ASCE STANDARD

ASCE/SEI 48–11

Design of Steel Transmission Pole Structures

This document uses both the International System of Units (SI) and customary units

ASCE

SEI ASCE STRUCTURAL ENGINEERING INSTITUTE

American Society of Civil Engineers

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STANDARDS

In 2006, the Board of Direction approved the revision to the ASCE Rules for Standards Committees to govern the writing and maintenance of standards developed by the Society. All such standards are developed by a consensus standards process managed by the Society's Codes and Standards Committee (CSC). The consensus process includes balloting by a balanced standards committee made up of Society members and nonmembers, balloting by the membership of the Society as a whole, and balloting by the public. All standards are updated or reaffirmed by the same process at intervals not exceeding five years.

The following standards have been issued:

ANSI/ASCE 1-82 N-725 Guideline for Design and Analysis of Nuclear Safety Related Earth Structures
ASCE/EWRI 2-06 Measurement of Oxygen Transfer in Clean Water
ANSI/ASCE 3-91 Standard for the Structural Design of Composite Slabs and ANSI/ASCE 9-91 Standard Practice for the Construction and Inspection of Composite Slabs
ASCE 4-98 Seismic Analysis of Safety-Related Nuclear Structures
Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/TMS 402-02) and Specifications for Masonry Structures (ACI 530.1-02/ASCE 6-02/TMS 602-02)
ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures
SEI/ASCE 8-02 Standard Specification for the Design of Cold-Formed Stainless Steel Structural Members
ANSI/ASCE 9-91 listed with ASCE 3-91
ASCE 10-97 Design of Latticed Steel Transmission Structures
SEI/ASCE 11-99 Guideline for Structural Condition Assessment of Existing Buildings
ASCE/EWRI 12-05 Guideline for the Design of Urban Subsurface Drainage
ASCE/EWRI 13-05 Standard Guidelines for Installation of Urban Subsurface Drainage
ASCE/EWRI 14-05 Standard Guidelines for Operation and Maintenance of Urban Subsurface Drainage
ASCE 15-98 Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD)
ASCE 16-95 Standard for Load Resistance Factor Design (LRFD) of Engineered Wood Construction
ASCE 17-96 Air-Supported Structures
ASCE 18-96 Standard Guidelines for In-Process Oxygen Transfer Testing
ASCE/SEI 19-10 Structural Applications of Steel Cables for Buildings
ASCE 20-96 Standard Guidelines for the Design and Installation of Pile Foundations
ANSI/ASCE/T&DI 21-05 Automated People Mover Standards—Part 1
ANSI/ASCE/T&DI 21.2-08 Automated People Mover Standards—Part 2
ANSI/ASCE/T&DI 21.3-08 Automated People Mover Standards—Part 3
ANSI/ASCE/T&DI 21.4-08 Automated People Mover Standards—Part 4
SEI/ASCE 23-97 Specification for Structural Steel Beams with Web Openings
ASCE/SEI 24-05 Flood Resistant Design and Construction
ASCE/SEI 25-06 Earthquake-Actuated Automatic Gas Shutoff Devices
ASCE 26-97 Standard Practice for Design of Buried Precast Concrete Box Sections
ASCE 27-00 Standard Practice for Direct Design of Precast Concrete Pipe for Jacking in Trenchless Construction
ASCE 28-00 Standard Practice for Direct Design of Precast Concrete Box Sections for Jacking in Trenchless Construction
ASCE/SEI/SFPE 29-05 Standard Calculation Methods for Structural Fire Protection
SEI/ASCE 30-00 Guideline for Condition Assessment of the Building Envelope
SEI/ASCE 31-03 Seismic Evaluation of Existing Buildings
SEI/ASCE 32-01 Design and Construction of Frost-Protected Shallow Foundations
EWRI/ASCE 33-09 Comprehensive Transboundary International Water Quality Management Agreement
EWRI/ASCE 34-01 Standard Guidelines for Artificial Recharge of Ground Water
EWRI/ASCE 35-01 Guidelines for Quality Assurance of Installed Fine-Pore Aeration Equipment
CI/ASCE 36-01 Standard Construction Guidelines for Microtunneling
SEI/ASCE 37-02 Design Loads on Structures during Construction
CI/ASCE 38-02 Standard Guideline for the Collection and Depiction of Existing Subsurface Utility Data
EWRI/ASCE 39-03 Standard Practice for the Design and Operation of Hail Suppression Projects
ASCE/EWRI 40-03 Regulated Riparian Model Water Code
ASCE/SEI 41-06 Seismic Rehabilitation of Existing Buildings
ASCE/EWRI 42-04 Standard Practice for the Design and Operation of Precipitation Enhancement Projects
ASCE/SEI 43-05 Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities
ASCE/EWRI 44-05 Standard Practice for the Design and Operation of Supercooled Fog Dispersal Projects
ASCE/EWRI 45-05 Standard Guidelines for the Design of Urban Stormwater Systems
ASCE/EWRI 46-05 Standard Guidelines for the Installation of Urban Stormwater Systems
ASCE/EWRI 47-05 Standard Guidelines for the Operation and Maintenance of Urban Stormwater Systems
ASCE/SEI 48-11 Design of Steel Transmission Pole Structures
ASCE/EWRI 50-08 Standard Guideline for Fitting Saturated Hydraulic Conductivity Using Probability Density Functions
ASCE/EWRI 51-08 Standard Guideline for Calculating the Effective Saturated Hydraulic Conductivity
ASCE/SEI 52-10 Design of Fiberglass-Reinforced Plastic (FRP) Stacks

ASCE/G-I 53-10 Compaction Grouting Consensus Guide
ASCE/EWRI 54-10 Standard Guideline for the Geostatistical Estimation and Block-Averaging of Homogeneous and Isotropic Saturated Hydraulic Conductivity
ASCE/SEI 55-10 Tensile Membrane Structures
ANSI/ASCE/EWRI 56-10 Guidelines for the Physical Security of Water Utilities
ANSI/ASCE/EWRI 57-10 Guidelines for the Physical Security of Wastewater/Stormwater Utilities
ASCE/T&DI/ICPI 58-10 Structural Design of Interlocking Concrete Pavement for Municipal Streets and Roadways

FOREWORD

This Standard includes commentary and appendices that are furnished as supplemental information. The commentary and appendices are not mandatory.

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, titled 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 Manual 72 served as the primary resource document for the development of the original version of this Standard, ASCE 48-05. This book is the first revision to this Standard and is intended to replace ASCE 48-05 in its entirety.

This Standard has been prepared in accordance with recognized engineering principles and should not be used without the user's competent knowledge for a given application. The publication of this Standard by ASCE is not intended as a warrant that the information contained herein is suitable for any general or specific use, and the Society takes no position with regard to the validity of patent rights. Users are advised that the determination of patent rights or risk of infringement is entirely their own responsibility.

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INTRODUCTION

The Design of Steel Transmission Pole Structures Standard applies to cold-formed single- and multiple-pole tubular steel structures that support overhead electrical transmission lines. Design parameters are applicable to guyed and self-supporting structures using a variety of foundation types, including concrete caissons, steel piling, and direct embedment. The Standard outlines key criteria that must be considered in the structural design, detailing, fabrication, testing, assembly, and erection of these structures. This Standard is a revision of ASCE/SEI 48-05 and provides some revisions to formulas based on other current industry standards. In addition, the Standard includes a detailed commentary and appendices with explanatory and supplementary information designed to provide the user with clarification and reference information.

The information presented has been prepared in accordance with established engineering principles using state-of-the-art information and is intended for general information. Whereas every effort has been made to ensure its accuracy, the information should not be relied upon for any specific application without the consultation of a competent engineer to determine its suitability. Nothing in the Standard shall be construed to alter or subvert the requirements of any existing code or authority having jurisdiction over the facility. Furthermore, alternate methods and materials to those herein indicated may be used, provided that the engineer can demonstrate their suitability to all agencies and authorities.

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CONTENTS

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. Distribution, substation, communication, and railroad electric traction structures are not included within the scope of this 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 titled *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 Manual 72 served as the primary resource document for the development of the original version of this standard, ASCE 48-05. This document is the first revision to this standard and is intended to replace ASCE 48-05 in its entirety.

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

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Chapter 2
APPLICABLE DOCUMENTS

The following standards are referenced in this document:

American Society of Civil Engineers (ASCE):
ASCE World Headquarters
1801 Alexander Bell Drive
Reston, VA 20191-4400
www.asce.org

ASCE 10-97 Design of Latticed Steel Transmission Structures

ASTM International (ASTM) Standards:
ASTM International
100 Barr Harbor Drive
P.O. Box C700
West Conshohocken, PA 19428-2959
www.astm.org

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

American Welding Society (AWS) Standards:
American Welding Society
550 N.W. LeJeune Road
Miami, FL 33126
www.aws.org

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

American Institute of Steel Construction (AISC):
American Institute of Steel Construction
One East Wacker Drive, Suite 700
Chicago, IL 60601-1802
www.aisc.org

ANSI/AISC 360-05 Specification for Structural Steel Buildings

Research Council on Structural Connections:
The Research Council on Structural Connections
www.boltcouncil.org

RCSC—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.

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.

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.**

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. May be an agent of the owner or fabricator.

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

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.

TRUSS MEMBER. Member designed to carry only axial force.

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Chapter 4
LOADING, GEOMETRY, AND ANALYSIS

4.1 INTRODUCTION

This section details the minimum basic information that the Owner shall provide in a written specification to enable the Structure Designer to design the structure. This section 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:

1. Conductor and shield wire properties,
2. Minimum legislated loads,
3. Historical climatic conditions,
4. Structure orientation,
5. Construction and maintenance operations,
6. Line security provisions, and
7. 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, 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.

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 design restrictions, including shipping length, shipping weight, diameter, taper, deflection, finish, shaft-to-shaft connection type, foundation type, and guy attachment and anchor location 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.

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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 Materials.

5.2.1.1 Specifications. Materials 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; and
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 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 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 –20ºF (–29ºC). 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:

$$\frac{P}{A_g} \le F_t \quad \text{where} \quad F_t = F_y \qquad \text{(Eq. 5.2-1)}$$

or

$$\frac{P}{A_n} \le F_t \quad \text{where} \quad F_t = 0.83F_u \qquad \text{(Eq. 5.2-2)}$$

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 following 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:

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

$$F_a = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \quad \text{when} \quad \frac{KL}{r} > C_c \qquad \text{(Eq. 5.2-4)}$$

$$C_c = \pi\sqrt{\frac{2E}{F_y}} \qquad \text{(Eq. 5.2-5)}$$

where F_a = compressive stress permitted;
F_y = specified minimum yield stress;
K = effective length factor;
L = unbraced length;
r = governing radius of gyration;
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 [C5-3].

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

$$F_a = F_y \quad \text{when} \quad \frac{w}{t} \le \frac{260\,\Omega}{\sqrt{F_y}} \qquad \text{(Eq. 5.2-6)}$$

$$F_a = 1.42F_y\left(1.0 - 0.00114\frac{1}{\Omega}\sqrt{F_y}\,\frac{w}{t}\right) \quad \text{when} \quad \frac{260\,\Omega}{\sqrt{F_y}} < \frac{w}{t} \le \frac{351\,\Omega}{\sqrt{F_y}} \qquad \text{(Eq. 5.2-7)}$$

Dodecagonal members (bend angle = 30°)

$$F_a = F_y \quad \text{when} \quad \frac{w}{t} \le \frac{240\,\Omega}{\sqrt{F_y}} \qquad \text{(Eq. 5.2-8)}$$

$$F_a = 1.45F_y\left(1.0 - 0.00129\frac{1}{\Omega}\sqrt{F_y}\,\frac{w}{t}\right) \quad \text{when} \quad \frac{240\,\Omega}{\sqrt{F_y}} < \frac{w}{t} \le \frac{374\,\Omega}{\sqrt{F_y}} \qquad \text{(Eq. 5.2-9)}$$

Hexdecagonal members (bend angle = 22.5°)

$$F_a = F_y \quad \text{when} \quad \frac{w}{t} \le \frac{215\,\Omega}{\sqrt{F_y}} \qquad \text{(Eq. 5.2-10)}$$

$$F_a = 1.42F_y\left(1.0 - 0.00137\frac{1}{\Omega}\sqrt{F_y}\,\frac{w}{t}\right) \quad \text{when} \quad \frac{215\,\Omega}{\sqrt{F_y}} < \frac{w}{t} \le \frac{412\,\Omega}{\sqrt{F_y}} \qquad \text{(Eq. 5.2-11)}$$

where F_y = specified minimum yield stress;
F_a = compressive stress permitted;
w = flat width of a side;
t = wall thickness;
Ω = 1.0 for F_y or F_a in ksi and 2.62 for F_y or F_a in MPa; and
Φ = 1.0 for F_a in ksi and 6.90 for F_a in MPa.

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 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. Eqs. 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), Eqs. 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 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.

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°	NA	5.2-8, 5.2-9
≥22.5° but <30°	NA	5.2-10, 5.2-11
<22.5°	NA	5.2-12, 5.2-13, 5.2-14, 5.2-15, 5.2-16

Note: NA means not applicable.

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

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1 \qquad \text{(Eq. 5.2-12)}$$

where f_a = compressive stress due to axial loads;
f_b = compressive stress due to bending moments;
F_a = compressive stress permitted; and
F_b = bending stress permitted.

$$F_a = F_y \quad \text{when} \quad \frac{D_o}{t} \le \frac{3800\,\Phi}{F_y} \qquad \text{(Eq. 5.2-13)}$$

$$F_a = 0.75F_y + \frac{950\,\Phi}{\frac{D_o}{t}} \quad \text{when} \quad \frac{3800\,\Phi}{F_y} < \frac{D_o}{t} \le \frac{12{,}000\,\Phi}{F_y} \qquad \text{(Eq. 5.2-14)}$$

$$F_b = F_y \quad \text{when} \quad \frac{D_o}{t} \le \frac{6000\,\Phi}{F_y} \qquad \text{(Eq. 5.2-15)}$$

$$F_b = 0.70F_y + \frac{1800\,\Phi}{\frac{D_o}{t}} \quad \text{when} \quad \frac{6000\,\Phi}{F_y} < \frac{D_o}{t} \le \frac{12{,}000\,\Phi}{F_y} \qquad \text{(Eq. 5.2-16)}$$

where D_o = outside diameter of the tubular section (flat-to-flat outside diameter for polygonal members);
t = wall thickness; and
Φ = 1.0 for F_y, F_a, or F_b in ksi and 6.90 for F_y, F_a, or F_b in 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:

$$\frac{VQ}{Ib} + \frac{Tc}{J} \le F_v \quad \text{where} \quad F_v = 0.58F_y \qquad \text{(Eq. 5.2-17)}$$

where V = shear force;
Q = moment of section about neutral axis;
I = moment of inertia;
b = 2 times wall thickness (t)
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 either of the following:

$$\frac{Mc}{I} \le F_t \qquad \text{(Eq. 5.2-18)}$$

or

$$\frac{Mc}{I} \le F_a \qquad \text{(Eq. 5.2-19)}$$

where M = bending moment;
c = distance from neutral axis to extreme fiber;
I = moment of inertia;
F_t = tensile stress permitted; and
F_a = compressive stress permitted.

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

$$\left[\left(\frac{P}{A}+\frac{M_x c_y}{I_x}+\frac{M_y c_x}{I_y}\right)^2 + 3\left(\frac{VQ}{It}+\frac{Tc}{J}\right)^2\right]^{(1/2)} \le F_t \quad \text{or} \quad F_a \qquad \text{(Eq. 5.2-20)}$$

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

$$\left[\left(\frac{P}{A}+\frac{M_x c_y}{I_x}+\frac{M_y c_x}{I_y}\right)^2 + 3\left(\frac{VQ}{It}+\frac{Tc}{J}\right)^2\right]^{(1/2)} \le F_t \quad \text{or} \quad F_b \qquad \text{(Eq. 5.2-21)}$$

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_t = tensile stress permitted by Section 5.2.2;
P = axial force on member;
A = cross-sectional area;
M_x = bending moment about X–X axis;
M_y = bending moment about Y–Y axis;
I_x = moment of inertia about X–X axis;
I_y = moment of inertia about Y–Y axis;
c_x = distance from Y–Y axis to point where stress is checked;
c_y = distance from 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
t = wall thickness.

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:

$$P \le P_{\max} \quad \text{where} \quad P_{\max} = 0.65\,\text{RBS} \qquad \text{(Eq. 5.3-1)}$$

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.

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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 Materials. 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; and
- 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:

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

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:

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

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

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

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

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

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:

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

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:

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

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

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

and minimum bolt spacing shall satisfy the following condition:

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

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:

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

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:

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

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" (2 mm); and
γ = 0.75 when hole diameter (d_h) is ≤ 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: $0.75(F_u)$

Shear rupture on net area: $0.45(F_u)$

Note: 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 the AWS D1.1. Weld material shall be compatible with the base material as specified in the 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 center line 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

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°	Depth of chamfer minus 1/8 in. (3.2 mm)
		≥60°	Depth of chamfer
Gas metal arc or flux cored arc	All	≥60°	Depth of chamfer
	Horizontal or flat	<60° but ≥45°	Depth of chamfer
	Vertical or overhead	<60° but ≥45°	Depth of chamfer minus 1/8 in. (3.2 mm)
Electrogas	All	>60°	Depth of chamfer

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 to 6-6.

In the case where 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.

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.

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*[a]

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

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. 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).

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.

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	0.50*X* nominal tensile strength of weld metal, except shear stress on base metal shall not exceed 0.58*X* yield stress of base metal	
Tension normal to effective area	0.50*X* nominal tensile strength of weld metal, except shear stress on base metal shall not exceed 1.0*X* 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.

6.4.2 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 welding and qualifications shall be in accordance with the applicable requirements of AWS D1.1. Preheat and interpass temperatures shall be in accordance with AWS D1.1 or the steel manufacturer's recommendations. Longitudinal seam welds shall have 60 percent minimum penetration (except as specified in Sections 6.3.4, 6.3.5, and 6.4.1).

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Chapter 8
TESTING

8.1 Introduction

The Owner shall specify in the contract documents 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.

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 assembly method of the prototype shall be approved by the Owner. The completed test structure shall be erected within the tolerances established by the Owner.

8.7 Test Loads

The Owner shall specify in the contract documents which load cases shall be tested as a minimum and if the structure is to be tested to destruction. The test loads shall be the factored design loads.

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 as required by the contract document. The test report shall describe the test procedure, test results, and any remedial action taken.

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Chapter 9
STRUCTURAL MEMBERS AND CONNECTIONS USED IN FOUNDATIONS

9.1 Introduction

This section specifies design procedures for steel members and connections embedded in concrete or other backfill material. This section 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:

1. foundation type,
2. depth to point of foundation fixity,
3. design limit for foundation rotation or deflection,
4. foundation reveal,
5. coating requirements,
6. grounding requirements,
7. concrete or backfill material strength,
8. corrosion protection, and
9. 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:

$$L_d = l_d\,\alpha\beta\gamma \qquad \text{(Eq. 9.3-1)}$$

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

$$l_d = \frac{1.27\,\Gamma\,A_g F_y}{\sqrt{f'_c}} \qquad \text{(Eq. 9.3-2)}$$

or

$$l_d = 0.400\,\Phi\,dF_y \qquad \text{(Eq. 9.3-3)}$$

for #14 and #14J (45M) bars

$$l_d = \frac{2.69\,\Theta\,F_y}{\sqrt{f'_c}} \qquad \text{(Eq. 9.3-4)}$$

for #18 & #18J (55M) bars

$$l_d = \frac{3.52\,\Theta\,F_y}{\sqrt{f'_c}} \qquad \text{(Eq. 9.3-5)}$$

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 ground line 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 ground line 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 quality assurance 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 quality assurance specification shall indicate the procedure for review of the design concept, design calculations, stress analyses, and the Fabricator's drawings.

10.2.2 Materials. The quality assurance 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 quality assurance 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 quality assurance 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 quality assurance 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 upon 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 upon 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 upon 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 upon by the Owner and the Fabricator.

The quality assurance 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 Materials. The quality control 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 quality control 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 percent 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 QCI. Weld inspection shall be performed in accordance with the requirements of Section 6, Inspection, Part C, of AWS Dl.1. Personnel qualification for nondestructive weld testing shall be in accordance with Section 6.14.6.1 of AWS Dl.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, flange plates, etc., 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. This requirement may be waived if the Fabricator can demonstrate through study and quality assurance records that it can control its material, forming, welding, and galvanizing processes to the degree that such continued inspection is unnecessary.

10.3.6 Shipment and Storage. The quality control program shall provide procedures that will prevent damage, loss, or deterioration to the structure during storage and shipment.

Chapter 11
ASSEMBLY AND ERECTION

11.1 Introduction

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

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 assembled in accordance with the Structure Designer's requirements as to method, equipment, and minimum and maximum permissible assembly force, as well as within the Fabricator's specified tolerance for maximum and minimum overlap length. 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 Fabricator's recommendations.

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 and erection procedure for frame structures shall be in accordance with the Fabricator's recommendations and the requirements listed in Section 11.3 except as modified by this section.

11.4.1 Slip Joints in Frames. Slip joint connections shall be assembled as described in Section 11.3.1. On multiple-leg structures, the assembled leg-length differential shall not exceed the practical adjustment length of the foundation system.

11.4.2 Erection. Erection of frame structures shall be in accordance with the Fabricator's recommendations, 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 shall be made in such a manner as to ensure that all anchor bolt nuts are tightened to both the top and the bottom of the pole base plate in accordance with the Structure Designer's recommendations.

11.5.2 Direct-Embedded Poles. The annular opening around the embedded pole shall be backfilled with soil or concrete. Soil shall be compacted 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 requirements of the Structure Designer.

11.7 Posterection Procedures

11.7.1 Inspection. Structures shall be inspected for proper tightening of all bolted joints, condition of protective coating, and vertical alignment (plumb or rake).

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

11.7.3 Coating Repair. Per 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. Remedial measures to reduce oscillation magnitudes shall be used if damage is considered likely.

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 part of the Standard. It is intended for informational purposes only. This information is provided as explanatory and supplementary material to assist in applying the recommended requirements.

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

Chapter 4 Commentary
LOADING, GEOMETRY, AND ANALYSIS

C4.2 Loading

C4.2.1 Factored Design Loads. The ASCE has developed information to aid in the selection of loads for transmission line structures. *Guidelines for Electrical Transmission Line Structural Loading,* ASCE Manuals and Reports on Engineering Practice No. 74, 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 NESC.

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

1. Individual utility planning criteria will usually dictate specific conductor and shield wire sizes.
2. 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).
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.
4. Local terrain and line routing procedures will determine the individual structure orientation criteria.
5. 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, etc., or the need for "hot line" maintenance capability.
6. Individual utility planning criteria and experience may require the need for a load condition to prevent progressive line failure (cascading).
7. 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, using a single orthogonal coordinate system as shown in Fig. 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 ASCE Manual of Practice No. 74, *Guidelines for Electrical Transmission Line Structural Loading*. If listed separately, the use of these coefficients should be defined by a formula.

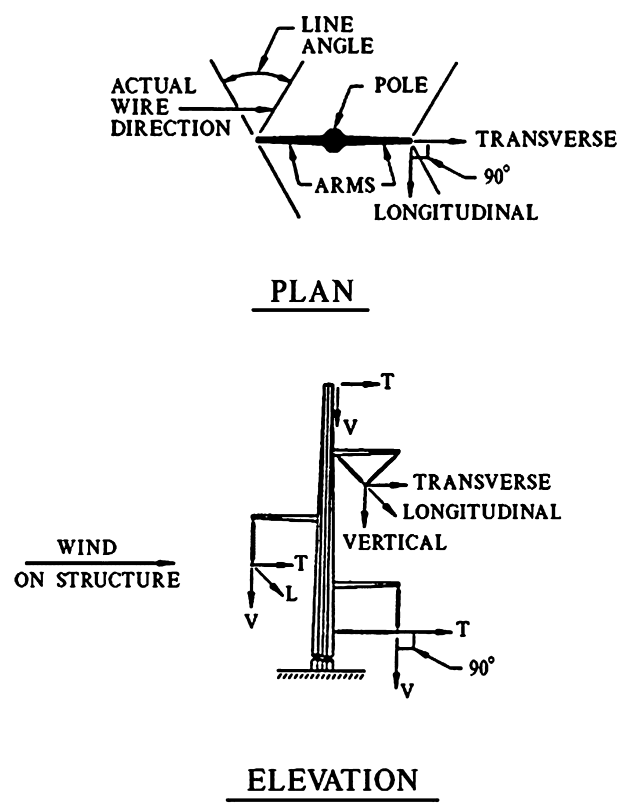

FIGURE C4-1 RECOMMENDED LOAD TREE FORMAT.

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).

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.

Generally, 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 center line 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.

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 generally nonlinear. Geometric nonlinearity (also called second-order effect or P-Delta effect) 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 where permanent damage is prohibited, a geometrically nonlinear elastic analysis is required. For the rare design conditions where 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 being 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 formulae in Appendix II.

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*, ASCE Manuals and Reports on Engineering Practice No. 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 ground line.

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.

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. Generally, 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's Standard 1307, *IEEE Standard for Fall Protection for Utility Work*.

C4.5.4 Pre-engineered Steel Poles ("Wood Pole Equivalents"). The use of steel poles for applications designed for standard-class wood poles often refers to the poles as "wood equivalent" steel poles. However, this is a misnomer as 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 05.1 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, the 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, applied wind forces, etc. 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:

1. 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 crossarms 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.

2. The 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.

3. 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. It should also be noted that the steel pole might be significantly lighter than the wood pole, decreasing its uplift resistance.

4. 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.
5. 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. Little testing of connections to pre-engineered steel poles as a system have been performed or shared with the industry. Most testing seems to have been 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 thru-bolt connection.
6. 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 is 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.
7. 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.
8. 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.
9. 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 5 Commentary
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 [C5-1, C5-2] 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 the American Institute of Steel Construction *Allowable Stress Design Specification*, 9th edition. AISC allowable stresses are applicable to equations where 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. For design of other members, the user should refer to ASCE Standard 10 [C5-3] or to the AISC Specification [C5-1], with appropriate conversions from allowable stress to ultimate strength design.

The AISC has published design criteria [C5-4] 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 formulae used in this section historically have been used in the industry to design members with cross-sectional shapes as shown in Appendix II and members with elliptical or rectangular cross sections that have maximum major to minor dimension ratios of 2 to 1.

C5.2.1 Materials

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, 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 is not recommended. Since 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. Generally, 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 impact requirements of this section are based on historical experience of structure performance. These requirements exceed those listed in Table 3.1 of ASCE Manual No. 72 [C5-5] 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.

This standard requires only heat-lot testing to be used. The requirement contained in ASCE Manual No. 72 [C5-5] for plate testing of controlled-rolled or as-rolled plates more 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 Eqs. 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, etc.). 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 [C5-1]. 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 has been shown to be dependent upon the relative load levels between the compression brace and the tension brace [C5-6, C5-7]. 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. Ref. [C5-6] provides 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 [C5-1].

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 [C5-1].

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. Eqs. 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 [C5-8] in 1987. Full-scale testing [C5-8, C5-9] demonstrates that regular polygonal shaped tubes with different numbers of sides have different buckling capacities.

Thus, different equations are provided for octagonal, dodecagonal, and hexdecagonal tubes. These equations are summarized graphically in Figs. C5-1, C5-2, C5-3, and C5-4.

Based on this testing, less conservative criteria than were previously used have been established for polygonal tubes with eight or fewer sides (Eqs. 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 Eqs. 5.2-8 and 5.2-9 should be used for tubes with eight or fewer sides.

Eqs. C5.2-1, C5.2-2, and C5.2-3 are provided here for reference. These are the elastic local buckling stresses based on E = 29,000 ksi (200 GPa) and 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°)

$$F_a = \frac{104{,}980\Phi}{\left(\frac{w}{t}\right)^2} \quad \text{when} \quad \frac{w}{t} > \frac{351\,\Omega}{\sqrt{F_y}} \qquad \text{(Eq. C5.2-1)}$$

Dodecagonal members (bend angle = 30°)

$$F_a = \frac{104{,}980\Phi}{\left(\frac{w}{t}\right)^2} \quad \text{when} \quad \frac{w}{t} > \frac{374\,\Omega}{\sqrt{F_y}} \qquad \text{(Eq. C5.2-2)}$$

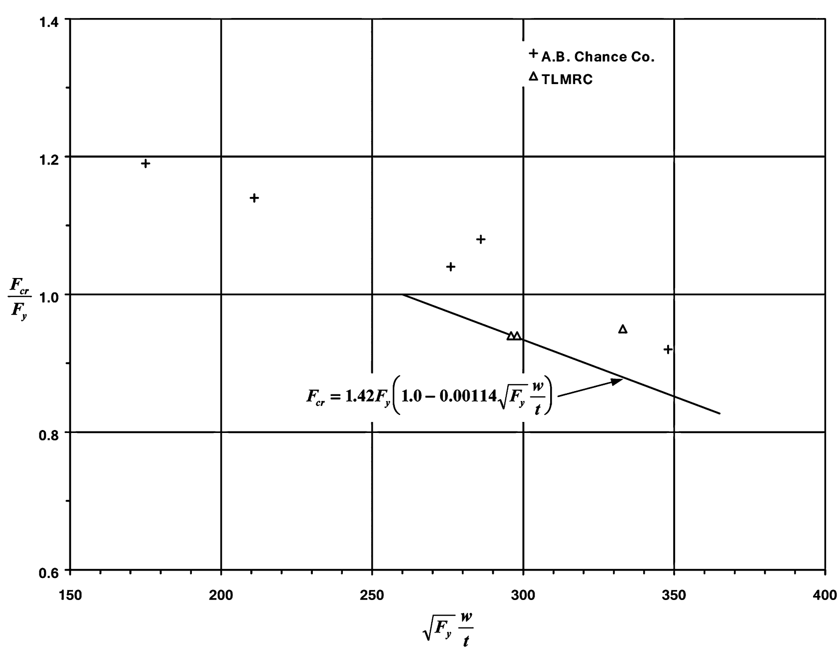

FIGURE C5-1. LOCAL BUCKLING TEST DATA FOR OCTAGONAL TUBES.

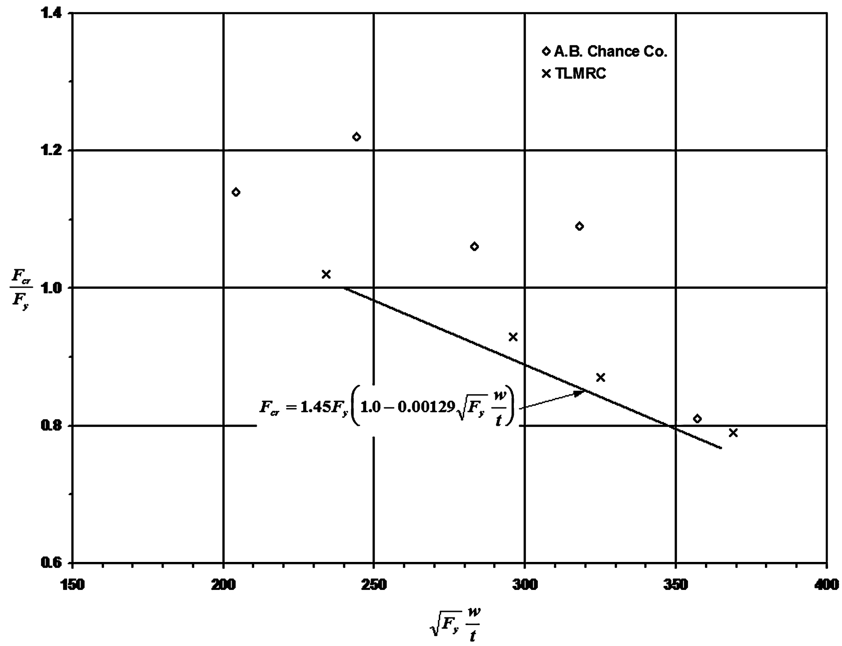

FIGURE C5-2. LOCAL BUCKLING TEST DATA FOR DODECAGONAL TUBES.

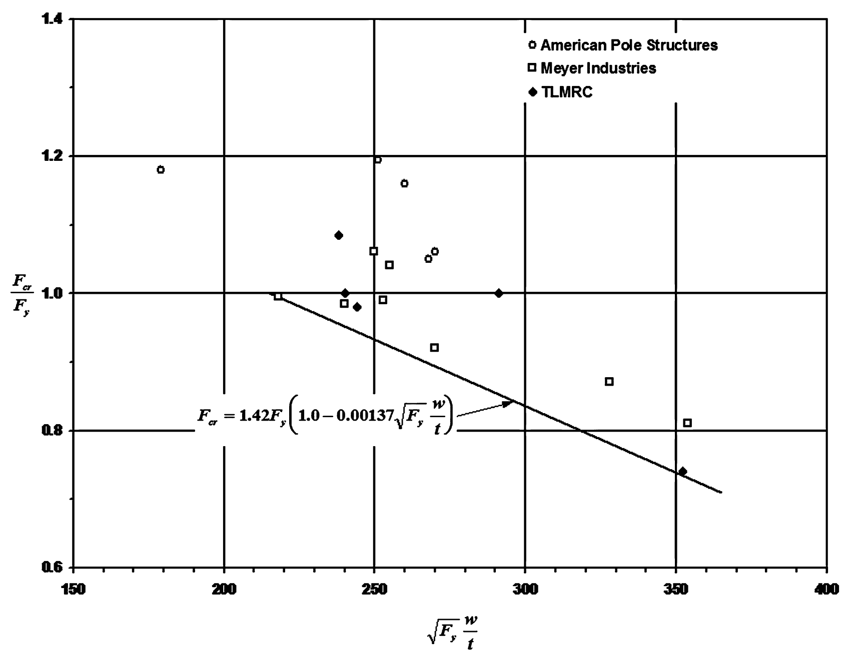

FIGURE C5-3. LOCAL BUCKLING TEST DATA FOR HEXDECAGONAL TUBES.

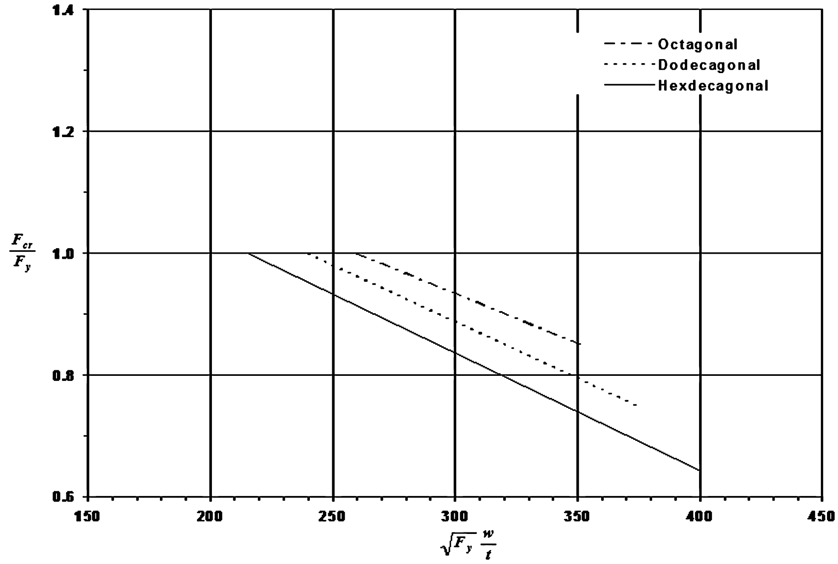

FIGURE C5-4. COMPARISON OF LOCAL BUCKLING EQUATIONS.

Hexdecagonal members (bend angle = 22.5°)

$$F_a = \frac{104{,}980\Phi}{\left(\frac{w}{t}\right)^2} \quad \text{when} \quad \frac{w}{t} > \frac{412\,\Omega}{\sqrt{F_y}} \qquad \text{(Eq. C5.2-3)}$$

where F_y = specified minimum yield stress;
F_a = compressive stress permitted;
w = flat width of a side;
t = wall thickness;
Ω = 1.0 for F_y or F_a in ksi and 2.62 for F_y or F_a in MPa; and
Φ = 1.0 for F_a in ksi and 6.90 for F_a in MPa.

C5.2.3.2.4 Round Members. Eq. 5.2-14 is the Plantema formula [C5-10, C5-11] where E = 29,000 ksi (200 GPa). Fig. C5-5 [C5-12] 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.

Eq. 5.2-16 is a modification of the Plantema formula derived from the test results shown in Fig. C5-6 and provides a good lower bound. The tests shown herein are from Refs. C5-11 and C5-12.

The use of circular tubes with D_o/t values exceeding the upper limits established by Eqs. 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. Eq. 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.

F_a 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 per ASTM A475 and aluminum clad steel strand per 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 (NESC [C5-14]). 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*, [C5-13]. Additionally, 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.

References

[C5-1] American Institute of Steel Construction. (2005). Specification for Structural Steel Buildings, Chicago.
[C5-2] American Iron and Steel Institute. (1980). Specification for the Design of Cold-Formed Steel Structural Members, New York.
[C5-3] American Society of Civil Engineers. (2000). Design of Latticed Steel Transmission Structures. ASCE 10-97, Reston, Va.
[C5-4] American Institute of Steel Construction. (1986). Load and Resistance Factor Design Specification for Structural Steel Buildings, Chicago.
[C5-5] American Society of Civil Engineers. (1990). Design of Steel Transmission Pole Structures. ASCE Manuals and Reports on Engineering Practice No. 72, New York.

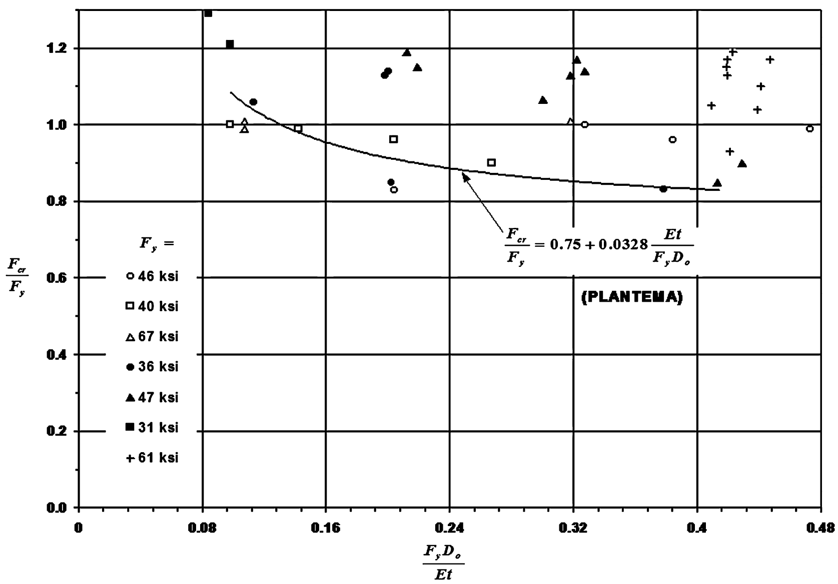

FIGURE C5-5. LOCAL BUCKLING TEST DATA FOR ROUND TUBES IN COMPRESSION.

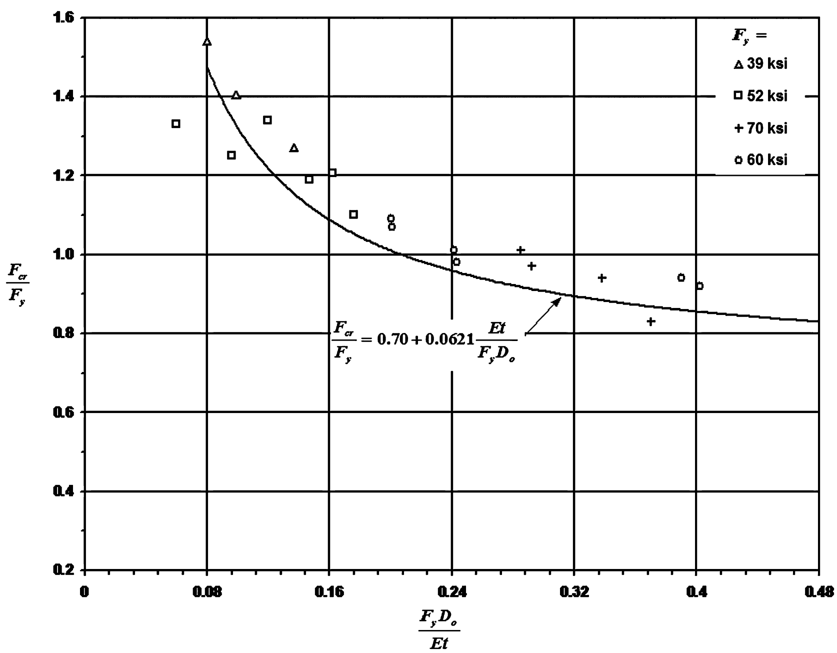

FIGURE C5-6. LOCAL BUCKLING TEST DATA FOR ROUND TUBES IN BENDING.

[C5-6] Thevendran, V., and Wang, C. M. (1993). “Stability of nonsymmetric cross-bracing systems,” Journal of Structural Engineering, 119, 1, pp. 169–180.

[C5-7] Khadivar, K. (1990). “Strength of tension/compression x-bracing systems with parallel and non-parallel posts,” Report No. TLMRC-89-R4, Transmission Line Mechanical Research Center, Haslet, Tex.

[C5-8] Cannon, D. D., and LeMaster, R. A. (1987). “Local buckling strength of polygonal tubular poles,” Report No. TLMRC-87-R3, Transmission Line Mechanical Research Center, Haslet, Tex.

[C5-9] Currance, W. C. (1974). “Local buckling stability of polygonal cross sections in bending,” paper presented at ASCE National Water Resources Meeting, Los Angeles.

[C5-10] Plantema, F. J. (1946). “Collapsing stresses of circular cylinders and round tubes,” Report No. S.280, Nationaal Luchtvaartlaboratorium, Amsterdam, Netherlands.

[C5-11] Schilling, C. G. (1965). “Buckling strength of circular tubes,” Journal of the Structural Division, 91, 5, Proc. Paper 4520, pp. 325–348.

[C5-12] “Tests on round tubes in bending.” Union Metal Manufacturing Company, Canton, Ohio, unpublished internal report.

[C5-13] American Society of Civil Engineers. (1997). Design of Guyed Electrical Transmission Structures. ASCE Manuals and Reports on Engineering Practice No. 91, New York.

[C5-14] Institute of Electrical and Electronic Engineers, Inc. (2001). National Electrical Safety Code, New York.

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Chapter 6 Commentary
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 where 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 the American Institute of Steel Construction (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 ones 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 0.375–0.5 in. (10–13 mm) oversized.

C6.2.1 Materials. Commonly used fastener specifications for steel transmission pole structures are 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); and

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.

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

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

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 is based on using a 0.75 resistance factor applied to a nominal strength of F_u times the effective net area. 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 the AISC *Allowable Bending Stress Design Aid*, 9th ed., 1987, 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 structure designer, manufacturer design/detailing practices, and construction assembly methods. Special attention may be needed on structures with high axial design loads such as H-frames or guyed poles. In these cases, the jacking force applied during assembly should exceed the design axial load.

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 manufacturer and design practice. Experience has shown that the slip joint length performance is dependent on satisfying the following assembly steps:

1. the manufacturer's minimum jacking force requirements are applied;
2. the minimum slip joint length as indicated by the manufacturer has been achieved;
3. no significant gaps between the mating sections are evident; and
4. the application of additional jacking force does not result in additional movement of the joint.

If the pole has been assembled using the manufacturer's recommended jacking force and gaps exist between the sections before wires are strung and exceed any of the following, the condition should be referred to the manufacturer for resolution:

1. The sum of the lengths of gaps that exceed 1/8 in. (3mm) is more than 30% of the slip joint's circumference, or
2. a gap extends across two full adjacent flats and the maximum gap exceeds 0.25 in. (6mm).

Maximum lap should be restricted by practical factors, such as maintaining the minimum height of the assembled structure, minimum clearances between crossarms, and interference with climbing devices. For frame structures where leg length tolerances are critical, the Structure Designer may consider using bolted flange connections as a substitute for slip joints.

Full-scale tests performed by Sumitomo Steel (in the 1970s in Japan) and 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.

ASCE Manual of Practice No. 72 and earlier versions of that document have 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 mm to 1800mm) and plate thicknesses from 0.1875 to 1.00 in. (5mm to 25 mm) (using those with $w/t \leq 40$) provides a range of values of 1.41 to 1.53. Additionally, the majority of this testing was performed on sealed pole sections that likely provided additional strength as compared to 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 VI provides a proposed method to determine the plate thickness for a base plate supported by anchor bolts with leveling nuts.

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.

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.

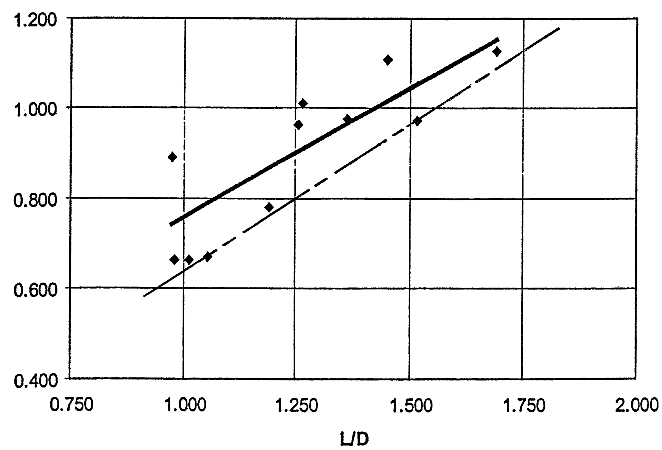

FIGURE C6-1. ALL EPRI SPLICE FAILURES.
(Figure is courtesy of Don Cannon, Project Manager, with acknowledgement to EPRI for funding the tests and to Falcon Steel, Thomas & Betts and Valmont Industries for supplying the products.)
The solid line represents the least squares fit of the data and the dashed line represents the line fitting the data for minimum strength.

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Chapter 7 Commentary
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:

1. "drilled hole" for holes specified as drilled and not punched;
2. "hot bend" when forming is to be done hot and not cold; and
3. "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 (clips and brackets, for example), 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 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 generally 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 8 Commentary
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.

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 should approve the proposed procedure for prototype testing. Also, the Structure Designer or designated representative should be present at all times during the testing sequence and approve each decision made during the process. The Owner should 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 should 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 should be approved by the Owner before testing begins.

Testing is commonly performed with the prototype in an upright position. Horizontal tests may be useful for component development and may be used for full prototype testing, providing all gravity loads are added or deducted as appropriate. Horizontal testing of full-scale structures may be used to prove the ability of a pole to withstand maximum design stress. All critical points along the pole shaft should be tested to this maximum stress level. Horizontal testing is primarily used to verify the structural integrity of freestanding, single-pole structures. One method of horizontal testing is shown in Appendix III.

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 ground line, 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 should review the testing arrangement for compliance with the contract documents.

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 case should have a maximum percent overload established before testing.

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 generally 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 has no visible local deformation after unloading. 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.

C8.15 Report

The following information is typically included in the test report:

1. The designation and description of the prototype tested.
2. The name of the Owner.
3. The name of the person or organization (Line Designer) that specified the loading, electrical clearances, technical requirements, and general arrangement of the prototype.
4. The name of the Structure Designer.
5. The name of the Fabricator.
6. A brief description and the location of the test facility.
7. The names and affiliations of the test witnesses.
8. The dates of each test-load case.
9. Design and detail drawings of the prototype, including any changes made during the testing program.
10. A rigging diagram with details of the attachment points to the prototype.
11. Calibration records of the load-measuring devices.
12. A loading diagram for each load case tested.
13. A tabulation of deflections for each load case tested.
14. In case of failure:
    a. Photographs of prototype and all failed members.
    b. Loads at the time of failure.
    c. A brief description of the failure.
    d. The remedial actions taken.
    e. The measured dimensions of the failed members.
    f. Test coupon reports of failed members.
15. Photographs of the overall testing arrangement and rigging.
16. Air temperature, wind speed and direction, precipitation, and any other pertinent meteorological data.
17. Mill test reports submitted in accordance with Section 8.3.
18. Foundation condition information.
19. Additional information specified by the Owner.

Chapter 9 Commentary

STRUCTURAL MEMBERS AND CONNECTIONS USED IN FOUNDATIONS

C9.1 Introduction

The material in Section 9.0 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. Three foundation types have become prevalent and are addressed in this Standard: drilled shaft foundation with anchor bolts (Fig. C9-1); direct-embedded foundation (Fig. C9-2); and embedded casing foundation (Fig. C9-3). Other types of foundations (spread, pile, rock anchor foundations, etc.) 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 necessary to apply an additional protective coating, such as a bitumastic compound, polyurethane coating,

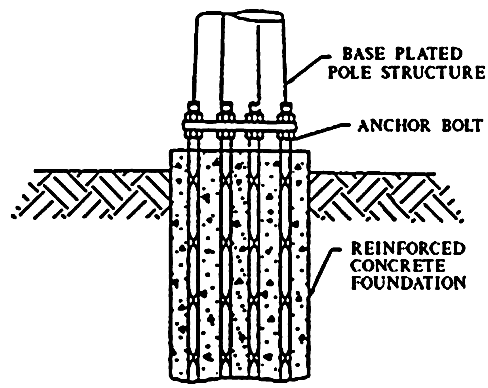

FIGURE C9-1. DRILLED SHAFT FOUNDATION WITH ANCHOR BOLTS.

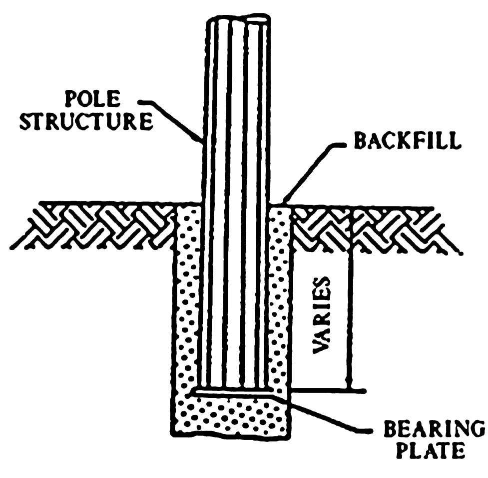

FIGURE C9-2. DIRECT-EMBEDDED POLE.

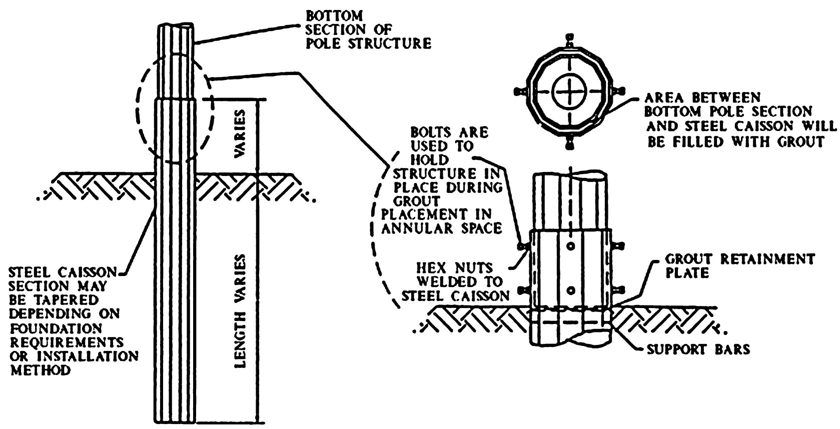

FIGURE C9-3. EMBEDDED CASING FOUNDATION.

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 ground-line forces. The Foundation Engineer designs the foundation for the same ground-line 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 #5 through #18 and grade 75 for bars #11 through #18 and #18J (ASTM A615M, grade 400 for bars 10M–55M and grade 500 for bars 35M–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 2 bolt diameters separating the bottom of the base plate and the top of the concrete. If the distance is greater than 2 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:

$$\left[\left(\frac{f_t}{F_t}\right)+\left(\frac{f_b}{F_b}\right)\right]^2+\left(\frac{f_v}{F_v}\right)^2 \le 1.0 \qquad \text{(Eq. C9.3-1)}$$

where $f_t = \dfrac{|T|}{A_s}$;

$f_b = \dfrac{0.65\, x_b V}{S_b}$;

$f_v = \dfrac{V}{A_g}$;

$F_t = 0.75\, F_u$;
$F_b = 0.90\, F_y$;
$F_v = 0.35\, F_u$;
T = maximum anchor bolt axial load (tension or compression);
V = shear per anchor bolt (shear and torsion components);
A_s = stress area per Eq. 6.2-3;
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;
$S_b = \dfrac{\pi d_e^3}{32}$; and
d_e = equivalent anchor bolt diameter based on stress area

$$d_e = \sqrt{\frac{4\,A_s}{\pi}}$$

C9.3.4 Development Length. The #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 upon 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 IV.

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 to 9 in. (76 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 to support steel structures. 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. The wall thickness of the vibratory steel caissons should be sized not only to resist the stresses caused by the ground-line reactions, but also to prevent buckling or fatigue cracking during installation. A minimum wall thickness of 3/8 in. (10 mm) is common. The geotechnical design parameters for the design of vibratory steel caissons in looser soils are improved because of densification caused by the vibratory installation.

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Chapter 10 Commentary
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 quality control 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 quality assurance specification, that fabrication procedures are satisfactory, that tolerances are within specified limits, and that the existing quality control program is satisfactory.

C10.2.1 Design and Drawings. The quality assurance 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 non-blasted 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 upon 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 quality control 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 quality control 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 Materials. 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 generally 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 upon 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 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 ultrasonic testing 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 quality control 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 11 Commentary
ASSEMBLY AND ERECTION

C11.1 Introduction

This commentary provides information supporting and explaining the requirements of Section 11.0. Additional relevant information on assembly and erection of steel transmission pole structures can be found in Appendix V. Section 11.0 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 structure assembly and erection may be obtained from the IEEE *Guide to the Assembly and Erection of Metal Transmission Structures* (IEEE Publication 951).

C11.2 Handling

The 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 15% from the Fabricator's calculated weights because of mill and fabrication tolerances.

Pole sections can be stored at a centralized marshaling yard or at the installation site. Poles stored at a marshaling yard can be partially or fully assembled before transportation to the installation site.

Care to prevent damage to protective coatings should be taken in both the shipping and the handling of pole members. Pole sections should be protected from chain tie-downs on trucks by plastic or nylon sheets. Slings for lifting should be nylon or other nonmetallic materials.

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. 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 and as shown on the Fabricator's drawings, and proper orientation of arms, brackets, and climbing accessories should be verified. The maximum and minimum lap lengths should be marked on the lower section. The sections should be overlapped as far as possible before application of jacks or other mechanical assembly equipment. Final assembly should be made using jacking force or other means as specified. Movement of one section during final assembly using a boom or other effective means aids in the successful assembly of the slip joint.

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. The section being added should be as plumb as possible during lowering so as to prevent binding of the sections during overlapping. The upper section should be slowly lowered onto the lower section. Mechanical means to complete the joint assembly should be used as specified by the Structure Designer. 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.

Special lubricants to aid in the assembly of slip joints are generally not required. If used, the lubricant should be nonstaining and water-soluble. The use and type of lubricant should be approved by the Structure Designer.

Sections should be carefully aligned to produce a tight, even joint without major gaps between the two sections. The use of shims to fill gaps between poorly mated sections in a slip joint is not recommended. Slip joints transfer load through friction between the section surfaces, and the use of shims interferes with the load transfer and reduces the load transfer capacity of the joint.

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 as compared to 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.

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 to minimize erection effort and ensure safety. 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.

Poles may be erected using lifting lugs (if installed) or a choker. The lift point for the choker will be field-determined and depends upon 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.

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. Slip-jointed legs of frame structures should be assembled on the ground to allow variations in leg lengths caused by slip tolerance to be compensated for by adjusting the anchor bolt system or embedment length.

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 crossarm 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 checked for vertical alignment (either plumbed or raked as required by the Line Designer).

C11.5 Installation on Foundation

For related information concerning helicopter erection, see Appendix V.

C11.5.1 Anchor Bolt and Base Plate Installation. The anchor bolt and base plate foundation system typically uses two nuts to attach each anchor bolt to the pole base plate. One nut is set below the base plate, and the other nut is set above 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, one nut is installed on each anchor bolt and turned down on the bolt to allow installation of the pole base plate and the top nut. The pole is lifted and set on the anchor bolts and bottom nuts, and the top nuts are installed and hand tightened. The pole is checked for alignment and plumb or, if a compensating deflection is being set, the pole is leaned to provide the desired deflection by adjusting both top and bottom nuts as required. For frame structures being assembled in the air, the anchor bolt nuts should be left snug-tight to allow for movement until assembly is complete.

Tightening of anchor bolt nuts is accomplished by snug-tightening all top nuts first, and then snug-tightening all bottom nuts. Bottom nuts are checked to ensure firm contact with the base plate, and then all top nuts are tightened in accordance with the Fabricator's requirements.

After installation, anchor bolt nuts can be secured to prevent loosening during service. The nuts may be secured by mechanically damaging the bolt threads, using a mechanical locking system, using a jam nut, or applying a tack weld between the anchor bolt nut and the base plate. Welds should not be applied to the bolt.

C11.5.2 Direct-Embedded Poles. The pole section is placed in the excavation, aligned, and oriented. Then the excavation is backfilled. Care should be taken during the backfilling and compaction process to prevent damage to the protective coating of the embedded pole section.

Specific recommendations and requirements should be made by the Line Designer as to the type of material and method of placement of backfill. This step helps ensure that the in-service behavior of the pole is in accordance with design assumptions regarding pole rotation at groundline.

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, *Design of Guyed Electrical Transmission Structures*, ASCE Manuals and Reports on Engineering Practice No. 91.

C11.7 Posterection Procedures

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

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 plumb should be made.

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 should be cleaned using a wire brush, scraper, and/or solvent as necessary to remove rust, grease, and other foreign matter. It might be desirable to lightly sand the edges of the repair area to feather the touch-up paint 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 paint 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 paint manufacturer's recommendations are followed during field application.

C11.7.4 Unloaded Arms. Conductors or ground wires attached to arms provide a vibration damping effect to the arms. When conductors or ground wires are not installed on the arms integrally with the line construction, the arms might be susceptible to damage from wind-induced oscillations of the unloaded arm. 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.

When arms 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 dampen the oscillations. Examples of these measures are internal or external damping devices, internal cables, installing weights, temporary tiebacks, and insulator assemblies. 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 V.

C11.7.5 Hardware Installation. Aeolian vibration of conductors and static wires is a common occurrence. The severity of the vibration depends upon 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 I
NOTATIONS

The following symbols are used in the 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_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);
C_c = column slenderness ratio separating elastic and inelastic buckling;
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 = stress, in tension or compression, on a member, ksi (MPa);
F_a = permitted compressive stress, ksi (MPa);
f_b = bending stress on a member, ksi (MPa);
f_{br} = bearing stress, ksi (MPa);
F_b = permitted bending stress, ksi (MPa);
f'_c = specified compressive strength of concrete at 28 days, 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 = shear stress, ksi (MPa);
F_v = permitted shear stress, ksi (MPa);
F_y = specified minimum yield stress, ksi (MPa);
I = moment of inertia, in.^{4} (mm^4);
I_{BCx} = anchor bolt cage moment of inertia about X–X axis, in.^{4} (mm^4);
I_{BCy} = anchor bolt cage moment of inertia about Y–Y axis, in.^{4} (mm^4);
I_x = moment of inertia about X–X axis, in.^{4} (mm^4);
I_y = moment of inertia about 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 = basic development length of anchor bolt, in. (mm);
L_d = minimum development length (embedment) 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 ground-line moment, in.-kip (mm-N);
M_x = bending moment about X–X axis, in.-kip (mm-N);
M_y = bending moment about 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 = thickness of element, in. (mm);
T = torsional moment, in.-kip (mm-N);
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 Y–Y axis, in. (mm);
y_i = distance of bolt from X–X axis, in. (mm);
α = unit factor as specified in the text;
β = unit factor as specified in the text;
γ = ratio of required tensile area to the gross area of the anchor bolt;
Γ = unit factor as specified in the text;
Θ = unit factor as specified in the text;
Φ = unit factor as specified in the text; and
Ω = unit factor as specified in the text.

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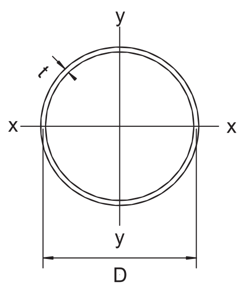
APPENDIX II
PROPERTIES OF VARIOUS TUBULAR SECTIONS

Approximate equations for commonly used section properties are shown in Figures A-II-1 through A-II-6.

Note: For polygon sections of Figs. A-II-1 to A-II-6, all properties except flat width (w) assume sharp-cornered section.

Notation for Appendix II

a = angle between the x-axis and the corner of the polygon;
A_g = gross area;



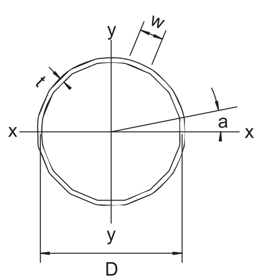
$$A_g = 3.14 \bullet D \bullet t \qquad r = 0.354 \bullet D$$

$$I_x = I_y = 0.393 \bullet D^3 \bullet t \qquad Max.\frac{Q}{It} = \frac{0.637}{D \bullet t}$$

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

$$C_y = 0.5 \bullet (D+t) \bullet \sin(a) \qquad Max.\frac{C}{J} = \frac{0.637 \bullet (D+t)}{(D^3 \bullet t)}$$

FIGURE A-II–1. PROPERTIES OF ROUND SECTIONS.



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

$$I_x = I_y = 0.403 \bullet D^3 \bullet t \qquad Max.\frac{Q}{It} = \frac{0.634}{D \bullet t}$$

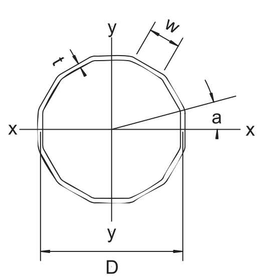
$$C_x = 0.510 \bullet (D+t) \bullet \cos(a)$$

$$C_y = 0.510 \bullet (D+t) \bullet \sin(a) \qquad Max.\frac{C}{J} = \frac{0.628 \bullet (D+t)}{(D^3 \bullet t)}$$

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

$a = 11.25°$

FIGURE A-II–2. PROPERTIES OF HEXDECAGONAL (16-SIDED POLYGON) SECTIONS.



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

$$I_x = I_y = 0.411 \bullet D^3 \bullet t \qquad Max.\frac{Q}{It} = \frac{0.631}{D \bullet t}$$

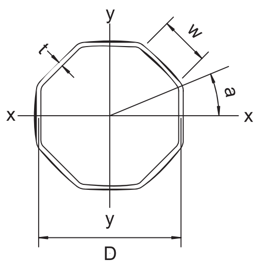
$$C_x = 0.518 \bullet (D+t) \bullet \cos(a)$$

$$C_y = 0.518 \bullet (D+t) \bullet \sin(a) \qquad Max.\frac{C}{J} = \frac{0.622 \bullet (D+t)}{(D^3 \bullet t)}$$

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

$a = 15°$

FIGURE A-II–3. PROPERTIES OF DODECAGONAL (12-SIDED POLYGON) SECTIONS.



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

$$I_x = I_y = 0.438 \bullet D^3 \bullet t \qquad Max.\frac{Q}{It} = \frac{0.618}{D \bullet t}$$

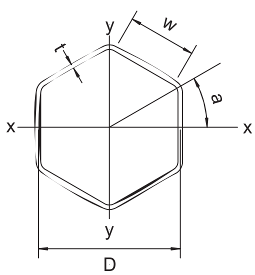
$$C_x = 0.541 \bullet (D+t) \bullet \cos(a)$$

$$C_y = 0.541 \bullet (D+t) \bullet \sin(a) \qquad Max.\frac{C}{J} = \frac{0.603 \bullet (D+t)}{(D^3 \bullet t)}$$

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

$a = 22.5°$

FIGURE A-II–4. PROPERTIES OF OCTAGONAL (8-SIDED POLYGON) SECTIONS.



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

$$I_x = I_y = 0.481 \bullet D^3 \bullet t \qquad Max.\frac{Q}{It} = \frac{0.606}{D \bullet t}$$

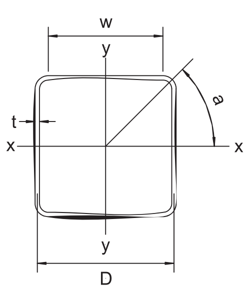
$$C_x = 0.577 \bullet (D+t) \bullet \cos(a)$$

$$C_y = 0.577 \bullet (D+t) \bullet \sin(a) \qquad Max.\frac{C}{J} = \frac{0.577 \bullet (D+t)}{(D^3 \bullet t)}$$

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

$a = 30°$

FIGURE A-II–5. PROPERTIES OF HEXAGONAL (6-SIDED POLYGON) SECTIONS.



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

$$I_x = I_y = 0.666 \bullet D^3 \bullet t \qquad Max.\frac{Q}{It_t} = \frac{0.563}{D \bullet t}$$

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

$$C_y = 0.707 \bullet (D+t) \bullet \sin(a) \qquad Max.\frac{C}{J} = \frac{0.500 \bullet (D+t)}{(D^3 \bullet t)}$$

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

$a = 45°$

FIGURE A-II–6. PROPERTIES OF SQUARE SECTIONS.

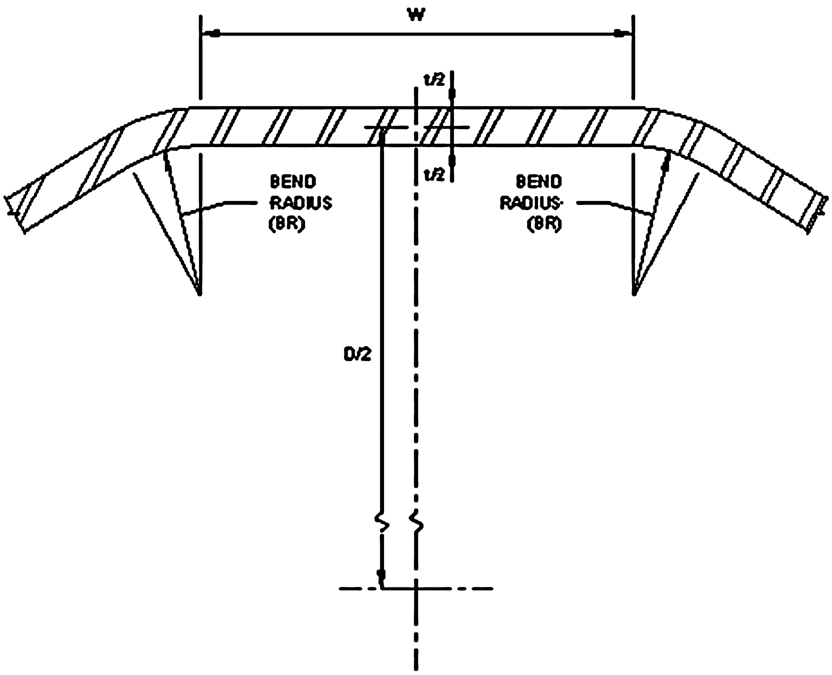


FIGURE A-II–7. TYPICAL DIMENSIONS OF POLYGON SECTIONS.

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; and
w = flat width of a side of a polygon.

APPENDIX III
HORIZONTAL TESTING

The structure is normally placed in a horizontal position as shown in Fig. A-III-1. An assembled pole is typically bolted to an identical bottom section. This bottom section is secured near the base plate to an uplift foundation while the other end rests on a compression pad. One or more locations along the shaft are selected as the load pulling points. The purpose of the load pull(s) is to duplicate maximum design stress at all critical points in the shaft based on the cross-sectional geometry of the shaft and yield strength of the material. (Critical points are those points on the shaft with the highest stress.) Axial, shear, and torsional stresses cannot be directly applied to the structure because of the test configuration. To develop comparable maximum stresses, the applied moment may be greater than the design moment.

Test Equipment

The vertical loads are pulled at predetermined points along the shaft by cranes or other suitable pulling devices. Loads may be measured using calibrated load cells located in the pulling lines. A transit set up a safe distance away from the test structure may be used to measure deflections.

Test Procedure for Pole Test

Dead Load Increase. Calculation of test loads should compensate for the dead weight of the structure in its horizontal position.

Design Load Test. Incremental loads should be pulled, as indicated in the test requirements, with deflection readings taken at predetermined points along the structure and at the uplift and compression points. Each incremental load should be held for the required time before proceeding to the next load increment. After testing the structure, it should be unloaded to the dead load so final deflection readings can be taken. A final inspection should be made of the structure.

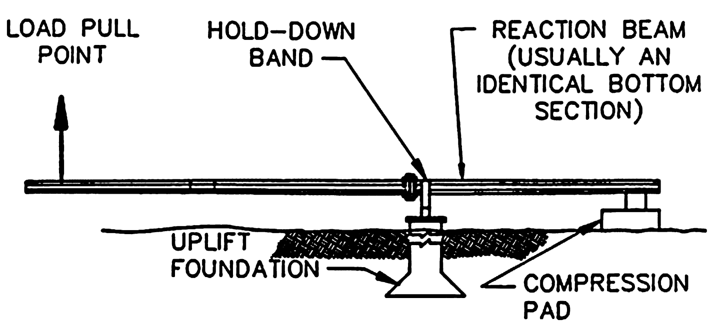


FIGURE A-III-1. HORIZONTAL TEST SETUP.

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Appendix IV
HEADED ANCHOR BOLTS

For headed bolts, materials used should be limited to ASTM A36, A193 (grade B7), A307 (grade B), A325, and A354 (grade BC).

Development Length

The development length for headed bolts is defined as the following:

$$l_d = 25d \qquad \text{(Eq. A IV-1)}$$

where l_d = embedment length of anchor bolt; and
d = bolt diameter.

Headed anchor bolts should have as a minimum a clear cover of two bolt diameters and 3 in. (76 mm) clear cover between the anchor bolts and reinforcing steel.

Background

In lieu of following ACI's methodology, where one should ensure that there is sufficient concrete area to resist the tensile forces, the object is to get the tensile load from the bolt to the adjacent vertical reinforcing bars.

Twenty-five diameters for a development length for a headed anchor bolt closely approximate the development length for the rebar following ACI 318 plus 3 in. (76mm) for top cover and 3 in. (76mm) for cover between rebar and anchor bolt. In this way, the Structure Designer provides the Foundation Designer adequate bolt length to provide sufficient vertical reinforcement for calculating reinforcing requirements.

Appendix V
ASSEMBLY AND ERECTION

Introduction

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

Helicopter Erection

Helicopter erection requires coordination between the Owner and the Fabricator as to the techniques to be used, weight limitations, special brackets, and other similar items unique to helicopter lifts. Special guides, which can be provided by the helicopter contractor, can eliminate the need for workers to be under the structure as it is being set or as sections are being added.

Helicopters can be used to transport pole sections from the marshaling yard to the structure site, or they can be used to erect structures already transported to the site. This latter technique can be used when restrictions limit the size of construction equipment at the pole site.

To lift the structure, a sling is attached to an electrically operated hook on the underside of the helicopter. A load cell is normally used to monitor the effective weight of the payload, which includes the effects of the aerodynamic drag and rotor downwash. The hook is controlled by the helicopter pilot, who can release the load if necessary. The slings should be attached to the structure in such a manner as to not overstress or excessively deflect or distort the structure.

Structures to be guyed can be flown with guys attached to the structure. Guys should be coiled and attached to the structure in such a manner as 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, and each guy should be marked to identify the proper ground anchor for attachment.

Unguyed structures should be secured on the foundation before release of the structure by the helicopter. Guyed structures should also have guys secured to the anchors before release by the helicopter.

In-Service Structure 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:

1. inspection of protective coatings and touch-up of damaged areas;
2. inspection of self-weathering structures for localized areas of continual moisture and excessive corrosion;
3. visual inspection of bolted connections and spot check of bolt tightness in selected connections;
4. visual inspection of welded connections and seams to detect cracks;
5. visual inspection of the ground-line area to ensure that soil and vegetation are not in contact with or otherwise creating a corrosive environment for the steel structure; and
6. inspection of climbing hardware and attachments for continued integrity and to ensure inaccessibility to unauthorized climbing.

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, decreasing the slenderness ratio of the member (L/r) 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 upon 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.

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APPENDIX VI
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 those that have developed 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 approach 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 being 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 in. to 6 in. (32 mm 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 A-VI-I). A comparison of these results to the thicknesses provided in the Fabricator's designs is provided in Table A-VI-1.

Review of the comparative results shows that the thicknesses for the four-bolt base plates are significantly different than the manufacturer's results, and 11 of the 23 designs were as much

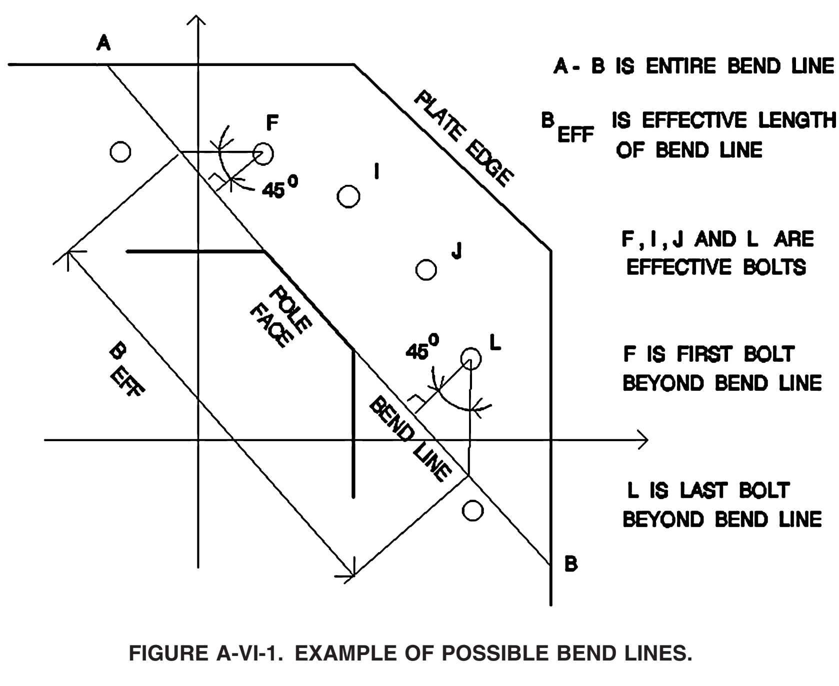

FIGURE A-VI-1. EXAMPLE OF POSSIBLE BEND LINES.

TABLE A-VI-1. Manufacturer Designs vs. ASCE 48-05 Method

Baseplate	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	–0.5
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	na	na
3-2	2.38	2.427	12 Sided	20 Bolts	0.047	na	na
3-3	2.378	2.444	12 Sided	20 Bolts	0.066	na	na
3-4	2.52	2.763	12 Sided	20 Bolts	0.243	na	na

SI Conversion: 1 in. = 25.4 mm.

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;
- *Design of Welded Structures* by Blodget; and
- Technical Manual 1, *Design of Monopole Bases* by 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: *Design of Monopole Bases* by Horn. The conclusion of that document suggests that the 48-05 method with the 45° effective bend-line limitation is appropriate. Because this method does not represent the submitted designs, several additional modifications to developing an effective bend line were considered:

- removal of all "vertex" effective bend lines;
- varying the 45-degree 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. It needs to be understood that 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. These are the elements 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 vertices 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 any bend line, the following parameters need to be established (Figure A-VI-1):

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

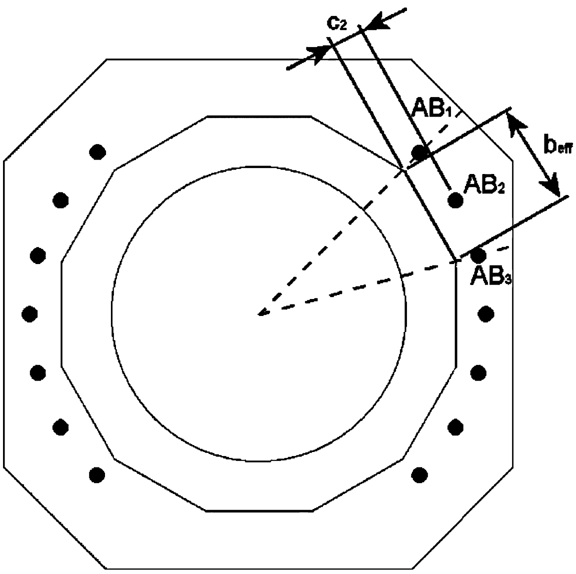

FIGURE A-VI-2. BEND LINE EXAMPLE.

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

BL = bolt load.

Figure A-VI -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 the formula:

$$f_b = \left(\frac{6}{b_{\text{eff}} t^2}\right)(\text{BL}_1 c_1 + \text{BL}_2 c_2 + \ldots \text{BL}_k c_k)$$

(Eq. A-VI-1)

The bending stress f_b for the assumed bend line in Figure A-VI-2 can be calculated by the formula:

$$f_b = \left(\frac{6}{b_{\text{eff}} t^2}\right)(1/2\text{BL}_1 c_1 + \text{BL}_2 c_2 + 1/2\text{BL}_3 c_3)$$

(Eq. A-VI-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 A-VI-1 can be rewritten as

$$t_{\min} = \sqrt{\left(\frac{6}{b_{\text{eff}} F_y}\right)(\text{BL}_1 c_1 + \text{BL}_2 c_2 + \ldots \text{BL}_k c_k)}$$

(Eq. A-VI-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 A-VI-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.

TABLE A-VI-2. Manufacturer Designs vs. Wedge Method

Baseplate	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

SI Conversion: 1 in. = 25.4 mm.

This method was developed for 8- and 12-sided shafts on base plates. The information for 8-sided poles was limited. There is not enough information at this time to extrapolate these findings for shafts of 4 sides, 16 or more sides, or round shafts.

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 by the following formula:

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

(Eq. A-VI-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;

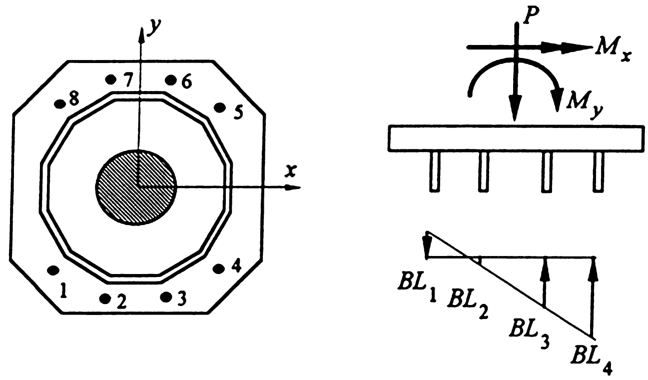

FIGURE A-VI-3. ANCHOR BOLT LOAD CALCULATION.

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;

$$I_{BCx} = \sum_{i=1}^{n}\left(A_{n(i)}y_i^2 + I_i\right) = \text{anchor bolt cage moment of inertia}$$

about X–X axis;

about X–X axis;

$$I_{BCy} = \sum_{i=1}^{n}\left(A_{n(i)}x_i^2 + I_i\right) = \text{anchor bolt cage moment of inertia}$$

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 A-VI-3 illustrates the use of Eq. A-VI-4 in establishing individual anchor bolt loads for a given pole base reaction.

APPENDIX VII
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, SSPC (Steel Structures Painting Council), 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

Whereas there are many definitions and descriptions of corrosion, the American Galvanizers Association (AGA) offers numerous publications that address this topic extensively. The AGA publication entitled *Hot-Dip Galvanizing for Corrosion Prevention: A Specifier's Guide* [A-VII-7] provides many of the key figures and descriptions of corrosion mechanisms detailed below.

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: the bimetallic couple and the concentration cell. A bimetallic couple (Fig. A-VII-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.

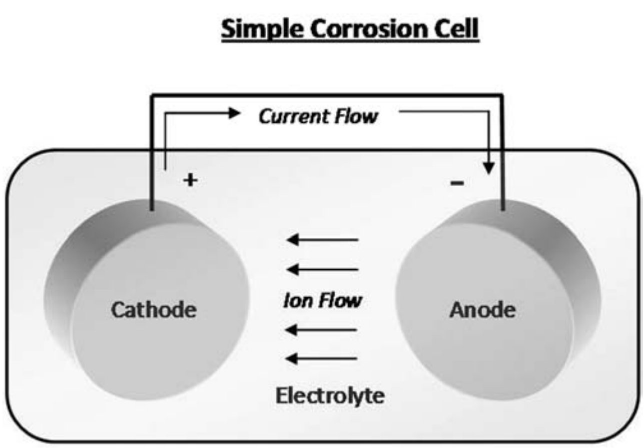

FIGURE A-VII-1. BIMETALLIC COUPLE.

In a galvanic cell there are four factors necessary for corrosion to occur:

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

The above 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 A-VII-2 lists metals and alloys in decreasing order of electrical activity. Metals nearer the top of the figure are referred 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:

1. have proven reliability based on accelerated laboratory tests and outdoor exposure tests;
2. be durable and abrasion resistant;
3. provide good cathodic protection;
4. have consistent application characteristics;
5. allow application by air or airless spraying;
6. be fast-drying to allow the product to be handled or shipped within a reasonable length of time after paint application;
7. have a topcoat that is highly resistant to chalking and fading; and
8. allow for easy field touch-up or coating repair if required.

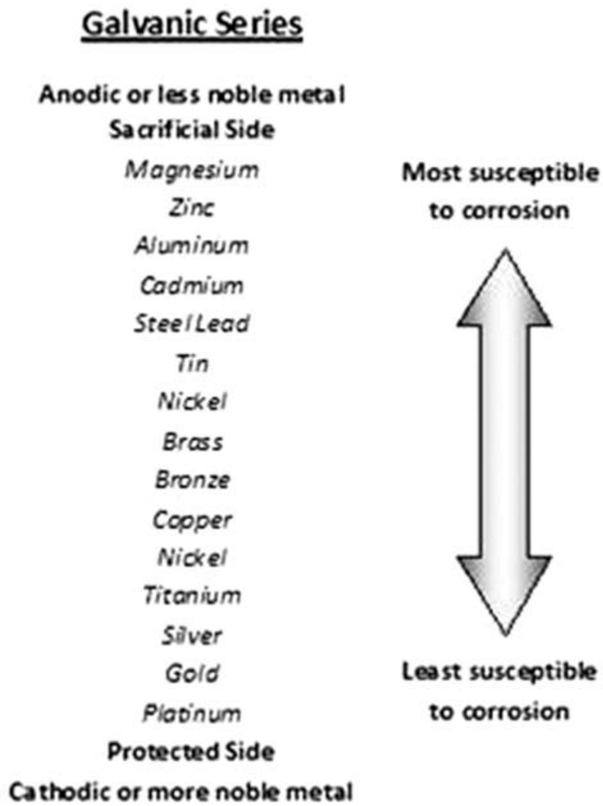

FIGURE A-VII-2. ARRANGEMENT OF METALS IN THE GALVANIC SERIES.

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 ground-line area of a direct-embedded steel pole. Depending upon 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 ground line (i.e., typically from 2 to 4 ft [0.6 m 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 SP6 or SP10 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, generally 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. This appendix 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 A-VII-3 (courtesy of the American Galvanizers Association) is a plot of this 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.

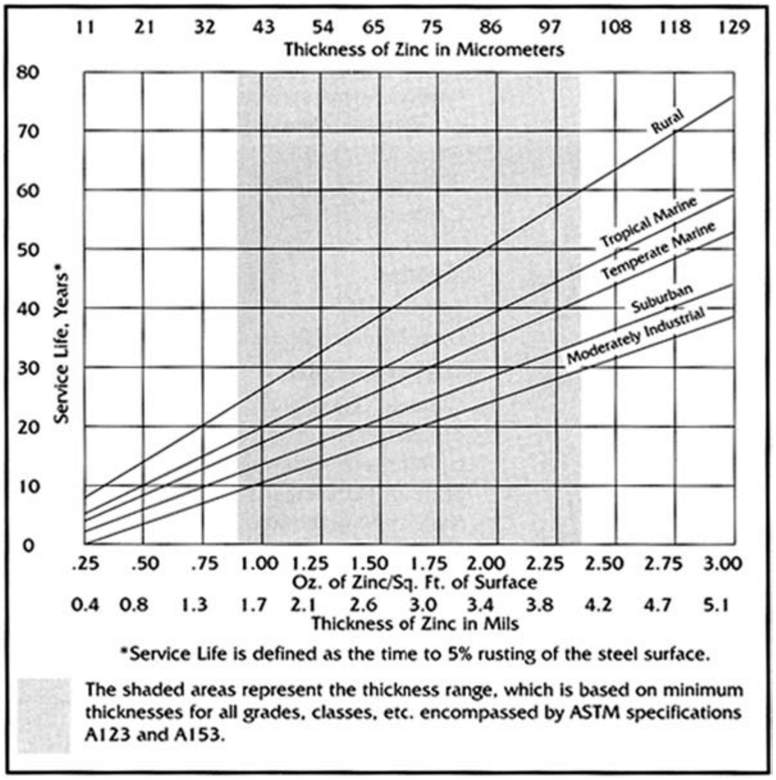

FIGURE A-VII-3. SERVICE LIFE OF HOT-DIPPED GALVANIZING.

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:

Carbon	0.25%
Phosphorus	0.05%
Manganese	1.35%
Silicon	0.06%

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 the reference *The Design of Products to be Hot-Dip Galvanized after Fabrication*" [A-VII-2] covers 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 below.

- *Steel selection*—refer to the section called Appearance above.
- *Cold working—The Design of Products to Be Hot-Dip Galvanized after Fabrication* [A-VII-2] 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 3*t*, the material should be stress-relieved at 1100° F (~600º C) for 1 hour per inch 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. (16mm) thick that are subject to tensile loading. Thicknesses less than 0.625 in. (16 mm) may be sheared.
  - Steel should be hot-worked above 1200° F (~650º 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 [A-VII-3].
- *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. If the manufacturer can provide historical proof that the practices used do not produce toe cracks after galvanizing, this requirement may be waived [A-VII-6 has additional discussion].

- *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, 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 build-up 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.

Application. The hot-dip galvanizing process consists of three basic steps: surface preparation, galvanizing, and inspection [A-VII-1].

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 upon 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 about 840° F. 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. 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 upon 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 the following:

- visual examination of samples and finished products and
- tests to determine 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 publication *Inspection of Products to Be Hot-Dip Galvanized after Fabrication* [A-VII-5] 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). Refer to AGA publication *Wet Storage Stain* [A-VII-4], AGA publication *Toe Cracks in Base Plate Welds—30 Years Later* [A-VII-6], and ASTM A780, *Standard Practice for Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings*, for methods of prevention, removal, and repair.

Acceptable methods for repairing damaged galvanized surfaces are as follows:

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 [A-VII-6].

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 past 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 mainly on the nature of the environment. Generally, 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 generally associated with the loss in metal thickness of about two 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 about 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 [A-VII-8]:

1. 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).
2. 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.)
3. Preferable fasteners are ASTM A325 Type 3 bolts installed in conformance with the *Specifications for Structural Joints Using ASTM A325 or A490 Bolts* [A-VII-9] approved by the Research Council on Structural Connections. 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 generally acceptable. This protection should extend well above the ground line 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. Upon 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. Dissimilar metals such as stainless steel, anodized aluminum, copper, bronze, and brass can generally 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:

1. wire-gas metalizing equipment,
2. powder-gas metalizing equipment,
3. electric arc metalizing equipment, or
4. 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 (AWS) Standard C2.2.

At least one layer of coating should be applied within 4 hours of blasting, and the blasted surface should be completely coated to the specified thickness within 8 hours of blasting. It is preferable to apply the full thickness within 2 hours 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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[A-VII-3] Suggested Specification for Hot-Dip Galvanizing, American Galvanizers Association, Englewood, Colo., May 2002.

[A-VII-4] Wet Storage Stain, American Galvanizers Association, Englewood, Colo., 1997.

[A-VII-5] Inspection of Products to Be Hot-Dip Galvanized after Fabrication, American Galvanizers Association, Englewood, Colo., 2001.

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[A-VII-7] Hot-Dip Galvanizing for Corrosion Prevention: A Specifier's Guide, American Galvanizers Association, Englewood, Colo., 2002.
[A-VII-8] Brockenbrough. R. L. "Considerations in the Design of Bolted Joints for Weathering Steel," Engineering Journal, First Quarter, 1983, pp. 40–45.
[A-VII-9] Specifications for Structural Joints Using ASTM A325 or A490 Bolts, Research Council on Structural Connections of the American Institute of Steel Construction, New York, 1985.

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APPENDIX VIII
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 thru plates (Figure A-VIII-1) and thru bolts (Figure A-VIII-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 in an effort to establish the primary issues that need 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 that need 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 thru 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 thru-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.
- There is a practical maximum thru-plate spacing relative to pole diameter at the elevation of the arm attachment.

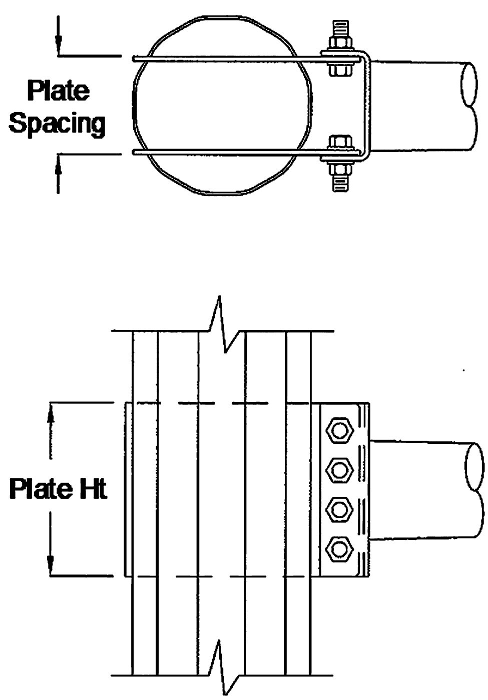

FIGURE A-VIII-1. ARM CHANNEL ON THRU-PLATES.

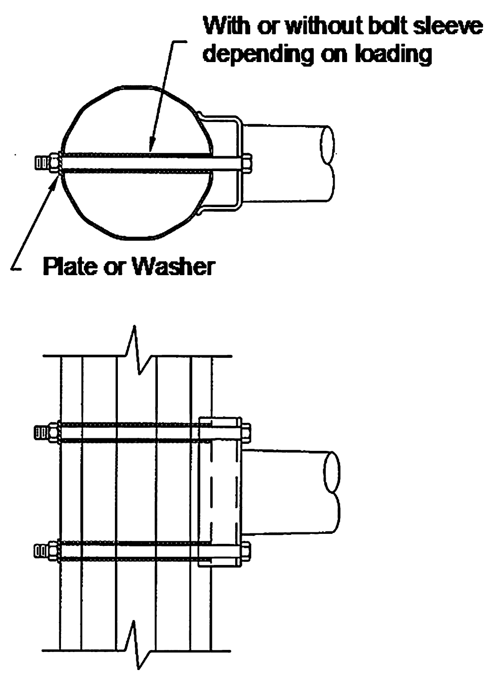

FIGURE A-VIII-2. ARM BASE WITH THRU-BOLTS.

- Bolt quantity, spacing, diameter, and grade are selected to satisfy shear criteria, including whether or not the bolts are in the shear plane.
- Thru-plate height, thickness, grade, and edge distances are selected to satisfy tension, bearing, and block shear criteria.
- Weld size and detail of the thru-plate connection to both the front and back pole shaft walls is determined to resist the applied shear loads.

Arm bases with thru 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 thru the center line 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 the base apart. 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 thru-bolt length is determined by the arm base geometry and the pole diameter. Thru-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, there are a number of other types that 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 wrap-around, band on, chain mount, and cross-bolt connections.

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