 48-05 Method.

Base plate	Manufacturer's Design Thickness (in.)	ASCE 48-05 Thickness (in.)	Pole Configuration	Bolt Pattern	Thickness Difference (in.)	Rounded Thickness (in.)	Rounded Thickness Difference (in.)
1-1	3	2.826	12 Sided	14 Bolts	−0.174	3	Same
1-2	3	3.76	12 Sided	8 Bolts	0.76	4	1
1-3	2.25	2.594	12 Sided	8 Bolts	0.344	2.75	0.5
1-4	2.25	3.347	12 Sided	4 Bolts	1.097	3.5	1.25
1-5	2	3.389	12 Sided	4 Bolts	1.389	3.5	1.5
1-6	5	4.25	12 Sided	62 Bolts Double Ring	−0.75	4.25	−0.75
1-7	6	5.009	12 Sided	58 Bolts Double Ring	−0.991	5.25	−0.75
1-8	4	3.533	12 Sided	44 Bolts	−0.467	3.75	−0.25
1-9	5.75	4.763	12 Sided	40 Bolts Double Ring	−0.987	5	−0.75
2-1	3.25	2.955	12 Sided	16 Bolts	−0.295	3	−0.25
2-2	4.5	3.795	12 Sided	52 Bolts	−0.705	4	−0.25
2-3	4.25	3.38	12 Sided	32 Bolts	−0.87	3.5	−0.75
2-4	4.25	3.439	12 Sided	32 Bolts	−0.811	3.5	−0.75
2-5	4	3.105	8 Sided	14 Bolts	−0.895	3.25	−0.75
2-6	3.25	2.447	12 Sided	14 Bolts	−0.803	2.5	−0.75
2-7	2.25	3.218	12 Sided	8 Bolts	0.968	3.25	1
2-8	2.75	3.671	12 Sided	4 Bolts	0.921	3.75	1
2-9	2	2.793	12 Sided	4 Bolts	0.793	3	1
2-10	1.25	1.658	8 Sided	4 Bolts	0.408	1.75	0.5
3-1	2.44	2.499	12 Sided	20 Bolts	0.059	n/a	n/a
3-2	2.38	2.427	12 Sided	20 Bolts	0.047	n/a	n/a
3-3	2.378	2.444	12 Sided	20 Bolts	0.066	n/a	n/a
3-4	2.52	2.763	12 Sided	20 Bolts	0.243	n/a	n/a

Note: SI conversion: 1 in. = 25.4 mm.

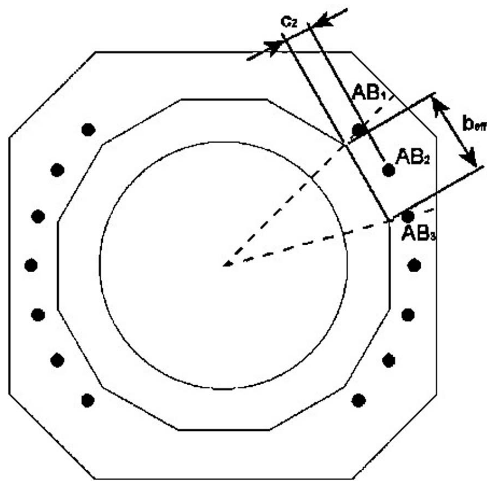


Figure F-2. Bend line example.

any bend line, the following parameters need to be established (Figure F-1)

where

c_i = Shortest distance from the center of each anchor bolt (i) to the bend line at the face of the pole,

b_{eff} = Length of bend line (depending on the shape of the pole), and

BL = Bolt load.

Figure F-2 shows one of twelve possible bend lines on that shaft and base plate design. The general bending stress, f_b, for an assumed bend line can be calculated by

$$f_b = \left(\frac{6}{b_{\text{eff}}t^2}\right)(\text{BL}_1c_1 + \text{BL}_2c_2 + \cdots\text{BL}_kc_k) \tag{F-1}$$

The bending stress, f_b, for the assumed bend line in Figure F-2 can be calculated by

$$f_b = \left(\frac{6}{b_{\text{eff}}t^2}\right)(1/2\text{BL}_1c_1 + \text{BL}_2c_2 + 1/2\text{BL}_3c_3) \tag{F-2}$$

where t is the base plate thickness, and BL is the effective bolt load.

The minimum base plate thickness is determined by keeping f_b below the yield stress F_y. To determine $t_{\min}$, Equation F-1 can be rewritten as

$$t_{\min} = \sqrt{\left(\frac{6}{b_{\text{eff}}F_y}\right)(\text{BL}_1c_1 + \text{BL}_2c_2 + \cdots\text{BL}_kc_k)} \tag{F-3}$$

When the wedge method was applied to the 23 various base plate designs submitted by the fabricators, it was found that the results more closely correlated to the fabricators' base plate thicknesses. These results are tabulated and compared to the manufacturers' designs in Table F-2.

Although no method yields identical results to all fabricators' proprietary methods, the wedge method closely correlates with the data submitted from the three fabricators that participated in this research effort.

This method was developed for eight- and twelve-sided shafts on base plates. The information for eight-sided poles was limited. There is not enough information at this time to extrapolate these findings for shafts of four sides, sixteen or more sides, or round shafts.

Table F-2. Manufacturer Designs versus Wedge Method.

Base plate	Manufacturer's Design Thickness (in.)	Pole Configuration	Bolt Pattern	Wedge Thickness Difference (in.)
1-1	3	12 Sided	14 Bolts	Same
1-2	3	12 Sided	8 Bolts	Same
1-3	2.25	12 Sided	8 Bolts	0.25
1-4	2.25	12 Sided	4 Bolts	Same
1-5	2	12 Sided	4 Bolts	0.25
1-6	5	12 Sided	62 Bolts Double Ring	0.25
1-7	6	12 Sided	58 Bolts Double Ring	0.25
1-8	4	12 Sided	44 Bolts	0.25
1-9	5.75	12 Sided	40 Bolts Double Ring	0.5
2-1	3.25	12 Sided	16 Bolts	0.25
2-2	4.5	12 Sided	52 Bolts	−0.25
2-3	4.25	12 Sided	32 Bolts	Same
2-4	4.25	12 Sided	32 Bolts	Same
2-5	4	8 Sided	14 Bolts	−0.25
2-6	3.25	12 Sided	14 Bolts	0.5
2-7	2.25	12 Sided	8 Bolts	Same
2-8	2.75	12 Sided	4 Bolts	−0.5
2-9	2	12 Sided	4 Bolts	Same
2-10	1.25	8 Sided	4 Bolts	Same
3-1	2.44	12 Sided	20 Bolts	0.5
3-2	2.38	12 Sided	20 Bolts	0.5
3-3	2.378	12 Sided	20 Bolts	0.5
3-4	2.52	12 Sided	20 Bolts	0.5

Note: SI conversion: 1 in. = 25.4 mm.

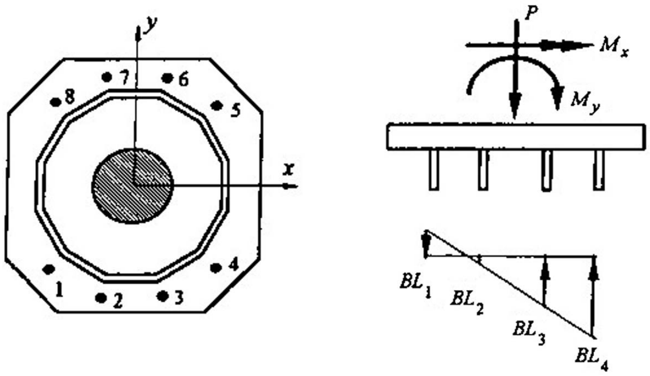


Figure F-3. Anchor bolt load calculation.

CALCULATION OF ANCHOR BOLT LOAD

The location of the anchor bolts is usually a function of the required clearance between the anchor bolt nuts and the pole, the minimum spacing allowed between anchor bolts, and the number of bolts required to resist the load. The clearance requirements should be in accordance with AISC, keeping in mind the base weld detail and manufacturing tolerances. The minimum spacing between anchor bolts should be in accordance with ACI 318. The number of bolts is dependent on the reactions at the base of the pole as well as the location of the bolts.

It is commonly assumed that the base plate behaves as an infinitely rigid body, and thus the axial load in any one anchor bolt (i), BL_i, can be calculated as follows:

$$\mathrm{BL}_i = \left(\frac{P}{A_{BC}} + \frac{M_x y_i}{I_{BCx}} + \frac{M_y x_i}{I_{BCy}}\right) A_{x(i)} \tag{F-4}$$

where

P = Total vertical load at the base of the pole,
M_x = Base bending moment about x–x axis,
M_y = Base bending moment about y–y axis,
x_i = Perpendicular distance from y–y axis to anchor bolt,
y_i = Perpendicular distance from x–x axis to anchor bolt,
$A_{n(i)}$ = Net area of anchor bolt (i),
A_{BC} = Total anchor bolt cage net cross-sectional area,
$I_{BCx} = \sum_{i=1}^{n} \left(A_{n(i)} y_i^2 + I_i\right)$ = anchor bolt cage moment of inertia about x–x axis,
$I_{BCy} = \sum_{i=1}^{n} \left(A_{n(i)} x_i^2 + I_i\right)$ = Anchor bolt cage moment of inertia about y–y axis,
n = Total number of anchor bolts, and
I_i = Moment of inertia of anchor bolt.

Because the moment of inertia of an individual anchor bolt, I_i, is normally a very small percentage of the total anchor bolt cage moment of inertia (I_{BCx} or I_{BCy}), this term is often ignored when calculating I_{BCx} and I_{BCy}. Figure F-3 illustrates the use of Equation (F-4) in establishing individual anchor bolt loads for a given pole base reaction.

APPENDIX G
CORROSION PROTECTION AND FINISH CONSIDERATIONS

INTRODUCTION

Corrosion protection and finish are important considerations when designing a line using steel structures. Exposure, application requirements, and appearance all affect the selection of appropriate corrosion protection and finish systems. With the many options and variations available for these systems, it is highly recommended that the line designer consult with steel pole fabricators during the design and selection process.

The most commonly used corrosion protection and finish options used for steel poles include paint and coating systems, galvanizing, weathering steel, and metalizing. ASTM, Steel Structures Painting Council (SSPC), and other industry standards and specifications can be used to assist the line designer in selecting the appropriate corrosion protection and finish measures as required.

CORROSION MECHANISMS

Because there are many definitions and descriptions of corrosion, the American Galvanizers Association (AGA) offers numerous publications that address this topic extensively. AGA (2002a) provides many of the key figures and descriptions of corrosion mechanisms detailed subsequently.

Galvanic Corrosion. Corrosion of metals is an electrochemical process that involves both chemical reactions and the flow of electrons. A basic electrochemical reaction that drives the corrosion of metals is galvanic action.

Two primary types of galvanic cells cause corrosion: bimetallic couple and concentration cell. A bimetallic couple (Figure G-1), which is the most common corrosion mechanism affecting steel poles, is like a battery consisting of two dissimilar metals immersed in an electrolyte solution. An electric current is generated when the two electrodes are connected by an external continuous metallic path.

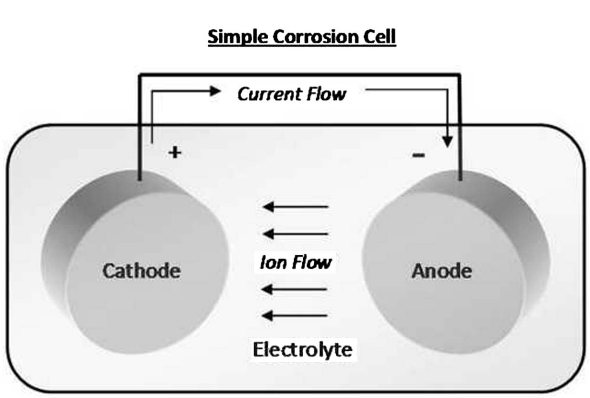


Figure G-1. Bimetallic couple.

In a galvanic cell, the following four factors are necessary for corrosion to occur:

- Anode: electrode where the anode reaction generates electrons. Corrosion occurs at the anode.
- Cathode: electrode that receives electrons. The cathode is protected from corrosion (i.e., by the anode).
- Electrolyte: conducting medium through which ion current is carried. Electrolytes include aqueous solutions of acids, bases, and salts.
- Return current path: metallic pathway connecting the anode and the cathode. It is often the underlying metal.

The aforementioned four factors contribute to the basis of corrosion and corrosion prevention. Removing any one of these factors stops the current flow and therefore corrosion does not occur.

Figure G-2 lists metals and alloys in decreasing order of electrical activity. Metals nearer the top of the figure are referred

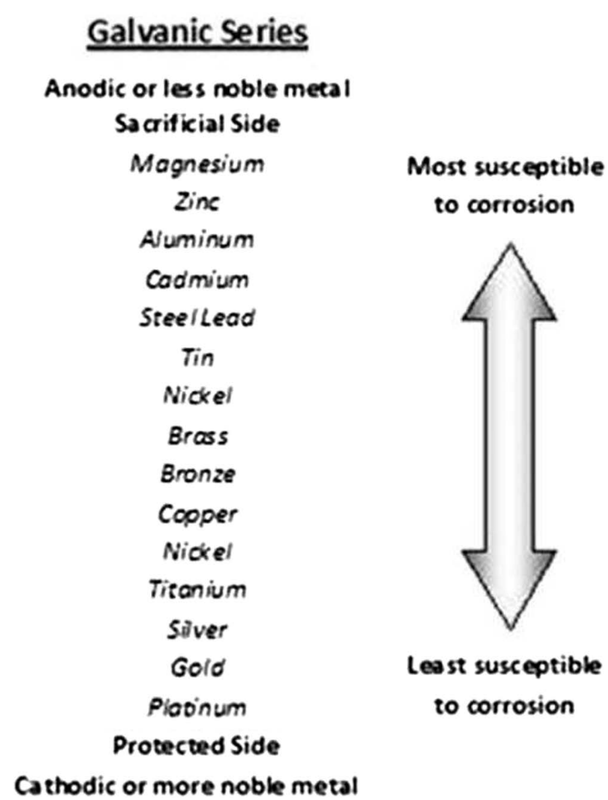


Figure G-2. Arrangement of metals in the galvanic series.

to as *less noble* metals and have a greater tendency to lose electrons. Therefore, these metals offer protection to the *more noble* metals found lower on the list.

CORROSION PROTECTION AND FINISH OPTIONS

For steel pole structures, there are many corrosion and finish options to choose from, including paint and coating systems, galvanizing, weathering steel, and metalizing.

Paint and Coating Systems. A large variety of paint and coating systems are available that not only provide corrosion protection for the underlying steel but also offer a choice of color. Regardless of which paint or coating system is actually used, it should provide the desired corrosion protection features and integrate well with the production system of the steel pole fabricator.

Any paint or coating system selected for use on steel poles should meet the following criteria:

- Have proven reliability based on accelerated laboratory tests and outdoor exposure tests.
- Be durable and abrasion resistant.
- Provide good cathodic protection.
- Have consistent application characteristics.
- Allow application by air or airless spraying.
- Be fast-drying to allow the product to be handled or shipped within a reasonable length of time after paint application.
- Have a topcoat that is highly resistant to chalking and fading.
- Allow for easy field touch-up or coating repair if required.

Above-Grade Protection. For above-grade corrosion protection of steel poles, superior paint systems use zinc-rich primers. These types of primers offer cathodic protection to the underlying steel.

Below-Grade Protection. The embedded portions of steel structures typically require additional corrosion protection. In most instances, a thick coating of polyurethane or epoxy material over the embedded steel provides significant protection. In areas known to have high corrosion rates on buried steel items such as ground rods or anchor rods, other measures, such as cathodic protection or a welded steel ground sleeve, may be considered.

Special attention should be paid to the groundline area of a direct-embedded steel pole. Depending on soil chemistry, water content, and density of surrounding foliage, the potential for corrosion can range from very low to very high. The below-grade corrosion coating or steel ground sleeve should extend well above and well below groundline [i.e., typically from 2 to 4 ft (0.6 to 1.2 m)].

Surface Preparation. For the paint or coating system to perform as designed, it is imperative that the base metal surface be prepared in accordance with the paint or coating supplier's recommendations. For most paint or coating systems, an initial blast cleaning of all exposed surfaces is required. Where blast cleaning is specified, it should be done in accordance with the latest edition of the SSPC's *Surface Preparation Specification* (SSPC-SPs). The typical requirement is for SP 6 or SP 10 blast cleaning.

Application. Paint or coating should be applied in strict accordance with the supplier's recommendations. Sufficient touch-up material should be provided with the product to allow for field repair of normal damage caused by transportation and handling.

Inspection. The steel pole fabricator should check paint or coating thicknesses to verify that the minimum dry film thickness requirements specified by the paint or coating supplier are met. In addition, a thorough visual inspection should be done for the purpose of detecting pinholing, cracking, or other undesirable characteristics.

Paint or Coating Over Galvanizing. When a paint or coating system is to be applied after a steel pole has been hot-dip galvanized, the painting or coating process should be done as soon as practical after galvanizing. Before applying the paint or coating, a light blast cleaning of the galvanized surface is recommended to remove any contaminants and to provide a better anchor pattern for the paint or coating to adhere to. The application of a vinyl wash primer before painting or coating also typically improves the adhesion characteristics of the paint or coating.

Galvanizing. Galvanizing, in the context of transmission structures, in general refers to the hot-dip process. This process involves the immersion of the section into a bath of molten zinc. Mechanical galvanizing has seen limited use. ASTM A123 (members) and A153 (hardware) have become widely accepted standards for galvanizing. These standards also reference applicable ASTM specifications for design practices and inspection.

Hot-dip galvanizing has been an economical means of corrosion protection for utility tubular and lattice structures. Galvanizing has been used extensively because of its dynamic protective nature consisting of a barrier coating and zinc's sacrificial action. The barrier coating provides a metallurgically bonded coating between the zinc and steel that completely seals the steel from the environment. Its sacrificial action further protects the steel when damage or minor flaws in the coating occur. Zinc is also a highly reactive metal. It exhibits a low corrosion rate only if a continuous passive film forms on the surface. A key requirement of corrosion control with zinc is that the surface needs to remain largely dry and in contact with air to develop and maintain this passive film. The production of high-quality galvanized steel poles depends on the metallurgical reaction between steel and molten zinc.

Considerable documentation exists on galvanizing. Appendix G attempts to provide a brief summary, some specifics applicable to transmission poles, and references for additional information.

Protection Provided. Hot-dip galvanizing has a long history of successful performance in numerous environmental conditions. Ultimately, the resistance of zinc to corrosion depends on the thickness of the zinc coating and the environment to which it is exposed.

Zinc coating behavior has been analyzed under various atmospheric conditions since 1926. These tests have been conducted throughout North America. Figure G-3 is a plot of these accumulated data and provides the thickness of galvanizing versus the expected life based on various environmental categories. The expected service life noted is based on the point at which 5% of the surface is showing red rust. This stage is unlikely to represent any actual steel weakening or jeopardy in the integrity of the structure caused by corrosion. Sufficient zinc remains in the substrate for implementation of remedial action.

Appearance. Initially a bright silver, the finish dulls with exposure. Appearance can be modified after galvanizing with various treatments (dulling or painting).

An uneven matte gray finish can result if the steel composition exceeds

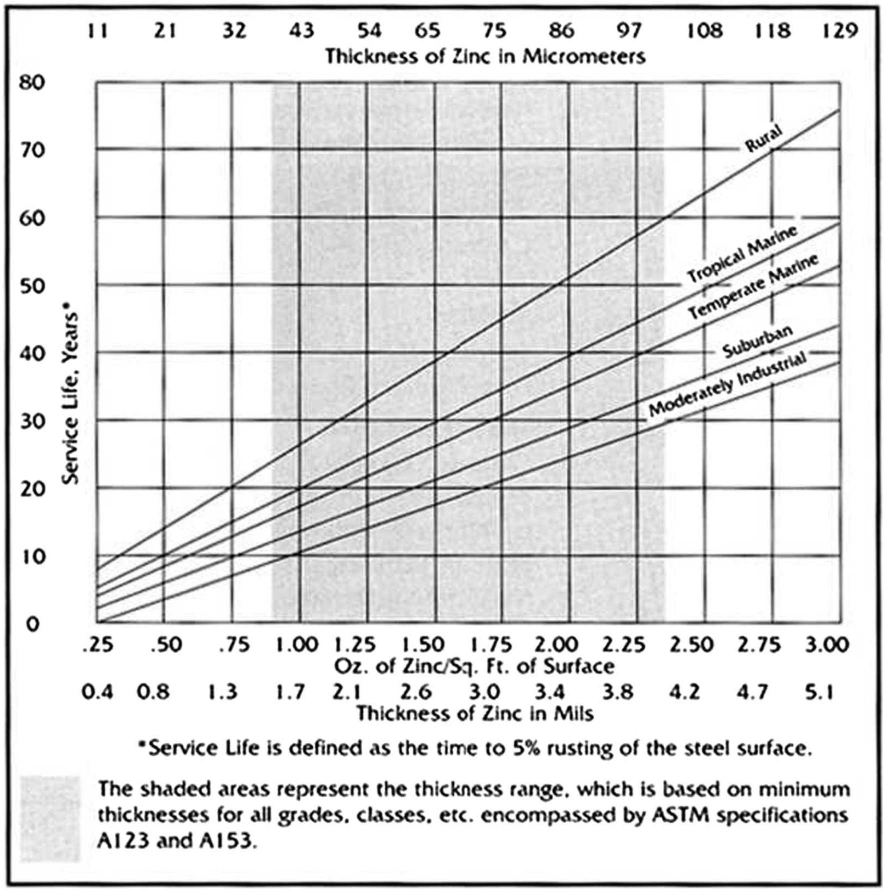


Figure G-3. Service life of hot-dipped galvanizing.
Source: Courtesy of the American Galvanizers Association.

- 0.25% carbon,
- 0.05% phosphorous,
- 1.35% manganese, and
- 0.06% silicon.

Silicon is often used in the deoxidation process when producing steel. This process can cause an uneven distribution of the silicon throughout the plate, resulting in an uneven appearance of the finish. This unevenness can be eliminated or reduced by specifying aluminum-killed steels.

The appearance can also be changed with addition of aluminum. Currently, the control of these elements is left up to individual galvanizers. No industry standard exists.

Special Design Considerations. ASTM A143 and A385 as well as AGA (2005) cover items including material selection, welding, effects on mechanical properties, drainage, and venting. These references should be consulted. Several specifics relating to transmission poles are listed as follows:

- *Steel selection*: See “Appearance” section.
- *Cold working*: AGA (2005) warns that “cold working is the strongest contributing factor to the embrittlement of galvanized steel” and lists the following eight precautions:
  - Select steels with carbon contents below 0.25%.
  - Select steels with low transition temperatures.
  - Specify aluminum-killed steels.
  - Use a bend radius of at least three times the section thickness; if bending is required less than $3t$, the material should be stress-relieved at 1,100 °F (~600 °C) for 1 h/in. of section thickness.
  - Avoid notches.
    - Drill holes (rather than punch) in material thicker than 0.75 in. (19 mm).
    - Cut edges greater than 0.625 in. (16 mm) thick that are subject to tensile loading. Thicknesses less than 0.625 in. (16 mm) may be sheared.
    - Steel should be hot-worked above 1,200 °F (~650 °C). Where cold working cannot be avoided, stress-relieve the affected part.
- *Welds*: Flux deposits (slag) should be removed before galvanizing. The normal pickling associated with the galvanizing process does not remove slag. Some welding processes do not produce slag. Welds that are inaccessible, such as the seam weld on enclosed shapes, should use one of these processes or involve joints where the slag deposits have a width of less than 3/16 in. (5 mm). The AGA’s recommendation for this 3/16 in. (5 mm) maximum dimension is based on the protection provided by the adjacent galvanizing should the slag drop off (AGA 2002b).
- *Toe cracking of weldments*: Toe cracks around T-joint welds are detected after galvanizing. The formation of these cracks is influenced by several factors in the fabrication process. Factors include the welding process (including preheat and interpass temperature regulation), material specification (including tensile strength), and product design [including the relative mass ratio between the base plate and 1 ft

(~0.3 m) of the pole section]. Requirements for postgalvanizing inspection should be considered.

- *Double dipping*: When an item is too large for total immersion in the molten zinc of the largest galvanizing kettle available, but more than half of the section will fit into the kettle, one end may be immersed and withdrawn, and then the other end may be galvanized. It is recommended that tubular structures be completely submerged in one dip in the galvanizing kettle. If double dipping is used, the tubular structure should be designed to permit access for inspection, cleaning, and possible repair of the overlap area. Trapped flux or improper fluxing can occur in the lapped portion. If not detected and repaired, corrosion may result, causing structural damage.
- *Hydrogen embrittlement*: Hydrogen embrittlement can occur when the hydrogen released during the pickling process is absorbed by the steel and becomes trapped in the grain boundaries. The heat involved in the galvanizing process normally causes the hydrogen to be expelled. If the ultimate tensile strength exceeds 150 ksi (1,034 MPa), such as is used in some fasteners, additional precautions should be used. This work could include blast cleaning rather than pickling.
- *Venting and drainage*: Excessive buildup of zinc, bare spots, and poor appearance can result from improper drainage design. Sections are immersed and withdrawn from the various kettles involved in the galvanizing process at an angle, and vent holes should be located at the highest and lowest points for best air venting and material drainage. It is recommended that openings at each end be at least 30% of the inside diameter area.
- *Backing bars*: To address possible entrapment of moisture from a buildup of interior condensate on nonsealed weathering steel pole sections, as well as possible reactivation of cleaning acids or fluxing salts deposited in cracks and crevices during the hot-dip galvanizing process, fabricators should consider sealing the top edge of backing bars used in certain welding details (circumferential butt joint welds, circumferential T-joint welds, and so forth) where accessible.

Application. The hot-dip galvanizing process consists of three basic steps: surface preparation, galvanizing, and inspection (AGA 2000).

Surface preparation is the most important step in the application of any coating. Surface preparation for galvanizing typically consists of three steps: caustic cleaning (or abrasive cleaning), acid pickling, and fluxing.

- *Caustic cleaning* uses a hot alkali solution to remove organic contaminants such as dirt, paint markings, grease, and oil from the steel surface. An *abrasive cleaning* can be used as an alternative or in conjunction with the chemical cleaning. Abrasive cleaning may be required for the removal of epoxies, vinyls, asphalt, or welding slag.
- *Pickling* uses a diluted solution of hot sulfuric acid or ambient temperature hydrochloric acid to remove scale and rust from the steel surface.
- *Fluxing* is the final surface preparation step; it removes oxides and prevents further oxides from forming on the steel surface before galvanizing. The method of application of flux depends on the galvanizer's process (wet or dry). The wet process involves a molten flux layer, which floats on top of the zinc bath. The fluxing occurs as the material is dipped into the galvanizing tank. The dry process requires that the steel be dipped in an aqueous ammonium chloride solution and then thoroughly dried before galvanizing.

Galvanizing is accomplished with the section fully immersed in a kettle of 98% pure molten zinc maintained at a temperature of approximately 840 °F (450 °C). The product is immersed until it reaches the temperature of the kettle. The zinc then reacts with the steel to form a zinc–iron intermetallic alloy at the steel surface. The metallurgical reaction continues after the removal of the product as long as the product is near the kettle temperature. A typical bath chemistry used in hot-dip galvanizing contains small amounts of iron, aluminum, and lead in addition to the zinc.

Because galvanizing is a total immersion process, all surfaces are coated. Galvanizing provides both inside and outside protection for tubular products. Coating thickness can be influenced by a number of factors, including the composition and physical condition of the steel, the pickling process, and the immersion time and removal rate. ASTM A123 requires an average zinc weight of 2 oz/ft^2 (3.3 mil) [600 g/m^2 (85 μm)] for metal thicknesses less than 0.25 in. (6 mm). A 2.3 oz/ft^2 (3.9 mil) [705 g/m^2 (100 μm)] average is required for 0.25 in. (6mm) and thicker parts. Depending on the procedures used, maximum thicknesses of up to 3 oz/ft^2 (5.1 mil) [920 g/m^2 (130 μm)] can be obtained at an additional cost. This additional zinc coating provides additional life.

Any zinc surface in contact with the surrounding air quickly forms a film of zinc oxide (this should not be confused with wet storage stain, which should be removed). When the zinc oxide has access to freely moving air in normal atmospheric exposure, it reacts with rainfall or dew to form a porous, gelatinous zinc hydroxide corrosion product. During drying, this product reacts with carbon dioxide in the atmosphere and becomes a thin, compact, and tightly adherent whitish-gray film layer. The long life normally associated with galvanized coatings in atmospheric service depends entirely on the protection of this layer. Chromate dips or rinses have been used as a safeguard against white rust. Availability of this process has been virtually eliminated, however, because of its hazardous status.

Inspection procedures and acceptance or rejection of galvanized steel material should conform to ASTM A123 or A153, as applicable. Inspections and tests can be performed to determine visual examination of samples and finished products, as well as the weight of zinc coating per square foot of metal surface or thickness and uniformity; this test is normally performed using the magnetic thickness measurement method. Note that the AGA (2001) provides useful guidance for evaluation of coating.

Repair. Wet storage stain (white rust) should be prevented and removed when discovered. Wet storage stain is a voluminous white or gray deposit. It is formed when closely packed, newly galvanized articles are stored or shipped under damp and poorly ventilated conditions (e.g., galvanized sheets, plates, angles, bars, and pipes). AGA (1997), Aichinger and Higgins (2006), and ASTM A780 (2009) describe methods of prevention, removal, and repair.

Acceptable methods for repairing damaged galvanized surfaces are described subsequently.

Cold Galvanizing Compound. Surfaces to be reconditioned with zinc-rich paint should be clean, dry, and free of oil, grease, and corrosion products. Areas to be repaired should be power disc sanded to bright metal. To ensure that a smooth reconditioned coating can be effected, surface preparation should extend into the undamaged, galvanized coating.

Touch-up paint should be an organic cold galvanizing compound with a minimum of 92% zinc dust in the dry film.

The renovated area should have a zinc coating thickness of 150% of that specified in ASTM A123 for the thickness grade of the appropriate material category. A finish coat of aluminum paint may be applied to provide a color blend with the surrounding galvanizing. Coating thickness should be verified by measurements with a magnetic or electromagnetic gauge.

Zinc-Based Solder. Surfaces to be reconditioned with zinc-based solder should be clean, dry, and free of oil, grease, and corrosion products. Areas to be repaired should be wire brushed.

According to ASTM A780, the surface to be reconditioned should be wire brushed, lightly ground, or mild blast cleaned. All weld flux and spatter should be removed by mechanical methods if wire brushing or light blasting is inadequate. The cleaned area should be preheated to 600 °F (315 °C) and at the same time wire brushed. Care should be exercised not to burn the surrounding galvanized coating.

Because solders are molten when applied, resultant coatings are inherently thin. The renovated area should have a zinc coating thickness at least as much as that specified in ASTM A123 for the thickness grade for the appropriate material category, but not more than 4 mil (100 μm). Coating thickness should be verified by measurements with a magnetic or electromagnetic gauge.

Experience has indicated that cracking may occur when zinc-based solders are used that contain tin (Aichinger and Higgins 2006).

Weathering Steel

General. Weathering steel develops a tight oxide coating that protects against continuing corrosion of the substrate. This type of steel develops a finish that is relatively maintenance free.

The use of bare, uncoated weathering steels should be evaluated for suitability in certain environments. Weathering steels should not be used in bare condition in atmospheres with high concentrations of corrosive chemicals, industrial fumes, or severe continuous salt fog or spray environments. Sulfur oxide concentrations are the most critical pollutant among the various chemical components affecting steel corrosion. Weathering steel has been used in marine environments; however, the suitability depends on a thorough evaluation of the surrounding conditions. Such evaluations have been made during the last 30 years in a number of industrial plant and marine atmospheres.

Material Properties. Weathering steels are defined as those materials meeting ASTM G101 with a corrosion index exceeding 6.0. Common designations for steel plate are ASTM A242, ASTM A588, and ASTM A871. ASTM covers the requirements for the steels with yield strengths of 50,000 psi (345 mPa) and 65,000 psi (448 mPa).

Oxide Formation. The type of oxide that forms on a steel is determined by the steel's alloy content, the nature of the atmosphere, and the frequency with which the surface is wetted by dew and rainfall and dried by the wind and sun.

The rapidity with which the steel develops its protective oxide coating and characteristic color depends primarily on the nature of the environment. In general, the weathering process is more rapid and the color darker in an industrial atmosphere, whereas the oxide formation is usually slower and the color lighter in rural atmospheres. The oxide coating usually forms over a period of 18 months to 3 years. The frequency of condensation and the time of wetness are factors that affect the period required for the formulation of the oxide.

The formation of the protective oxide is usually associated with the loss in metal thickness of about 2 mil (0.05 mm). Approximately half of the oxide formed in the early stages is retained, and the balance is lost through the eroding action of wind and rain. Observations made of structures and specimens indicate that drainage of soluble corrosion products, amounting to approximately 0.1% of the initial weight loss, occurs in the early stages of exposure. The precipitate of the soluble corrosion products causes staining on some materials. This staining continues at a reduced rate for an indefinite period. The line designer should consider this possibility and take steps, if necessary, to contain or divert the drainage products.

The texture of the oxide depends on the action of the wind and rain and the drying effect of sunlight. On boldly exposed surfaces, a tightly adherent protective oxide develops. On sheltered but exposed exterior surfaces, a somewhat granular, loosely adherent, but protective oxide forms.

Shot blast cleaning of weathering steel structures is not necessary; however, all grease, oil, and shop markings should be removed. Blast cleaning, however, provides a cleaner and more uniform weathering appearance in a shorter period of time.

Design Considerations. For weathering steels to develop their oxide coating and provide proper protection, they should be exposed to a proper wetting and drying cycle.

Surfaces that are wet for prolonged periods of time corrode at an unacceptably rapid rate. Therefore, the detailing of members and assemblies should avoid pockets, crevices, faying surfaces, or locations that can collect and retain water, damp debris, and moisture. One phenomenon related to weathering steels is known as *pack-out*. Pack-out has appeared in tightly bolted joints where moisture is present on two interior faying surfaces of the joint for an extended period. A lightweight, bulky substance (pack-out) develops over time, generating sufficient forces to actually bend or deform steel. This condition has been primarily isolated to transmission lattice towers because of moisture entrapment in the bolted joints. This entrapment has not been found to be a problem on tubular steel structures.

In general, weathering steel structures should either be sealed or well ventilated to ensure the proper corrosion protection for the pole's interior surfaces. When a ventilated structure is specified, weep holes should be of sufficient size to allow water to flow freely. The holes should also be positioned to allow for periodic inspection and cleaning as necessary to remove debris.

Special bolting patterns may be considered for flange joints to ensure proper corrosion protection. Bolted joints should be stiff and tight. To provide this stiffness and tightness, the following guidelines are suggested (Brockenbrough 1983):

- The pitch on a line of fasteners adjacent to a free edge of plates or shapes in contact with one another should not exceed 14 times the thickness of the thinnest part, nor exceed 7 in. (180 mm).
- The distance from the center of any bolt to the nearest free edge of plates or shapes in contact with one another should not exceed 8 times the thickness of the thinnest part, nor 5 in. (125 mm). (Edges of elements sandwiched between splice plates need not meet this requirement.)
- Preferable fasteners are ASTM A325 Type 3 bolts installed in conformance with Research Council on Structural Connections (RCSC) (1985). A dielectric coating may be considered for the faying surfaces of a flange joint.

Bare weathering steel should not be buried without a method of protection against corrosion. The line designer should select the method of protection for the particular application. Conventional methods of protection, such as concrete encasement or a high-quality coating such as coal tar epoxy and/or polyurethane, are usually acceptable. This protection should extend well above the groundline to ensure that the bare weathering steel does not come in contact with soil or debris that can retain moisture for extended periods of time.

Fabrication. On receipt and during fabrication, the fabricator should accurately identify all weathering steels to ensure that proper materials are used for the order. Welding materials and welding procedures should be compatible with the parent material to ensure proper welding characteristics.

Compatibility with Other Materials. In general, dissimilar metals such as stainless steel, anodized aluminum, copper, bronze, and brass can be used adjacent to weathering steel, provided coupled areas do not act as crevices that might collect and hold water and debris. Galvanized steel line hardware has commonly been used on weathering steel transmission structures for many years. There are no known problems resulting from this particular interfacing of dissimilar metals.

Experience has shown that certain types of backfilling foams containing fire retardants may become corrosive when wet. Weathering steel surfaces exposed to foam may need to be protected by paint systems that are compatible with the foam. The foam supplier's recommendations for paint systems should be followed.

Lumber that is treated with salts to retard decay and fire should not be used in contact with weathering steel unless the lumber and the contacting steel surfaces, including fasteners, are painted. Otherwise, the combination of water and salts used in lumber treatment chemically attacks and corrodes the steel. Treated lumber suppliers' recommendations for paint systems for such situations should be followed.

Coatings. Weathering steel may be painted as readily as regular carbon steel. Weathering steels may also be galvanized; however, the appearance may not be uniform because of the higher silicon content.

Metalizing

General. The structure may be protected against corrosion by thermal spraying a coating over the base metal (substrate). Thermal spraying includes flame spraying, electric arc spraying, and plasma spraying.

Material. The material used for spraying should be made especially for that purpose. Zinc used for spraying should have a minimum purity of 99.9%. Aluminum used for spraying should have a minimum purity of 99.0%. Single-use abrasives used in the preparation of steel surfaces should consist of sand, special crushed slag, flint, or garnet abrasives. These abrasives should not be reclaimed and reused.

The abrasive should be hard, sharp, and angular. Round silica sand or similar materials should not be used. Multiple-use abrasives used for the preparation of steel surfaces should consist of angular chilled iron grit. These abrasives may be reclaimed. Round iron shot or rounded grit should not be used. Abrasives to be reused should be checked to see that at least 80% conforms to the original requirements. Abrasives and their sizes may vary, depending on the special requirements of the work to be done. All abrasives should be clean, dry, and free from oil or other contamination.

Equipment. Any commercial type of *dry* blasting equipment may be used to clean and roughen the surface. Compressed air equipment capable of producing 25% greater volume of air than required at any one time should be used. The compressor should be equipped with an efficient oil and water separator to prevent contamination of the surface to be metalized.

The spray equipment may be one of the following types:

- Wire-gas metalizing equipment
- Powder-gas metalizing equipment
- Electric arc metalizing equipment
- Plasma spraying equipment

The equipment should be operated according to the manufacturer's written instructions and recommendations.

Surface Preparation. The surface should be thoroughly cleaned by blasting to SSPC Standard SP 5 and roughened for proper bond.

Coating Application. The metalizing application should be accomplished in accordance with the manufacturer's recommendations. Further instructions may be found in the American Welding Society Standard C2.2 (AWS 1967).

At least one layer of coating should be applied within 4 h of blasting, and the blasted surface should be completely coated to the specified thickness within 8 h of blasting. It is preferable to apply the full thickness within 2 h after blasting, if possible. Multiple passes may be used to apply the coating, and in no case should fewer than two passes be made over the surface being coated. The sprayed metal should overlap on each pass of the gun to ensure uniform coverage.

Coating Inspection. The surface of the structural steel prepared for spraying should be inspected visually. The metalized coating should be inspected for thickness by magnetic thickness gauge. The inspection should follow as soon as possible after completion of the spraying. Any metalized surface that exhibits visible moisture, rust, scale, or other contamination should be reblasted before spraying.

An adherence test may be made by cutting through the coating with a knife. Bond will be considered unsatisfactory if any part of the coating lifts away without cutting the zinc or aluminum metal. Defective areas should be sand blasted clean before respraying, except where the rejection is caused by insufficient thickness. Compliance with coating thickness requirements should be checked with a magnetic thickness gauge.

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APPENDIX H
ARM-TO-SHAFT CONNECTION ANALYSIS CONSIDERATIONS

A common method for supporting a conductor or shield wire involves a *davit* arm, which is a cantilevered element extending out from the pole shaft that provides both the necessary electrical clearance from the pole as well as the minimum ground clearance for overhead power lines.

A variety of details have been used to connect a davit arm to a pole shaft. Two of the most common connections are through-plates (Figure H-1) and through bolts (Figure H-2). Each of these connection types uses a base mount on the arm that provides for attachment to the pole and a transfer of the loads to the pole shaft.

Most fabricators have used empirical methods, including full-scale testing, to develop analysis procedures for their own arm connections. These proprietary procedures account for their own specific detailing and manufacturing practices. No standard design methods have been developed for general use within the industry.

Because there are no universal procedures available to the industry, this discussion is provided to establish the primary issues to be considered when developing an appropriate methodology for analyzing arm connections. These are basic guidelines only and should not be construed as being a complete methodology for the design of arm connections.

GENERAL LOAD, DIMENSION, AND MATERIAL SPECIFICATIONS

The primary issues to be considered when developing an appropriate methodology for analyzing arm connections include the following:

- Pole and arm geometries,
- Fasteners,
- Material grades, and
- Applied loads.

Arm channels on through-plates are typically used to support heavier loads, which include transverse, vertical, and longitudinal loads in any kind of application: tangent, angle, or dead end.

- For a given arm design, the channel legs have a minimum inside-to-inside spacing dimension that needs to correspond with an outside-to-outside through-plate spacing such that a bolt group can be designed to resist the loads in shear. It is important to set fabrication and assembly tolerances to ensure that the bolts are not subject to bending and that the recommended bolt torque is not exceeded.

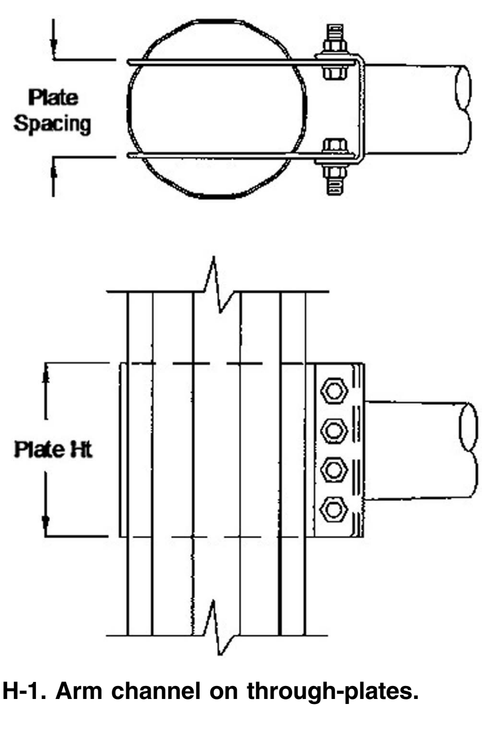

Figure H-1. Arm channel on through-plates.

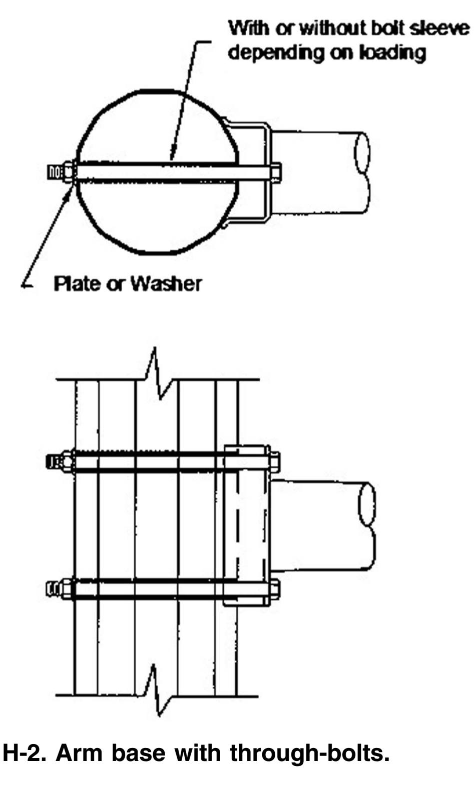

Figure H-2. Arm base with through-bolts.

- There is a practical maximum through-plate spacing relative to pole diameter at the elevation of the arm attachment.
- Bolt quantity, spacing, diameter, and grade are selected to satisfy shear criteria, including whether or not the bolts are in the shear plane.
- Through-plate height, thickness, grade, and edge distances are selected to satisfy tension, bearing, and block shear criteria.
- Weld size and detail of the through-plate connection to both the front and back pole shaft walls are determined to resist the applied shear loads.

Arm bases with through-bolts are typically used for lighter loads, such as transverse and vertical suspension loads resulting from tangent applications.

- Arm bases that are shaped to accommodate a range of pole diameters are typically referred to as *gain* bases and are commonly used for line post insulators. The concept is for the base to cradle the pole shaft while being held tight against it by bolts passing through the centerline of the pole. The intended contact between the flared leg and the pole surface is along the broad surface provided by the base geometry. Occasionally, a through-pipe is used to reinforce the pole shaft and enhance the bearing performance.
- Depending on the diameter of the pole and the width of the base, the mating of the surfaces varies. This mating may result in the edges of the base contacting the surfaces of the pole or the pole tending to spread apart the base. Because of the variations in mating surfaces, it is important to follow installation instructions and not to overtighten the bolts to avoid deforming the arm base and/or the pole shaft.
- The through-bolt length is determined by the arm base geometry and the pole diameter. Through-bolt diameter and grade are determined by the vertical hole spacing in the arm base and the applied loads. Bearing and bending should be considered; however, the primary load is typically tension.
- Washers are sized to effectively distribute the load over the back wall of the pole.

In addition to the aforementioned connection types, a number of other types have been used within the industry, and within each connection type, there are various styles and details. Examples of such connections include surface-stiffened channels, box, doubler or wraparound, band on, chain mount, and cross-bolt connections.

APPENDIX I
FLANGE AND BASE PLATE CONNECTIONS

INTRODUCTION

Flange and base plate welded connections are critical joints for transmission structures. The most common application of these connections is base plates supported by anchor bolts and splices between tubular sections. The design and fabrication of these connections have been based on years of field experience combined with experimental and analytical investigations without the benefit of a comprehensive uniform standard.

The purpose of this appendix is to bring awareness to the owner, structure designer, and fabricator of some of the considerations that may be appropriate for a specific project. The applicability of these considerations to a specific project should be determined through collaboration among the owner, the structure designer, and the fabricator.

DESCRIPTION

In general, flange and base plate connections consist of comparatively thinner-wall tubular shafts connected to thicker transverse plates often with limited or no access to the interior of the tubular shaft.

T-joints are the most common type of joint used to connect transverse plates to tubular sections. A T-joint has the base of the shaft butted up against a transverse plate. The shaft wall tension and compression stresses are transferred by applying normal (through-thickness) stresses to the transverse plate. Complete penetration T-joints are required according to Section 6.3.5 for flange and base plate connections to transmission pole shafts. A reinforcing fillet weld is used around the perimeter of the shaft to reduce through-thickness stresses according to Sections 6.3.3.1 and 6.3.5.

CONSIDERATIONS

Material. When relatively thick plates are connected to thin-wall tubular sections, other supplementary material properties may be desired in addition to the supplementary Charpy V-notch (CVN) material requirement specified in Section 5.2.1.3. Many of the steel material specifications outlined in Section 5.2.1 that are commonly used for base and flange plate joints do not specify a maximum tensile strength; therefore, limiting the maximum tensile strength may be desirable to improve weldability.

The same supplementary requirements may apply to forged rings that are used as base or flange plates. Refer to Appendix J for additional information regarding forged rings.

Welding. AWS D1.1 classifies connections into three general categories: nontubular, tubular, and cyclically loaded. Although a tubular shaft is involved, the tubular joint provisions of AWS D1.1 were adopted through experience from the welding practices common to offshore platforms, which have unique tubular connections that differ from the flange and base plate connections used for transmission pole structures. In addition, transmission pole structures are designed for static loading conditions.

For the aforementioned reasons, the provisions of AWS D1.1 for nontubular joints are recommended for flange and base plate connections for transmission pole structures. The purpose is to provide some optional alternative methods for determining welding conditions (principally preheat) to avoid cold cracking.

Welding Procedure Qualification. The welding procedures for T-joints should be qualified in accordance with AWS. As specified in Section 7.2.3, qualification tests must be performed to demonstrate that a welding process will result in the same absorbed energy for the weld metal and heat-affected zone (HAZ) as the minimum absorbed energy specified for the base metal. Welding parameters such as pass size, electrode type and diameter, travel speed, among others, may have a significant effect on the HAZ and the deposited weld metal.

Backing Bars. A continuous backing bar should be considered and located with backing bar splices away from corners of polygonal shapes and away from longitudinal shaft seam welds. The backing bars should be welded to the flange or base plate. The welding detail of the backing bar should not interfere with subsequent ultrasonic (UT) inspection of the base weld.

UT Inspection. In general, the provisions of AWS D1.1 Clause 6 pertaining to UT inspection have been found to be insufficient for application to flange and base plate connections for transmission structures. The provisions of Clause 6 are also limited to a minimum plate thickness of 5/16 in. (8 mm), which is an issue for many transmission structures fabricated from thinner material. Annex Q of AWS D1.1 provides guidelines for UT examination by alternative techniques. A unique qualified UT procedure should be developed under the supervision of an SNT-TC-1A, (ASNT 2016) Level III inspector familiar with the joint configuration including the type and size of backing bars when used. The transducer size, beam angle, and frequency should be documented in the procedure. Calibration blocks and mock-up joints should be used to establish the qualified procedure and be used to train production UT inspectors.

T-joints subject transverse plates to through-thickness stresses, which may be a concern if laminar defects exist in the plate. Additional through-thickness stresses may occur due to weld shrinkage stresses. As the flange or base plate thickness increases, the probability increases for laminar defects being present because of plate manufacturing processes. The need for UT inspection for laminar defects should be considered after the weld-out of relatively thick flange or base plate T-joint connections.

Toe Cracks. Postgalvanizing toe crack inspection is required according to Section 10.3.5. The probability of toe cracks occurring increases as the ratio of flange or base plate thickness to shaft wall thickness increases. For some joints, zinc coatings other than hot-dip galvanizing of the flange or base plate may be an option to mitigate the risk of toe cracks caused by galvanizing. Refer to Appendix G for additional considerations regarding toe cracking of weldments.

Undetected toe cracks may grow under service conditions depending on the size and location of the crack and the magnitude and type of loading. UT inspection has proven to be the most effective method for detecting toe cracks as a first-line inspection method. Magnetic particle (MT) inspection has limited value in finding toe cracks on a galvanized finish because of the inherent surface roughness associated with galvanizing. When UT inspection has provided an indication of a toe crack, MT inspection can be effective as a validation of the UT indication. Locations with indications should have the galvanized surface ground off prior to MT inspection.

Toe cracks can be successfully repaired using qualified procedures. Repairs to galvanized surfaces can also be performed using qualified procedures. Caution should be used when using zinc-based solders that contain high levels of lead and tin to repair a galvanized surface because these elements may cause additional cracking after application.

Base and Flange Plate Thickness. Thicker plates increase stiffness and minimize stress concentrations at the shaft wall. Secondary bending stresses in anchor bolts are also reduced. The use of thicker plates, however, may cause welding and galvanizing issues due to excessive restraint and higher thermal stresses during galvanizing, which may result in weld cracks.

Center Openings. The size specified for a center opening in a flange or base plate should be determined by considering several factors. Center openings reduce the mass of base and flange plates and may reduce the immersion time required in a galvanizing kettle. Larger center openings may also reduce the thermal stresses that occur during the hot-dip galvanizing process and may facilitate the removal of ash inside a tubular section upon removal from the kettle. On the other hand, smaller center openings increase the strength and stiffness of a flange or base plate. Some methods for determining the thickness of flange and base plates depend on beam action and assume that the stiffness of the plate continues beyond the shaft wall toward the interior of the tubular section. The reduction in stiffness due to the presence of a center opening of a flange or base plate in a T-joint should be considered when determining the thickness of a flange or base plate. Flange and base plate design methods should be validated by experimental or analytical investigations.

Bolt Patterns. Unsymmetrical bolt patterns may result in higher stress concentrations at the shaft wall compared to symmetrical patterns. The same effect occurs with bolt patterns that have large center-to-center spacings or that have large spacings between the pole shaft and the flange bolts or anchor bolts. These effects may be undesirable depending on the magnitude and type of loading, in which case the use of unsymmetrical bolt patterns should be based on comprehensive testing or analytical models for the type of loading anticipated.

REFERENCE

ASNT (American Society for Nondestructive Testing). 2016. *Personnel qualification and certification in nondestructive testing*. Recommended Practice No. SNT-TC-1A.

APPENDIX J
FORGED/ROLLED RING BASE PLATES AND FLANGE PLATES

BACKGROUND

Tubular steel transmission pole structures can be supported by a variety of foundation types. One of the most common foundation support details is a conventional concrete pier foundation cast with anchor bolts on which a base-plated tubular steel pole can be anchored. Because more tubular steel poles are being used at higher voltages, these structures have become increasingly larger in size. As the structure size increases, both the square size and thickness of the resulting base plate also increase.

Material typically used for base plate has been ASTM A572, Grade 60 or 50 for galvanized finishes or ASTM A871, Grade 60 or 50 for weathering (and galvanized) finishes. With the extra specification demands the utility industry has placed on plate material beyond the standard ASTM requirements [longitudinal Charpy V-notch (CVNL) testing requirements, among others], steel, in the plate thicknesses and widths needed, has become more difficult to purchase. This has become especially true for base plate thicknesses of ≥4 in. (10.2 cm). Forged and rolled rings are a potential solution to these plate material availability issues.

MANUFACTURING PROCESS

Although their design criteria and connection details are different, large wind towers have been using a forged/rolled ring for flanges and base plates for some time owing to the extra-large dimensions this process allows for such parts. ASTM A350 LF6 and ASTM A1090 are the respective ASTM specification equivalents to the currently used ASTM A572 and ASTM A871. Essentially, a forged/rolled ring is a billet (small cylinder) of steel that is heated and forged into a *doughnut* shape and then rolled into a ring of a specific required diameter and thickness. Postforging/rolling processes may include drilling any required holes (i.e., for anchor bolts) and/or machining the top, bottom, and side surfaces. The completed forged/rolled ring would then be welded to the pole shaft.

Although manufacturing processes may vary by material supplier, the following outline is a basic explanation of one process:

1. The starting stock is cut to size by weight from a steel billet and is heated. It is then upset (pressed) to achieve structural integrity and directional grain flow (Figure J-1).
2. The work piece is then punched and pierced to achieve the starting doughnut shape needed for the ring rolling process (Figure J-2). The doughnut shape is called the *preform*.
3. The completed preform is now ready for placement on the ring mill for rolling (Figure J-3). The preform is returned to the oven for reheating prior to ring rolling.
4. The ring rolling process begins with the idler roll applying pressure to the preform against the drive roll to begin increasing the inside diameter (Figure J-4).
5. The ring diameters (inside and outside) are increased as continuous pressure from the top and bottom rolls reduces the part height/thickness (Figure J-5).
6. The process continues until the desired size (thickness and outside and inside diameters) is achieved (Figure J-6).

During the entire process, the work piece is kept above the recrystallization temperature to improve the formability. Temperatures during the forming and rolling processes are usually maintained between 1,600 °F (870 °C) and 2,300 °F (1260 °C), which is essentially the same temperature used during the conventional hot-rolling processes for steel plate product. The lower limit for hot-rolling the ring is primarily a function of being able to effectively manipulate the material without damaging the work piece.

After the rolled ring has air cooled, it is then heat treated to achieve the required mechanical and toughness properties. Heat treatment consists of heating the ring to a temperature at which the material is completely austenitic, then liquid quenching (normally in water), and finally tempering the ring. Quenching the steel provides the required strength characteristics, and tempering softens the steel and provides the necessary toughness. This heat-treatment approach is common throughout the steel industry and typically provides consistent results for the desired mechanical and toughness properties.

ADVANTAGES AND DISADVANTAGES

Advantages of rolled ring base plates are as follows:

- No thermal cutting is required.
- Thick, very large diameter base plates/flange plates can be produced.
- There are usually fewer nonmetallic inclusions.
- The resulting forged/rolled ring has a consistent, circular grain flow that allows for more consistent mechanical properties around the base plate.

The differences are illustrated in Figures J-7 and J-8.

Disadvantages of rolled ring base plates are as follows:

- Forged/rolled rings may be more expensive than parts fabricated from hot-rolled plate, although that cost disadvantage is sometimes offset by market conditions and, particularly with larger ring sizes, by the cost savings resulting from a process with virtually no wasted material.
- Planning and coordination are required to determine how to obtain sufficient material from the produced forged/rolled ring to be used in testing for conformance to requirements.
- In general, the industry is unfamiliar with the material and manufacturing process.

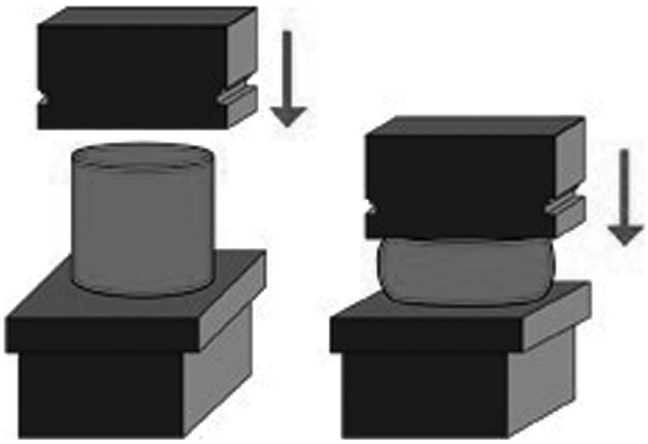

Figure J-1. Manufacturing process: Step 1.

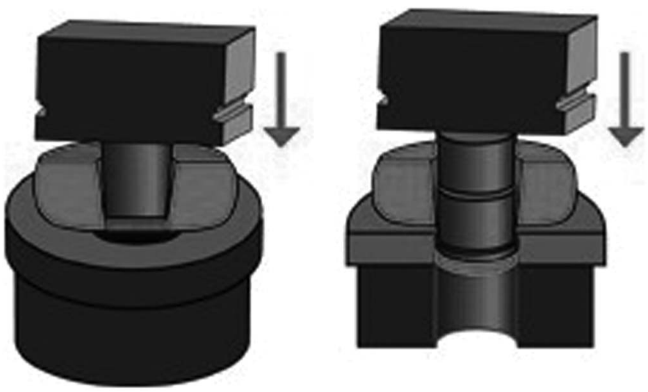

Figure J-2. Manufacturing process: Step 2.

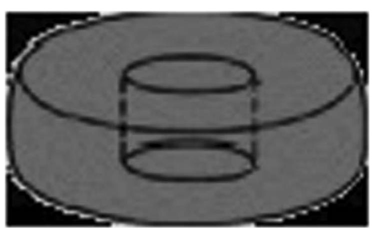

Figure J-3. Manufacturing process: Step 3.

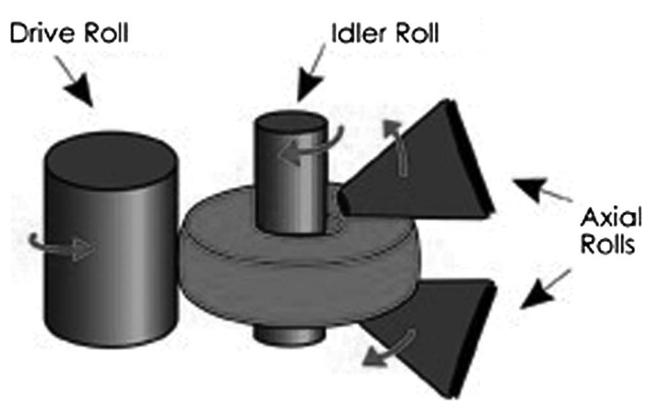

Figure J-4. Manufacturing process: Step 4.

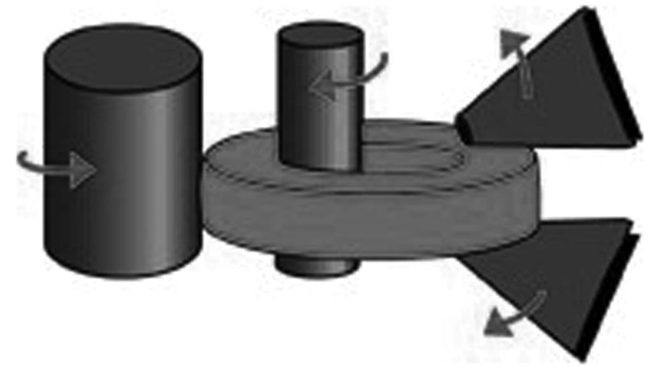

Figure J-5. Manufacturing process: Step 5.

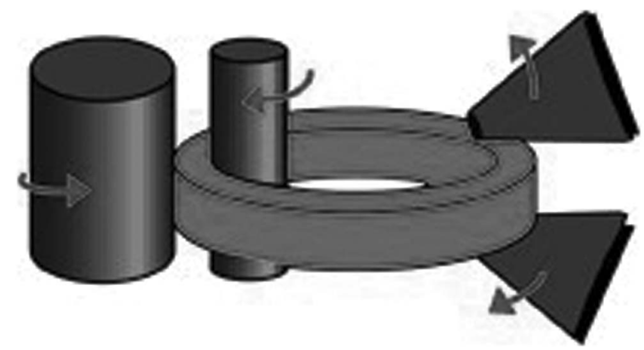

Figure J-6. Manufacturing process: Step 6.

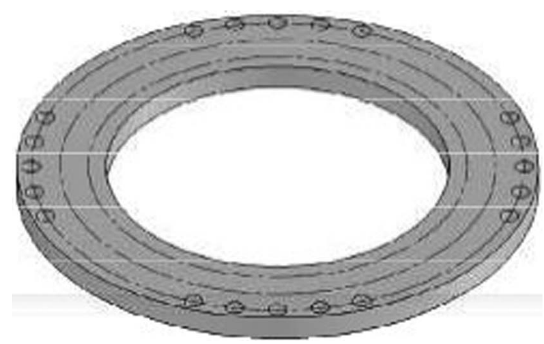

Figure J-7. Rolled ring base plate showing grain flow in a circumferential direction.

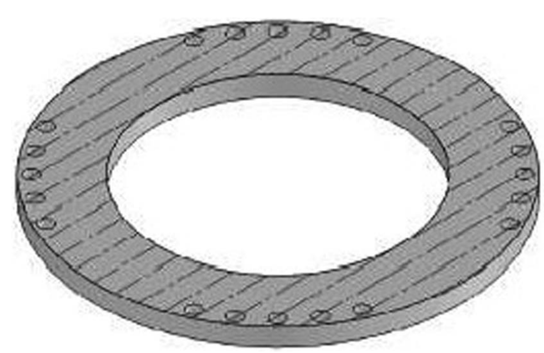

Figure J-8. Conventional base plate cut from hot-rolled plate showing grain flow in a single direction.

- The large center opening may affect existing base plate/flange plate design philosophies/practices as well as the methodology/results in Appendix F. It is recommended that details and design practices be substantiated by experiential or analytical investigations.
- It is necessary to review and change welding procedure specifications (WPSs) and procedure qualification records (PQRs).

SPECIFYING FORGED/ROLLED RING BASE PLATES

Some considerations for specifying the use of forged/rolled ring base plates and flanges are as follows:

- Probably the most critical requirement in specifying forged/rolled ring base plates is to ensure that they meet the same mechanical and chemical properties as the standard ASTM hot-rolled base plates that are commonly used in the electric utility industry (A633, A572, A588, A871). ASTM A350 LF6 and ASTM A1090 are the specifications corresponding to the currently used ASTM A572 and ASTM A871, respectively.
- In addition, all base plates should meet the industry standard toughness requirements based on Charpy V-notch testing [15 ft-lbs at −20 °F (20J at −29 °C)]. The same considerations for requiring heat lot or plate testing should be considered as with rolled plate material.
- Weld procedures must be specifically qualified based on the rolled ring material.
- The rolled rings should be inspected for laminar material defects in accordance with ASTM A388 and A435.
- As with hot-rolled plate material, the typical mechanical preparation for fit-up and welding (such as a clean, smooth surface) is also required with forged/rolled ring base plates.

SUMMARY

Forged/rolled ring base plates are becoming an important and necessary material to meet the ever-increasing demands of the steel transmission pole industry. The forged/rolled ring forming and heat-treating processes are similar to those used in the manufacturing of hot-rolled plates. When ordered to the appropriate ASTM specification that matches the chemical and mechanical properties for materials currently being used, forged/rolled rings should have similar mechanical and toughness properties and similar weldability and serviceability as traditional hot-rolled base plate material.

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Figures are indicated by *f;* tables are indicated by *t.*