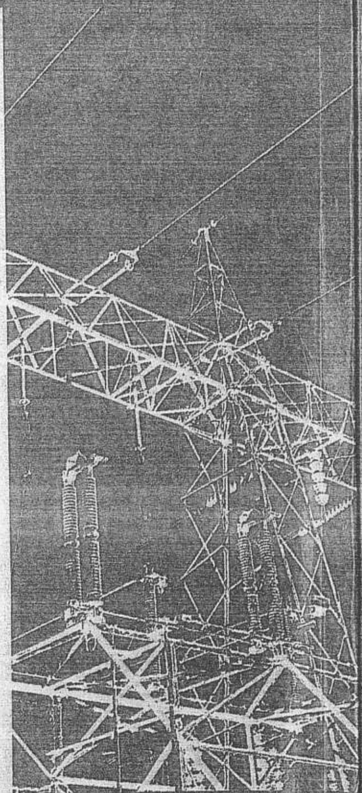
Civil Engineering
Structural Engineering
Substation Structures



Substation Structure Design Guide (ASCE Manuals and Reports on Engineering Practice No. 113) is a comprehensive resource for the structural design of outdoor electrical substation structures. Prepared by the ASCE Subcommittee on the Design of Substation Structures, this manual offers the most current guidelines available on analysis methods; structure loads; deflection criteria; member and connection design; structure testing; quality control; quality assurance; connections used in foundations; detailing; fabrication; construction; and maintenance. Utility engineers, structural and electrical engineers, and anyone that works in the field of transmission line substation design will benefit from this manual.

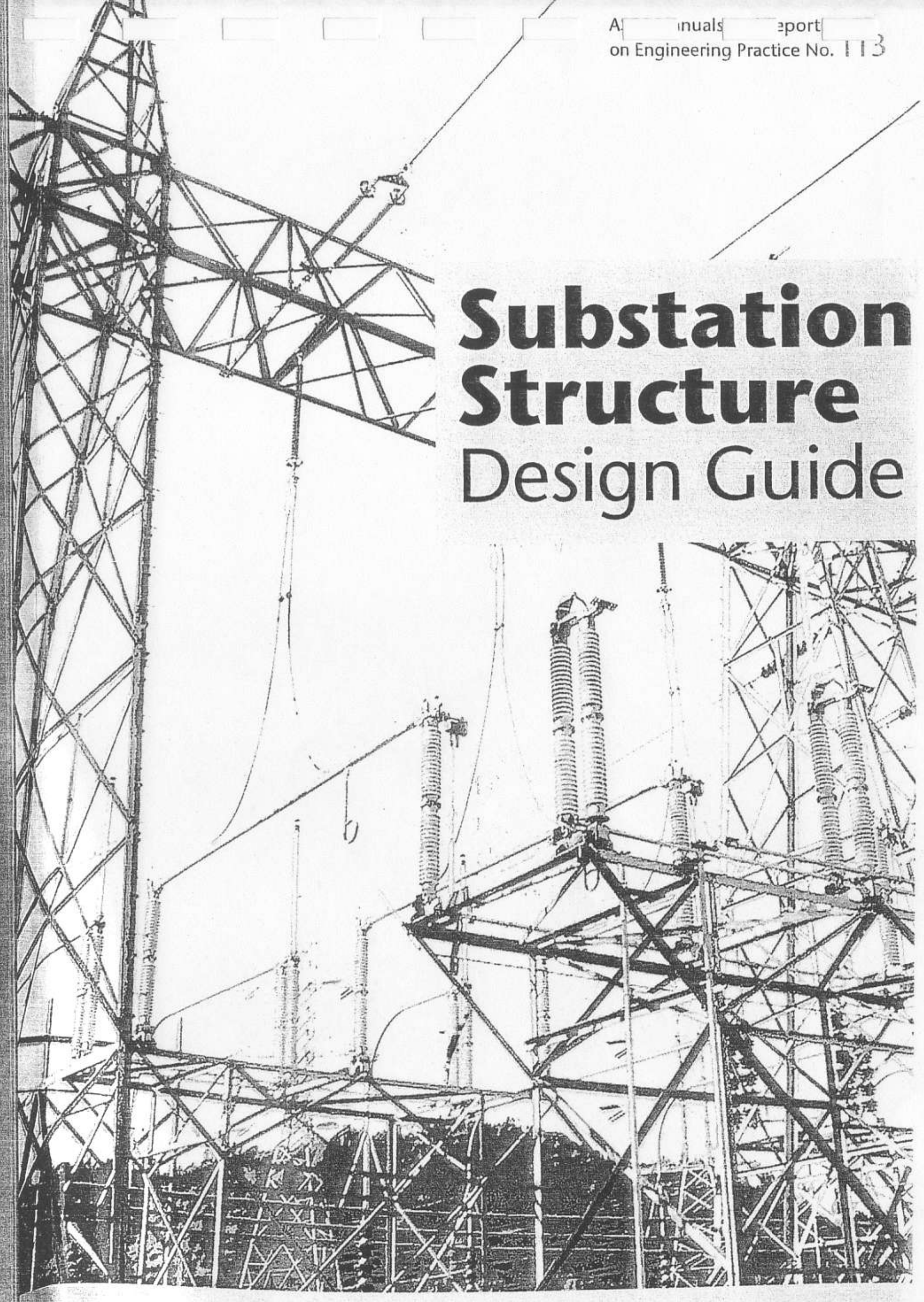
ISBN-10: 0-7844-0935-8
ISBN-13: 978-0-7844-0935-0
9 780784 409350

ASCE
American Society of Civil Engineers

SUBSTATION STRUCTURE DESIGN GUIDE
ASCE

A nuals eport
on Engineering Practice No. 113

Substation Structure Design Guide



ASCE

SEI
Structural Engineering Insti
of the American Society of Civil Engineers

ASCE Manuals and Reports on Engineering Practice No. 113

Substation Structure Design Guide

Prepared by
the Subcommittee on the Design of Substation Structures of the
Committee on Electrical Transmission Structures of the
Structural Engineering Institute of the
American Society of Civil Engineers

Edited by
Leon Kempner, Jr.

ASCE American Society of Civil Engineers

SEI Structural Engineering Institute of the American Society of Civil Engineers

Library of Congress Cataloging-in-Publication Data

Substation structure design guide : ASCE manuals and reports on engineering practice no. 113 / Prepared by the Subcommittee on the Design of Substation Structures of the Structural Division of the American Society of Civil Engineers ; edited by Leon Kempner.

p. cm.
Includes bibliographical references and index.
ISBN-13: 978-0-7844-0935-0 (alk. paper)
ISBN-10: 0-7844-0935-8 (alk. paper)
1. Structural design–Handbooks, manuals, etc. I. Kempner, Leon. II. American Society of Civil Engineers. Subcommittee on the Design of Substation Structures .
TA658.3.S83 2007
621.31'26–dc22

2007016342

Published by American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, Virginia 20191

www.pubs.asce.org

ISBN 13: 978-0-7844-0935-0
ISBN 10: 0-7844-0935-8
Manufactured in the United States of America.

15 14 13 12 11 10 09 08 1 2 3 4 5

MANUALS AND REPORTS ON ENGINEERING PRACTICE

(As developed by the ASCE Technical Procedures Committee, July 1930, and revised March 1935, February 1962, and April 1982)

A manual or report in this series consists of an orderly presentation of facts on a particular subject, supplemented by an analysis of limitations and applications of these facts. It contains information useful to the average engineer in his or her everyday work, rather than findings that may be useful only occasionally or rarely. It is not in any sense a "standard," however; nor is it so elementary or so conclusive as to provide a "rule of thumb" for nonengineers.

Furthermore, material in this series, in distinction from a paper (which expresses only one person's observations or opinions), is the work of a committee or group selected to assemble and express information on a specific topic. As often as practicable, the committee is under the direction of one or more of the Technical Divisions and Councils, and the product evolved has been subjected to review by the Executive Committee of the Division or Council. As a step in the process of this review, proposed manuscripts are often brought before the members of the Technical Divisions and Councils for comment, which may serve as the basis for improvement. When published, each work shows the names of the committees by which it was compiled and indicates clearly the several processes through which it has passed in review, in order that its merit may be definitely understood.

In February 1962 (and revised in April 1982) the Board of Direction voted to establish a series entitled "Manuals and Reports on Engineering Practice," to include the Manuals published and authorized to date, future Manuals of Professional Practice, and Reports on Engineering Practice. All such Manual or Report material of the Society would have been refereed in a manner approved by the Board Committee on Publications and would be bound, with applicable discussion, in books similar to past Manuals. Numbering would be consecutive and would be a continuation of present Manual numbers. In some cases of reports of joint committees, bypassing of Journal publications may be authorized.

MANUALS AND REPORTS ON ENGINEERING PRACTICE

CONTENTS

PREFACE

The Subcommittee on the Design of Substation Structures of the Committee on Electrical Transmission Structures of the Structural Engineering Institute of ASCE developed this manual. The subcommittee membership represented utilities, manufacturers, consulting firms, academia, research, and general interest. The combined expertise of the subcommittee members contributed to make this a valuable substation structure design guide for the utility industry.

The primary purpose of this manual is to document electrical substation structural engineering practice and to give guidance and recommendations for the design of outdoor electrical substation structures. The guide presents a review of structure types and typical electrical equipment. Guidelines for analysis methods, structure loads, deflection criteria, member and connection design, structure testing, quality control, quality assurance, connections used in foundations, detailing, fabrication, construction, and maintenance issues are presented. The recommendations presented herein are based on the professional experience of the subcommittee members, and although the subject matter of this manual has been thoroughly researched, its application should be based on sound engineering judgment.

The subcommittee wishes to thank the Peer Review Committee for their assistance and contributions to this document.

Peer Review Committee

Henry Ho, Pacific Gas & Electric
Duane Alston, Bonneville Power Administration
Kamran Khan, Trench Limited
William Mills, FWT, Inc.

Hay Yin Yu, San Diego Gas & Electric
Hanna Abdallah, Arizona Public Service
Michael Brown, Allgeier, Martin and Associates
Otto Lynch, Power Line Systems
Jerry Tang, Florida Power & Light (retired)

The subcommittee thanks all the individuals who have contributed to the completion of this manual. Without their contributions, guidance, and dedication, this manual would not have been published. The following individuals have contributed to this manual, either as past subcommittee members or as corresponding members:

David Ackermann	Michael F. Banat	Dale Beason
Richard Byrne	Patrick A. Calizar	Bing C. Chan
J. R. Clayton	Warren Crossman	Jean-Bernard Dastous
Gary Engmann	Gertrud P. Germann	Steven Groom
William J. Hamilton	Husein Hasan	Don G. Heald
Curt Hinkel	R. N. Hutcherson	David Insinger
Mircea Iordanescu	Magdi F. Ishac	James T. Kennedy
M. Kescharvarzian	Alan J. King	Brian C. Koch
Michael E. Kozlowski	Jake Kramer	Donald Laird
Don Lott	Peter Moskal	Hank Page
Al Peabody	Raphael O. Peters	Alain Peyrot
Larry (Long) Shan	Joe Shepard	M. P. Singh
T. S. Spangenberg, Jr	Lon C. Spencer	Tom Teevin
Elwood Treadwell	Gary C. Violette	A. J. Vorderbruggen

This edition of the manual is dedicated to Richard Byrne, James Kennedy, and Jake Kramer. These individuals were instrumental in initiating, contributing, and mentoring this subcommittee's activity.

ACKNOWLEDGMENTS

Prepared by the

Subcommittee on the Design of Substation Structures
of the Committee on Electrical Transmission Structures
of the Structural Engineering Institute of the
American Society of Civil Engineers

Leon Kempner, Jr., Chair

George T. Watson, Co-chair	Wendelin H. Mueller, Secretary
Reyes M. Barraza	Terry G. Burley
Martin L. de la Rosa	Harry V. Durden, Jr.
Rulon Fronk	James M. Hogan
Gary A. Johnston	Massoud Khavari
Steve M. Krohn	Paul M. Legrand II
Denis R. Lemelin	William L. Magee
Kenneth C. Malten	Jean-Robert Pierre
Craig H. Riker	Wayne P. Schumm
David Tennent	Albert J. Tharnish
C. Jerry Wong	

Committee on Electrical Transmission Structures

Habib J. Dagher	James W. Davidson
Dan Jackman	Leon Kempner, Jr.
Robert E. Nickerson, Chair	Wesley J. Oliphant
C. Jerry Wong	

CHAPTER 1
INTRODUCTION

The purpose of this manual is to provide a comprehensive resource document for the structural design of outdoor electrical substation structures. The recommendations herein apply to substation structures that support electrical equipment and rigid bus and other conductors. The electrical equipment can be of significant weight and have attachments of porcelain or composite components. Knowledge of the operational requirements of the equipment being supported is required and discussed. Deflection limits for operability can control the design of a substation structure.

Specific guidelines for structural loads, deflection limits, analysis, design, fabrication, maintenance, and construction of substation structures are recommended. Guidelines for the design of the structure connections to their foundations are presented. This manual addresses steel, concrete, wood, and aluminum used for the design of substation structures. Design equations are provided when references to existing structural design standards and codes (e.g., American Concrete Institute, American Institute of Steel Construction, American Institute of Timber Construction, and ASCE) are not appropriate or convenient. Some figures (i.e., maps and graphs) are shown for information; the user of these figures can consult the reference for more detail.

The utility industry uses both the allowable stress design (ASD) and ultimate strength design (USD) methods. *Allowable stress design* is a method of proportioning structural members such that elastically computed stresses produced in the members by nominal loads do not exceed specified allowable stresses (also called working stress design). *Ultimate strength design* is a method of proportioning structural members such that the computed forces produced in the members by the factored loads do not exceed the member design strength (also called load and resistance factored design, LRFD).

A significant issue discussed during the development of this manual was the direction it should take with respect to design of substation structures using either ASD or USD concepts. Because of the diversity in the utility industry with respect to the use of these two concepts, it was decided that both ASD and USD would be addressed. USD is the preferred method for substation structures.

Guidelines for the development of substation structure loads for wind, ice, seismic, short circuit, line tensions, equipment reactions, construction, maintenance, and regulatory codes (e.g., National Electric Safety Code (NESC 2007), General Order 95 (2006), and ASCE) are recommended. The specific recommendations are based on structure type, such as dead-end structures, disconnect switch supports, and bus supports. Recommended load factors and load combinations are presented.

The seismic load section complements IEEE 693 (2005). IEEE 693 (2005) addresses electrical equipment and its first support requirements. First support could be a pedestal for a current transformer (CT) or a support beam for a capacitor bank. This manual will reference IEEE 693 (2005) and provide seismic requirements for structures not covered by that reference.

Substation structures and the electrical equipment they support should be considered as a system. Excessive structure movement could cause the electrical equipment to experience mechanical damage, operational difficulties, and electrical faults. Recommended deflection limits and structure classes are defined in Chapter 4 of this manual.

Analysis techniques and structural modeling concepts as they relate to substation structures are discussed in Chapter 5. Both static and dynamic analyses are covered. Guidelines are given for selecting the appropriate analysis method for different structural behavior, such as large versus small displacements.

This manual references other appropriate design documents for design equations and in general notes only exceptions to the referenced documents.

Recommendations on when it is appropriate to test a unique substation structure design concept or perform individual component testing are given. Requirements for seismic testing are covered in IEEE 693 (2005).

Guidelines for quality control and quality assurance programs for substation structures are presented in Chapter 8. References are given to the appropriate industry documents that address steel, aluminum, concrete, and wood structures.

Foundation design is not presented in this manual. However, the following information should be considered for substation structure foundation design. A variety of structure types are used in electrical

substations, and these structures have a wide range of ground line reactions. Typical substation structure foundation types can be slabs on grade, spread footings, drilled shafts, and piling with and without pile caps. Substation foundations should be designed such that they do not adversely affect the deflection criteria recommended herein. The effects of soil–structure interaction from earthquakes are important and do exist, especially for large loads, such as that caused by power transformers. Foundation design should, where applicable, consider the effect of ground frost heave and the effect of buoyancy of the groundwater table. Foundations in substations should be designed according to accepted practice, the same as foundations designed for other structures. IEEE 691 (2001) is one source of information regarding the design of utility-type structure foundations.

The design of substation structure anchorage to the foundation is presented in Chapter 7. Many different types of anchorages are used to connect substation structures to their foundations. The most common anchorage is anchor bolts cast in concrete. This manual gives design recommendations for this type of anchorage. Special design considerations for seismic anchorage are covered.

The application of this manual is limited to the structural design and analysis of new electrical substation facilities. Any modification to existing structures that results in structural load variation or structural response behavior alteration should be in compliance with (a) the code or standard that was in effect at the time of the original installation, or (b) the code or standard in effect in a subsequent modification to which the structure has been previously brought into compliance, or (c) the recommendations of this manual.

CHAPTER 2

ELECTRICAL EQUIPMENT AND STRUCTURE TYPES

2.1 PURPOSE

Substation and switchyard structures are used to support the above-grade components and electrical equipment such as cable bus, rigid bus, and strain bus conductors; switches; surge arresters; insulators; and other equipment. Substation and switchyard structures can be fabricated from latticed angles that form chords and trusses, wide flanges, tubes (round, square, and rectangular), pipes, and polygonal tubes (straight or tapered). Common materials used are concrete, steel, aluminum, and wood.

This chapter gives an overview of electrical equipment, identifies the various components and structure types, and describes structure outlines. Photographs of selected substation structures are also included in this section. The photographs are shown for reference and pictorial purposes only; the structures shown are not necessarily representative of good engineering practice and are not necessarily the only support type to be utilized.

2.2 DEFINITIONS

2.2.1 Substation

A common definition is "an assemblage of equipment through which electrical energy in bulk is passed for the purpose of switching or modifying its characteristics." Larger substations may contain control houses, transformers, interrupting and switching devices, and surge protection (Fig. 2-1).

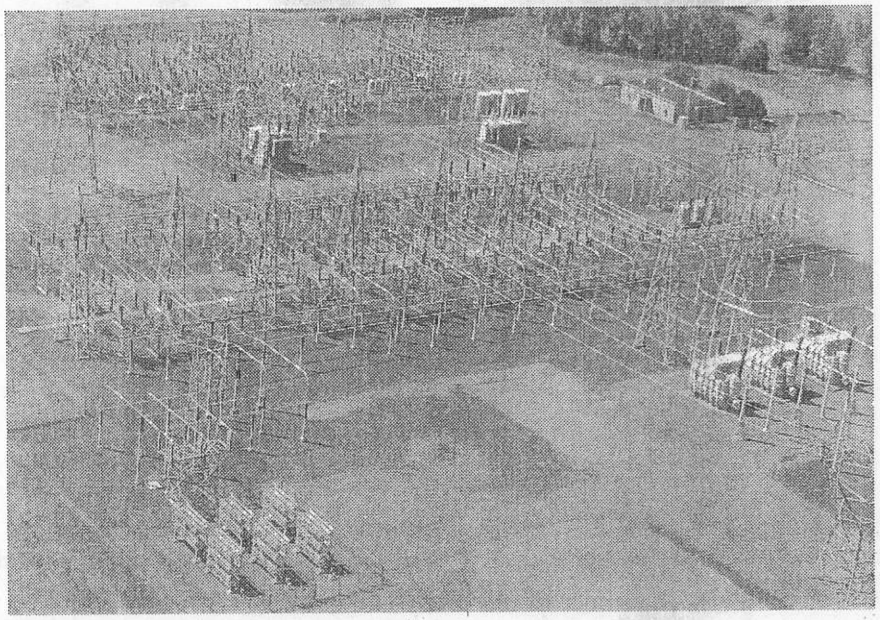


FIGURE 2-1. Substation Aerial View.

2.2.2 Switchyard

The term *switchyard* is generally applied to the assemblage of switches, power circuit breakers, buses, and auxiliary equipment that is used to collect power from the generators of a power plant and distribute it to the transmission lines at a load point. The switchyard may include step-up or step-down power transformers.

As far as structures are concerned, the terms *substation* and *switchyard* will be used interchangeably.

2.2.3 Unit Substation

For lower voltages (typically 69 kV and lower), metal-enclosed unit substations are typically used to house switches, fuses, circuit breakers, transformers, and controls. They are usually mounted on reinforced concrete pad foundations.

2.2.4 Transmission Line

Transmission lines are power lines, typically with voltages at 69 kV and above. Voltages lower than 69 kV are usually referred to as distribution lines. Transmission lines can be protected from lightning strikes by shield

wires or surge arresters. Shield wires found on transmission lines entering substations are used for protection of the substation equipment. Shield wires may also be used for communication.

2.2.5 Air-Insulated Substation and Switchyard

An air-insulated substation or switchyard has the insulating medium of air. The high-voltage bus is bare metallic tubing or cable, supported by insulators, and insulated from adjacent conductors, grounded structures, and substation grade by air. This type of switchyard or substation uses disconnect switches, which when open depend on the air for insulation across the switch's open gap. Bushings, porcelain or composite, are used to route electrical energy into the circuit breaker or the transformer.

2.2.6 Gas-Insulated Substation

An alternative to the open-air substation is a sulfur hexafluoride (SF_6) gas-insulated substation (GIS) (Fig. 2-2). The high-voltage conductors are inside a metallic sheath filled with SF_6 gas under pressure. The metallic sheath is at ground potential and can be placed at or near the ground level. Disconnect switches are located inside the metallic sheath and use

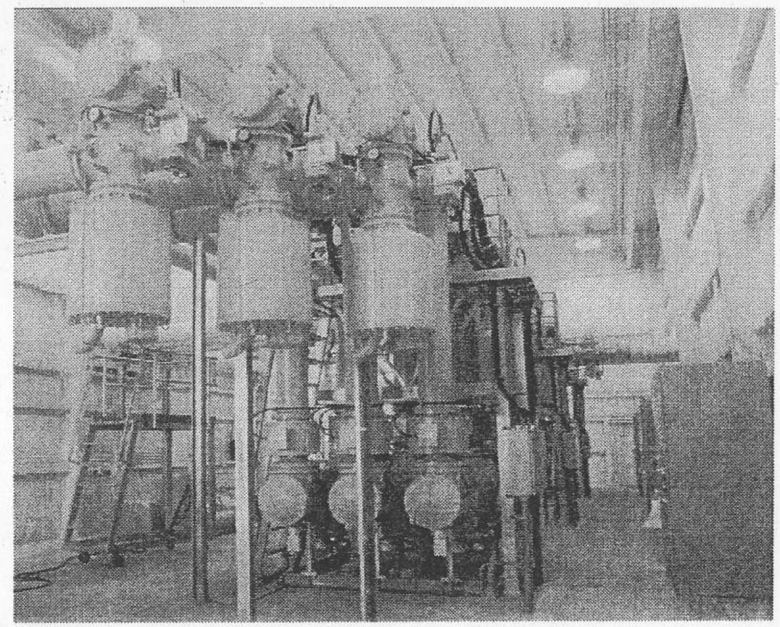


FIGURE 2-2. Gas-Insulated Substation (GIS).

SF_6 gas for phase-to-ground and open-gap insulation. Transmission lines and their associated equipment (surge arresters, wave traps, coupling capacitor voltage transformers, and line disconnect switches) usually remain air-insulated. Air-insulated leads from this equipment are routed to air-to-gas bushings.

2.2.7 Electrical Clearance

Electrical clearances provide the physical separation needed for phase-to-phase, phase-to-structure, and phase-to-ground air gaps to provide safe working areas and to prevent flashovers. Minimum electrical clearances are specified in the National Electric Safety Code (NESC 2007).

2.2.8 Buswork System

The buswork system in a substation is the network of conductors that interconnects transmission lines, transformers, circuit breakers, disconnect switches, and other equipment. The term *buswork system* includes the conductors and the material and equipment that support these conductors.

The buswork system is selected based on the desired switching arrangement and is configured to provide an orderly, efficient, reliable, and economic layout of equipment and structures. Three types of buswork systems commonly used are cable, rigid, and strain buses.

2.2.8.1 Rigid Bus System. A rigid bus conductor is an extruded metallic conductor. The conductor material is usually an aluminum alloy, but it could also be copper.

2.2.8.2 Strain Bus System. A strain bus conductor is a stranded wire conductor installed under tension.

2.2.8.3 Cable Bus System. Cable bus conductors are low-tension, stranded conductors supported on station post insulators.

2.2.9 Short-Circuit Force

Short-circuit forces are structure loads that are caused by short-circuit currents. Short-circuit currents are the result of electrical faults caused by equipment or material failure, lightning or other weather-related causes, and accidents.

The switchyard equipment and supports must be structurally adequate to permit the equipment to sustain, without damage, the severe thermal and mechanical stresses of short-circuit currents until the circuit breakers interrupt the fault current.

The short-circuit forces on the bus system will cause the phases to be attracted to or repelled from one another, depending on the direction of the short-circuit current and the phase angle. These forces cause the bus conductor to apply a transverse dynamic load, perpendicular to the bus conductor, on the insulator.

2.2.10 Dead-End Structure

Dead-end structures (also called takeoff structures, pull-off structures, termination structures, anchor structures, or strain structures) are designed to resist dead-end pulls from phase conductors and shielding wires (Fig. 2-3). Additionally, they may support switches or other electrical equipment. They can support a single bay (three-phase alternating current [AC], two-poles direct current [DC]), or multiple bays. The first dead-end structure inside the substation or switchyard is designed to support the transmission line conductors either at full tension or at a reduced tension (slack span).

2.2.11 Box-Type Structure

Box-type structures (single bay or multiple-bay space frames, also referred to as rack structures) can be used to support rigid bus conductors,

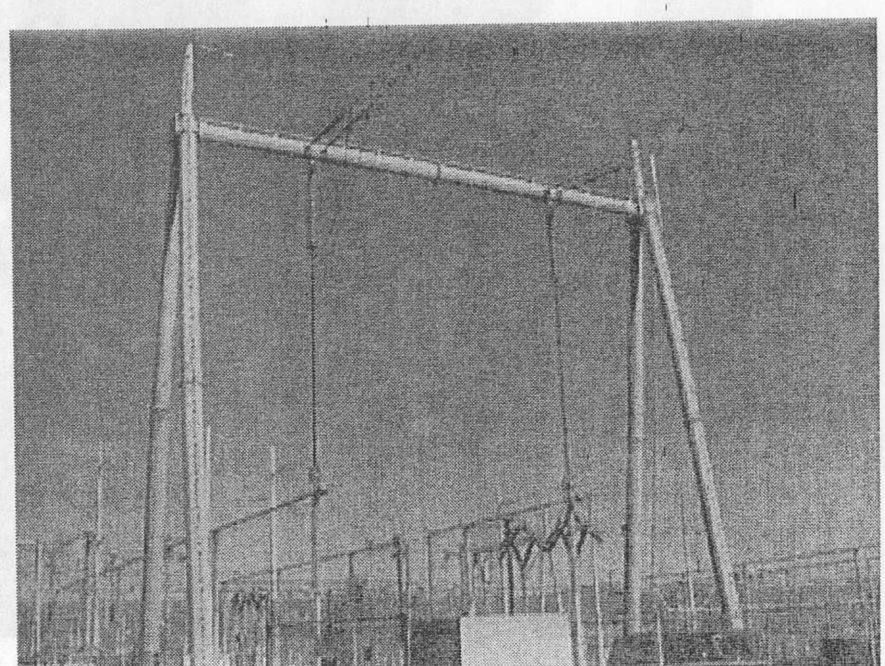


FIGURE 2-3. *Dead-End Structure, Direct Current.*

switches, and other equipment (Fig. 2-4). Box-type structures are typically used at voltages of 138 kV and below.

2.2.12 Shielding Mast

This structure (also referred to as a ground mast, lightning mast, shield wire mast, or static wire mast) shields equipment in the substation from direct lightning strikes (Fig. 2-5). These structures may or may not have overhead wires attached to enhance protection and dampen mast vibration.

2.3 ELECTRICAL EQUIPMENT AND SUPPORTS

This section provides an overview of typical types of support structures and a brief description of the electrical equipment they support. An understanding of the function, operation, and relationship of electrical equipment and the support structures is a prerequisite to good structural design.

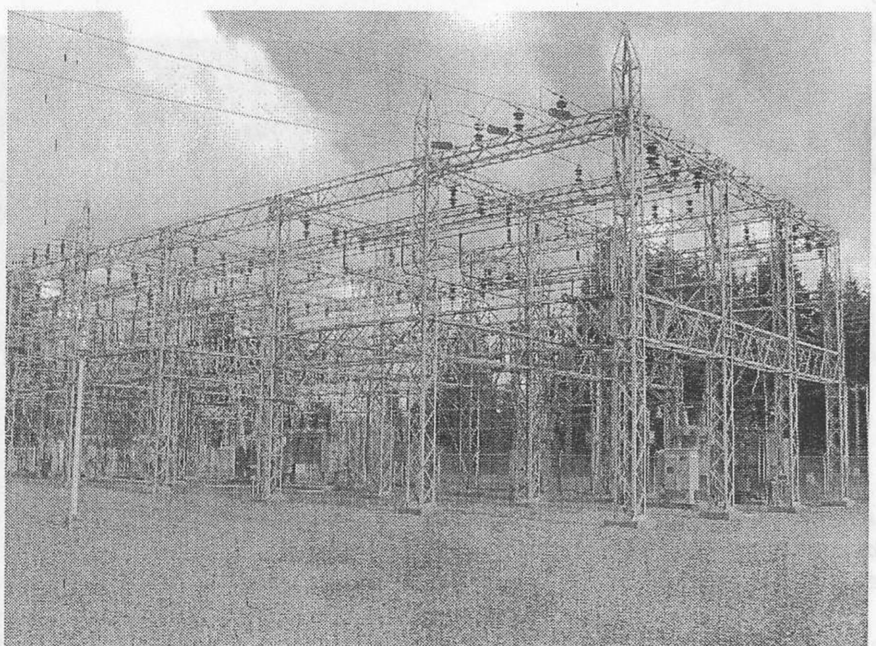


FIGURE 2-4. *Box-Type Structure.*

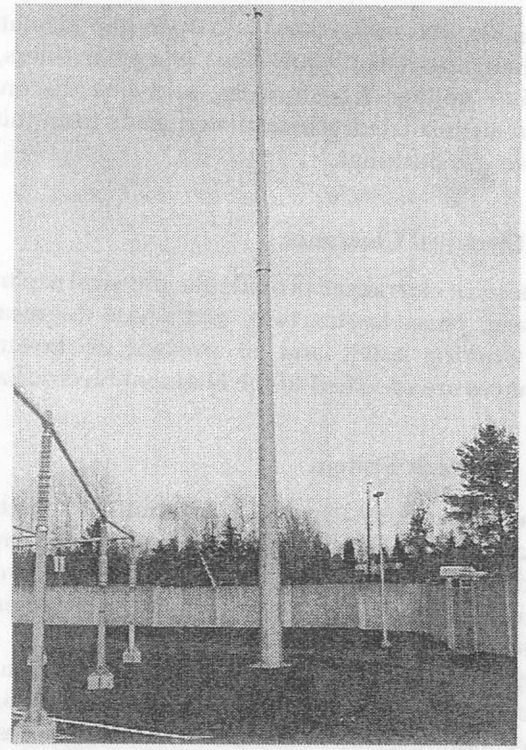


FIGURE 2-5. *Shielding Mast.*

2.3.1 Power Transformer and Autotransformer

The power transformer and autotransformer are devices used to provide a connection between power systems of different voltage levels to permit power transfer from one system to the other.

Support: The power transformer and autotransformer are supported directly on a foundation.

2.3.2 Shunt Reactor

The shunt reactor is a device used to compensate for the shunt capacitance and the resulting charging current drawn by a transmission line (Fig. 2-6). If no compensation for charging current is provided, the voltage at the receiving end of a long transmission line can exceed the sending end voltage by as much as 50% under light loading or load rejection.

Support: The shunt reactor is supported directly on a foundation.

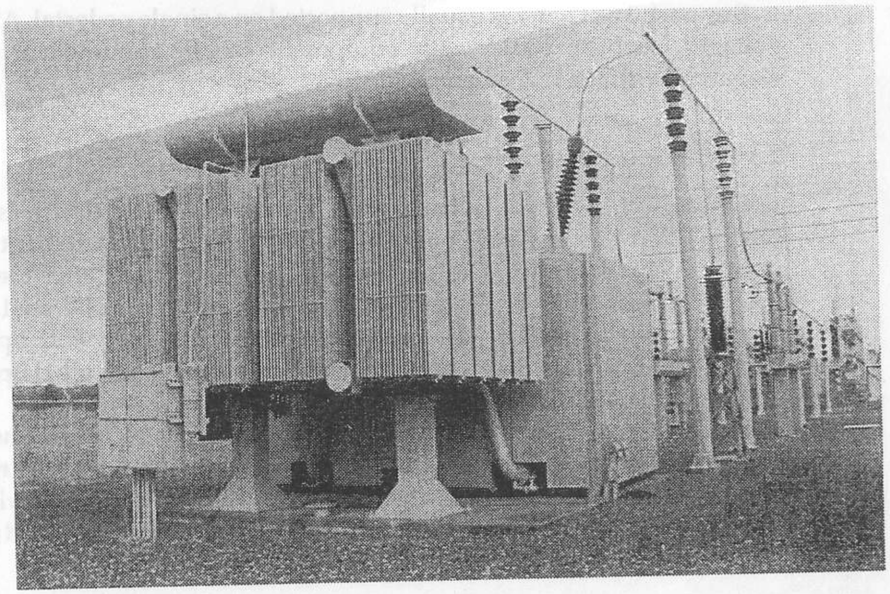


FIGURE 2-6. Shunt Reactor.

2.3.3 Current-Limiting Inductor or Air Core Reactor

A current-limiting inductor adds inductive reactance to the system and is a device used to limit the amount of short-circuit current that can flow in a circuit (Fig. 2-7).

Support: Current-limiting inductors are typically dry inductors and are usually not provided with magnetic shielding. These inductors have a magnetic field surrounding them under normal conditions, which increases in intensity when carrying short-circuit current. This magnetic field interacts with the magnetic field of other inductors if they are spaced too closely. The magnetic field induces eddy currents in metallic objects placed in the inductor's magnetic field. However, currents are induced in any closed conductive loops located within the inductor's magnetic field.

Dry current-limiting inductors are usually supported on insulators and aluminum or fiberglass pedestals to maintain the manufacturer's recommended magnetic clearance, as well as the required personnel clearance. The supporting pedestals are bolted directly to the foundation.

2.3.4 Line Trap

A line trap (also referred to as a wave trap) presents high impedance to carrier frequencies and negligible impedance to normal 60-Hz line current (Fig. 2-8). It is a blocking filter that is used to restrict the carrier signal to

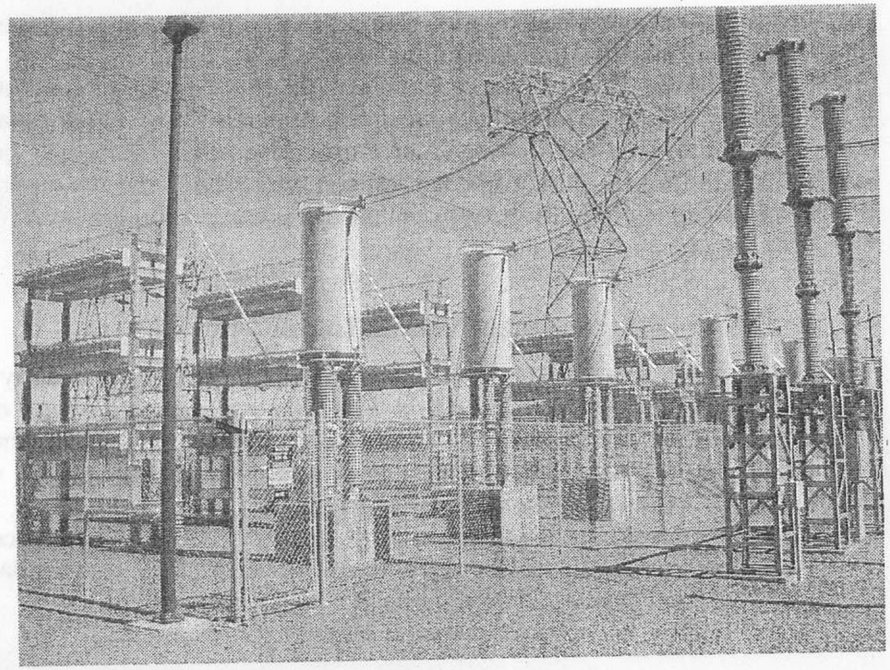


FIGURE 2-7. Current-Limiting Inductor or Air Core Reactor.

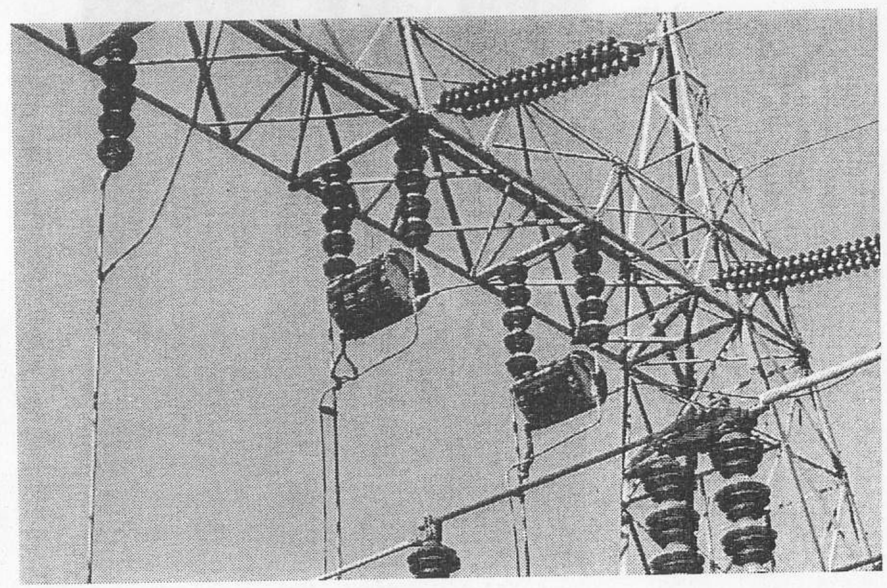


FIGURE 2-8. Line Trap.

the transmission line on which it is installed and to prevent the carrier signal from entry into substation equipment.

Support: The line trap can be mounted vertically or horizontally on either a single- or multiple-pedestal support structure. They have also been mounted with one end of the line trap supported by a coupling capacitor or coupling capacitor voltage transformer and the other end by an insulator. The line trap can also be suspension mounted from a structure.

2.3.5 Coupling Capacitor Voltage Transformer

Coupling capacitor voltage transformers, CCVT (formerly called coupling capacitor potential devices), are capacitance voltage dividers used to obtain voltages in the 66-V to 120-V range for relaying and metering (Fig. 2-9). When supplied with carrier accessories, they can be used to couple a carrier signal to a transmission line.

A coupling capacitor is a capacitance device with carrier accessories used to couple the carrier signal to the line conductor. It is similar to the CCVT, except that it does not have the voltage transformer.

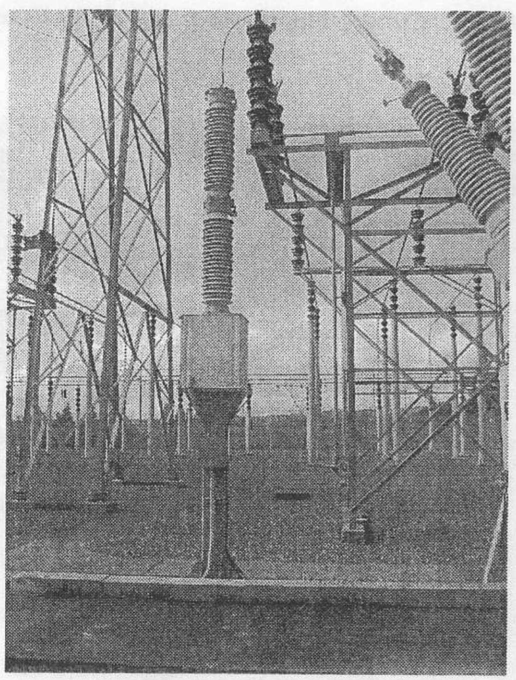


FIGURE 2-9. Coupling Capacitor Voltage Transformer.

Support: The CCVT is usually supported on a single pedestal. At higher voltages, such as 500 kV, this equipment can be mounted at ground elevation within a fenced area.

2.3.6 Disconnect Switch

Disconnect switches (when open) are used to electrically isolate a transmission line, a circuit breaker, or other electrical equipment (e.g., a transformer) (Fig. 2-10). Disconnect switches are opened after the circuit has being deenergized. They can be manually or motor operated. Motor operators are applied to disconnect switches when physical operating requirements dictate and when automatic or remote-controlled operation of the switches is required.

Disconnect switches may use a vertical break, center break (including V switches), single side break, or double side break. The side break requires larger phase spacing. The switches can be equipped with buggy whip, gas blast, and vacuum interrupting devices to give the switch some load current

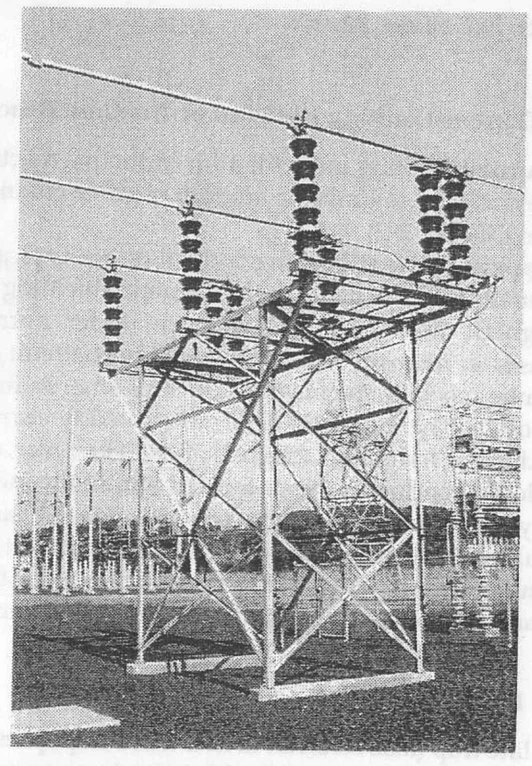


FIGURE 2-10. Disconnect Switch.

(magnetizing current) or line charging interrupting ability. Disconnect switches can also be equipped with opening and closing resistors. The disconnect switch can be provided with a maintenance grounding switch or high-speed grounding switch on either the hinge end or the jaw end of the switch. Grounding switches require their own operating mechanism and can be interlocked with the main switchblade. The plane of motion of the grounding switchblade can be either parallel to the plane of the main switchblade or perpendicular to it. If the grounding switch operates in a plane perpendicular to the main switchblade, physical clearance from the grounding switchblade in the open position must be checked.

Support: The three phases (AC) of a disconnect switch are usually supported on a common structure for voltage less than 500 kV. At 500 kV and higher voltage, a common structure would become too wide, and individual structures are used for each phase. The structure legs support the operating mechanism and control junction box. The structure should have adequate rigidity to permit proper switch operation. Each switch support may have a switch operating platform or two operating platforms if the switch has a grounding blade.

2.3.7 Load Interrupter Switch

A load interrupter switch (Fig. 2-11), sometimes referred to as a circuit switcher or line circuit breaker, is a device that combines a disconnect switch with an SF_6 interrupter. It has electrical load (energized circuit) switching capability and limited fault current interrupting capability. It provides isolation and limited fault interrupting capability in a single device.

Support: The three phases (AC) of circuit switchers are usually supported on a common structure for voltage lower than 500 kV. At 500 kV and higher voltage, a common structure would become too wide and an individual structure is used for each phase. The structure legs support the operating mechanism and control junction box.

The circuit switcher imparts a dynamic load on opening or closing, and the structure should have adequate rigidity to permit proper switching operation.

2.3.8 Circuit Breaker

Circuit breakers (Figs. 2-12 and 2-13) are used for electrical load switching and fault current interruption. They must be able to interrupt the fault current of the circuit in which they are applied. Oil, compressed air, SF_6 gas, and vacuum are used as the insulating and interrupting media in the circuit breaker. Dead tank circuit breakers typically have a current transformer at the base of one or more bushings (Fig. 2-13).

The tank referred to, with regard to circuit breakers, is the chamber that

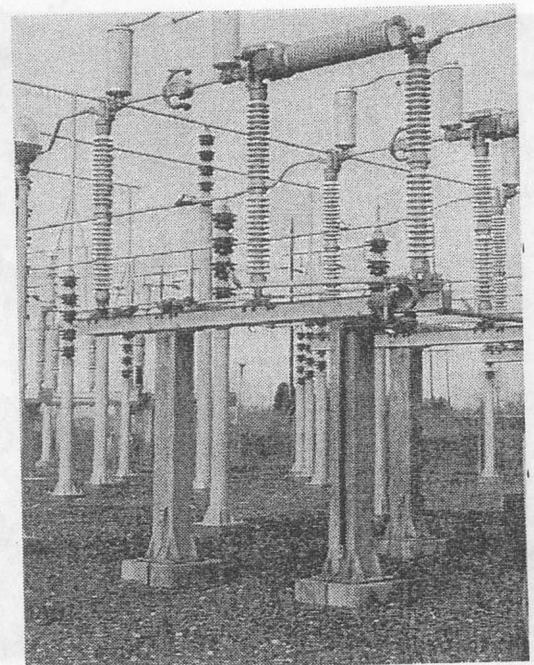


FIGURE 2-11. *Load Interrupter Switch.*

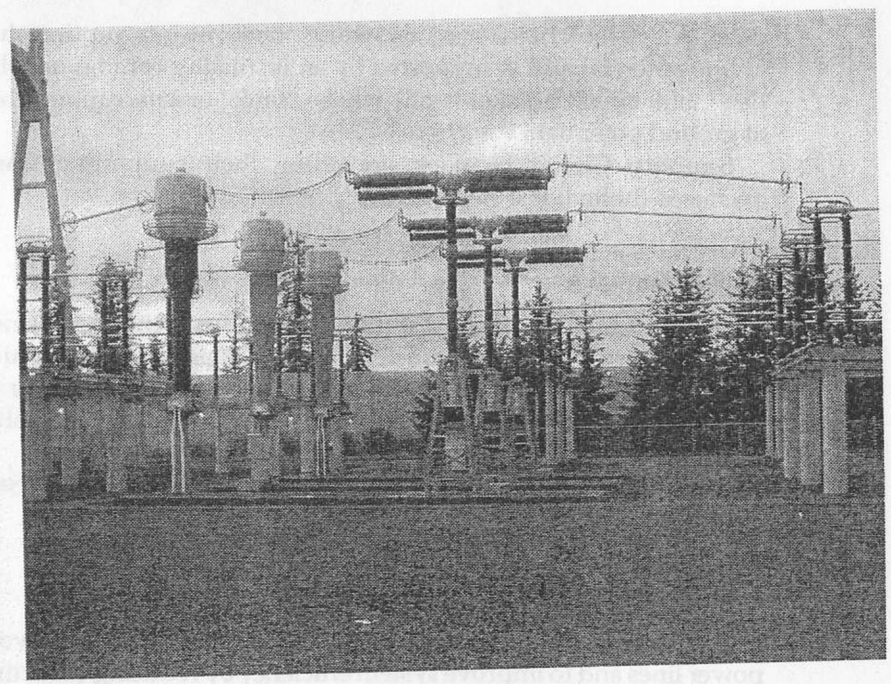


FIGURE 2-12. *Live Tank Circuit Breaker.*

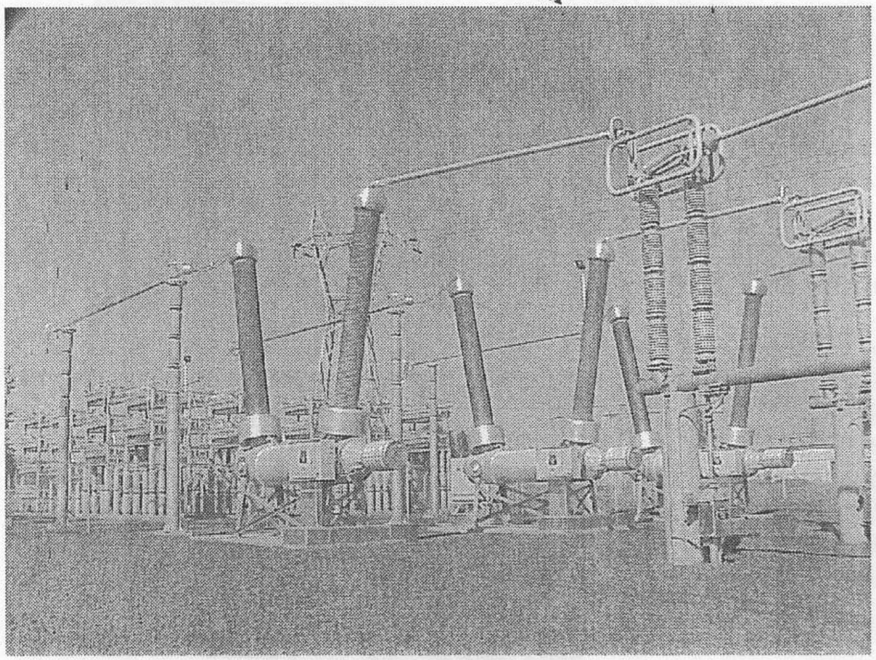


FIGURE 2-13. *Dead Tank Circuit Breaker.*

dead. A live tank breaker is one whose "tank" or interruption chamber is at line potential and is supported by an insulating column or columns. A dead tank circuit breaker is one whose "tank" or interruption chamber is at ground potential.

Support: Circuit breakers, including their supporting frames, are anchored (bolted or welded) directly on the foundation.

2.3.9 Potential and Current Transformers

Potential transformers (PTs) (Fig. 2-14) and current transformers (CTs) (Fig. 2-15) are instrument transformers that change the magnitude of the primary circuit voltage to a secondary value that is suitable for use with relays, meters, or other measuring devices. The PT measures voltage, and the CT measures current.

Support: PTs and CTs are usually supported on a single pedestal or lattice stand structure.

2.3.10 Capacitor Bank

A grouping of capacitors is used to maintain or increase voltages in power lines and to improve system efficiency by reducing inductive losses.

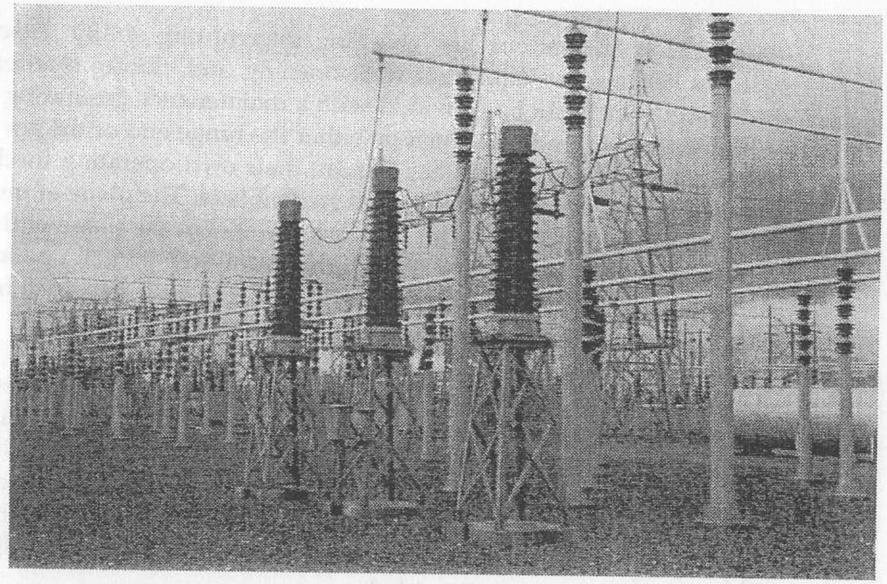


FIGURE 2-14. *Potential Transformer.*

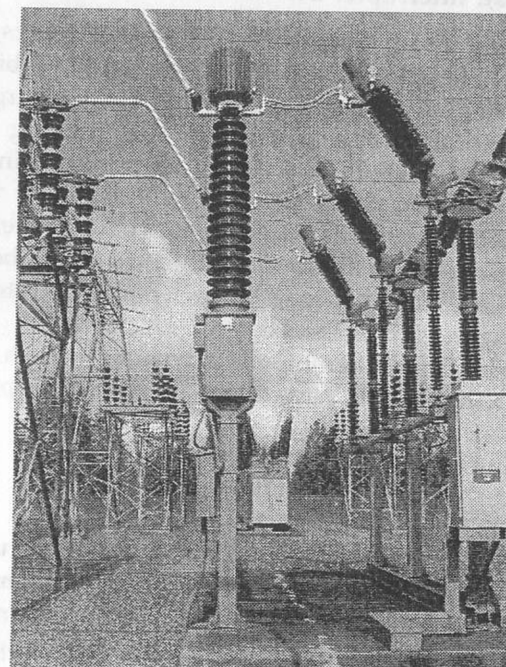


FIGURE 2-15. *Current Transformer.*

After the capacitor bank is installed, the outer periphery of the bank should be enclosed inside a fence for protection of personnel; if electrical clearance is not provided.

2.3.10.1 Shunt Capacitor. A shunt capacitor is an installation of fused or fuseless capacitors and associated equipment—generally located in substations—and used to provide reactive power to increase system voltage and to improve the power factor at the point of delivery (Fig. 2-16). The shunt capacitor improves the power transmission efficiency by adding capacitance volt-amperes reactive, VARS, to the system. They are used to stabilize the voltage in areas where loads on the system result from generation jump start-up, which can cause a voltage drop.

Support: The shunt capacitor is supported on a single frame-type structure. Shunt capacitor banks for low voltages are in an enclosed cabinet and are fastened directly to a foundation.

2.3.10.2 Series Capacitor. A series capacitor is an installation of capacitors with fuses and associated equipment in series with a transmission line. Series capacitors are used (typically at 230 kV and above) to improve power transfer capability by compensating for voltage drop along a transmission line (Fig. 2-17). Generally located near the center of a line (but they can be located at any point), they are used to increase the capability of interconnections and in some cases to achieve the most advantageous and economical division of loading between transmission lines operating in parallel. Series capacitors can also force more power to flow over the transmission line with larger conductors when parallel lines have different conductor sizes.

Support: The support is provided by a metal platform. Because the series capacitors are at line potential, the platform must be mounted on insulators that are bolted to the foundation.

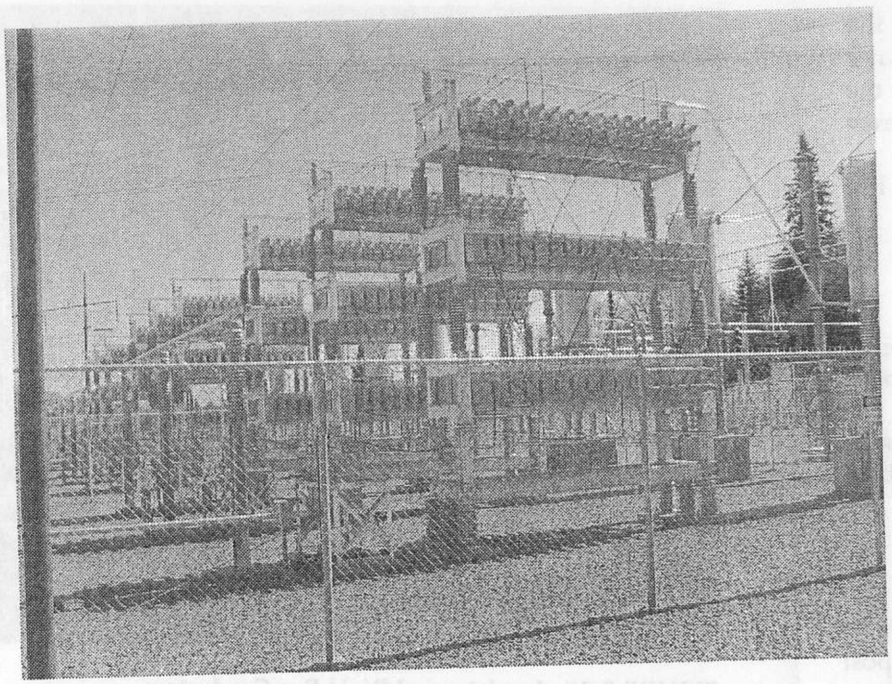


FIGURE 2-16. Shunt Capacitor.

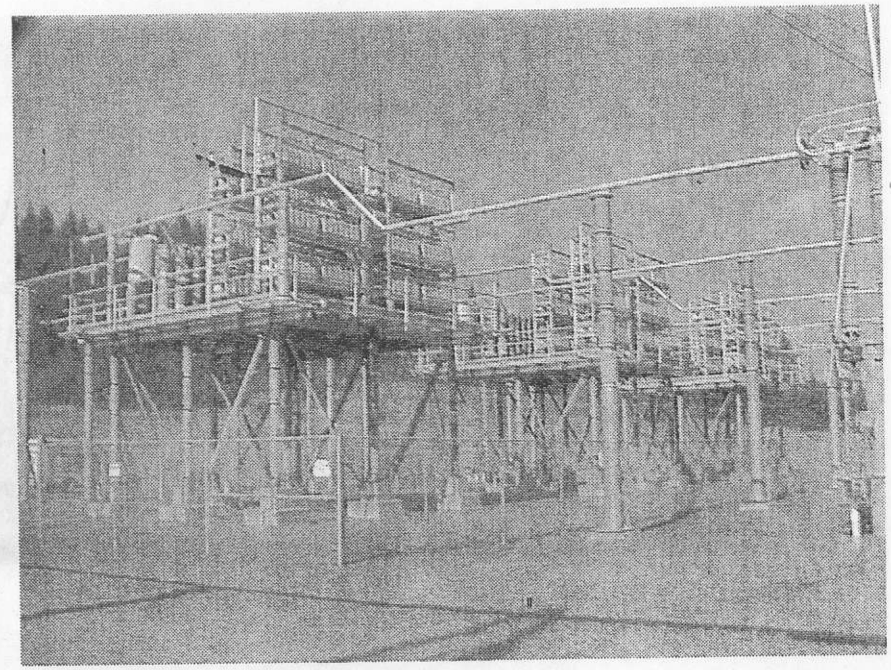


FIGURE 2-17. Series Capacitor.

2.3.11 Surge Arrester

Surge arresters, sometimes called arresters, protect power equipment from overvoltages caused by switching surges or lightning (Fig. 2-18).

Support: Surge arrester support structures are usually single-phase supports, but could be three-phase supports, depending on the voltage. Surge arresters can be supported on a single pedestal or a lattice stand structure or directly mounted on a transformer.

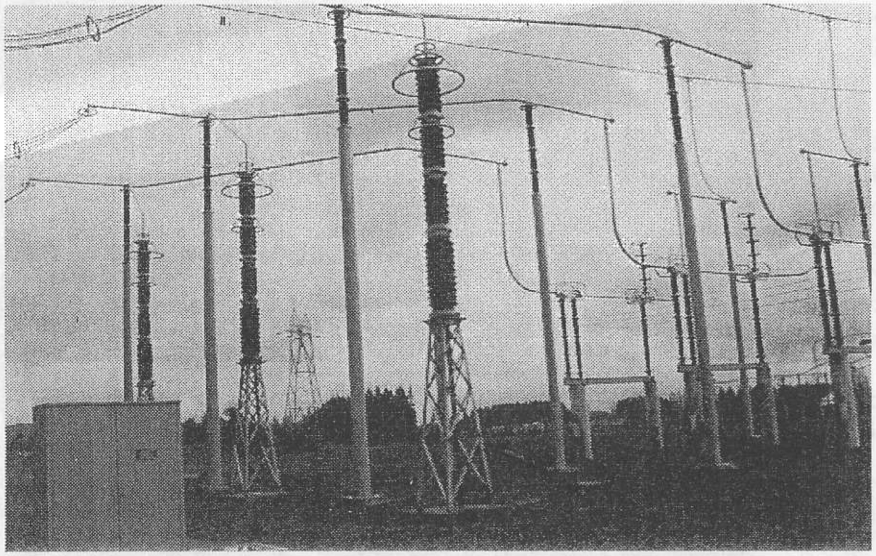


FIGURE 2-18. Surge Arrester.

2.3.12 Neutral Grounding Resistor

The neutral grounding resistor provides resistance grounding of the neutral transformers to limit ground fault current to a value that does not damage generating, distribution, or other associated equipment in the power system, yet allows sufficient flow of fault current to operate protective relays to clear the fault.

Support: Resistors are sometimes mounted on separate structures but are usually mounted on the transformer tank.

2.3.13 Cable Terminator

The cable terminator (also called a pothead) is used to change from a bare overhead conductor to a dielectric insulated cable (Fig. 2-19). Exposed cables may require protection from ultraviolet rays.

Support: Support structures of cable terminators of individual phases can be columns resting on a foundation. A structure supporting three phases can also be used.

2.3.14 Insulator

Insulators electrically isolate energized conductors or components from supporting structures. They can be either suspension or station post insulators (Fig. 2-20). Suspension insulators transfer tension forces from

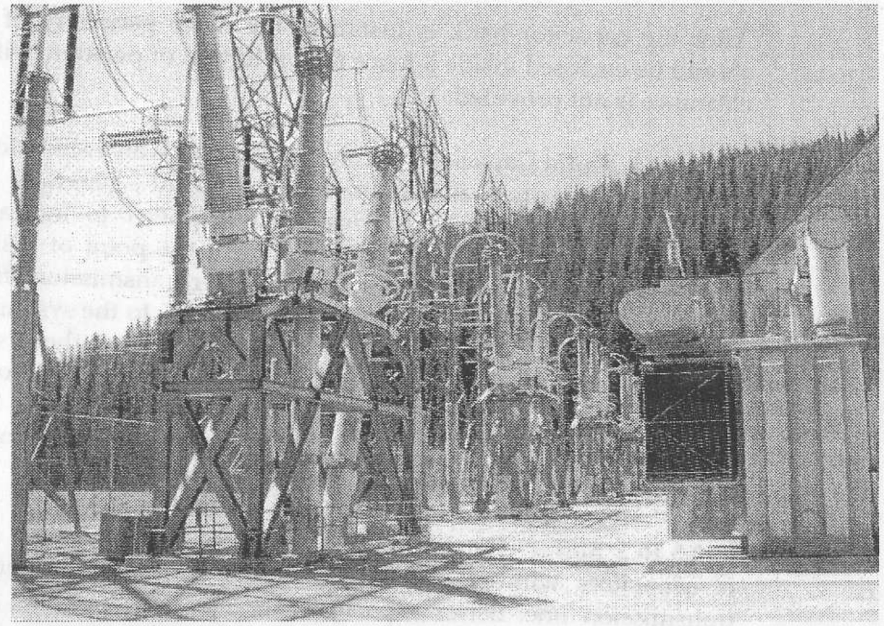


FIGURE 2-19. Cable Terminator.

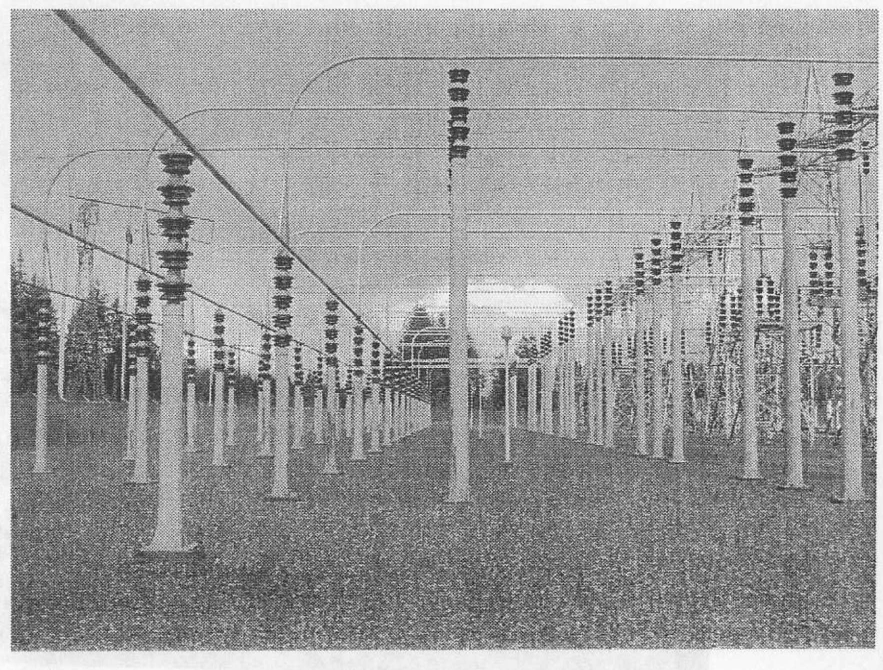


FIGURE 2-20. Insulator and Rigid Bus Conductors.

the conductor to the structure. Station post insulators can transfer tension forces, compression forces, and bending moments to the structure. They have various strength ratings and lengths for different basic impulse levels (BIL). Porcelain, glass, and composite materials are used for suspension and post insulators.

Support: Insulators can be supported on a single-phase or three-phase structure. Single phases are usually supported by a support column (Fig. 2-20). Three phases can be supported by a single structure, but they become impractical and uneconomical at 500 kV and above because the phase spacing is greater than 25 feet (7.62 m).

2.3.15 Bus Duct

There are three types of bus ducts. One is an isolated (iso) phase bus duct, which connects the generator to the main power transformer and unit auxiliary transformers. The isolated phase bus duct consists of three separate conductors, each in its own grounded metal enclosure.

The second type is a nonsegregated phase bus duct, which connects switchgear to switchgear and station auxiliary transformer to station auxiliary transformer. This type of bus duct consists of three bus bars in a common grounded metal enclosure.

The third type is a segregated bus duct and is the same as the nonsegregated phase bus duct except that there are grounded metal segregation barriers between bus bars.

Support: The iso phase bus duct may be supported from the building steel and/or generator foundation when in the building and on individual common structure supports outside the building. The segregated and nonsegregated bus ducts are typically supported from the roof purlins or joists when inside a control building and on common structure supports when outside of a building.

CHAPTER 3

LOADING CRITERIA FOR SUBSTATION STRUCTURES

All substation structures should be designed to withstand applicable loads from wind, ice, line tensions, earthquakes, construction, maintenance, electrical equipment, and other specified or unusual service conditions.

This chapter discusses guidelines for developing substation structure loading criteria. Loads and load cases recommended in this manual are considered appropriate for providing reliable substation structures described in Chapter 2, excluding control building structures. Design engineers using this manual may substitute or modify these recommendations based on experience, research results, or test data.

3.1 BASIC LOADING CONDITIONS

3.1.1 Dead Loads

Structure, support equipment, and accessory weights should be included in the dead loads applied in conjunction with applicable design loads.

3.1.2 Equipment Operating Loads

Operation of equipment, such as switches and circuit-interrupting devices, can create dynamic loading on support structures. These loads should be combined with other load cases if the equipment has to be operable when weather conditions are most severe. The equipment manufacturer should be consulted regarding the application and magnitude of such loads. Equipment manufacturers should also be consulted with regard to operational and functional deflection requirements specific to the particular equipment. Deflection limits are recommended in Chapter 4.

The design engineer should be involved in the equipment procurement review process to assess design information, such as equipment center of gravity, weights, component sizes for wind surface area, and dynamic or operating forces.

3.1.3 Terminal Connection Loads for Electrical Equipment

The primary purpose of terminal connections on substation equipment is to serve as electrical conduits to the equipment. They are not intended as a primary load support system. Terminal connection loads are forces and moments that are created by flexible and rigid bus connectors to electrical equipment. Flexible bus connectors to equipment should be installed with sufficient slack to accommodate equipment movement.

Bus connections to the terminal pads of electrical equipment use hardware that generally varies from 6 to 18 in. (15.2 to 45.7 cm) long. The hardware can act as a lever arm to create additional moments on the terminal pads. Substantial forces and moments can be transmitted to the terminal pads of the equipment by the connector. Test terminals, for use by operation and maintenance personnel, may also be installed between the hardware and the terminal pads. Test terminals increase loads on the terminal pads.

Terminal pad connection capacities for disconnect switches are specified in IEEE C37.32 (2002), for circuit breakers in IEEE C37.04a (2003), and for transformer bushings in IEEE C57.19.01 (2000) and IEEE C57.19.100 (1995). Moment capacities are typically not provided by these documents. When it is determined that the terminal pads may be subjected to a significant moment load, the design engineer should consult with the equipment manufacturer for the limiting terminal pad moment capacity.

It is recommended that the following information be included in specifications for purchasing electrical equipment:

- the type of connectors (flexible or rigid bus) to the electrical equipment,
- drawings of connection hardware, if available,
- preferred orientation of terminal pads, and
- unfactored design forces and moments that act on the terminal pads.

3.1.4 Wire Tension Loads

Structures supporting wire (conductor and shield wire) into and out of a substation are called substation dead-end structures. The dead-end structure wire tensions can be based on the transmission line wire tensions or a specified reduced tension, called slack-span tension. The use of a slack-span tension (as an example, 40%–60% of the transmission line tension) is to take advantage of a standard dead-end structure that is not dependent on the transmission line wire tension parameters. When a dead-end slack-span structure is used, the adjacent transmission line tower has to be a full transmission line tension dead-end structure. Dead-end structures that support wires that extend outside the substation yard should meet the requirements of the NESC (2007).

The strain bus conductor is composed of flexible conductors suspended above the ground between supporting structures using strain insulators. The span length is typically 300 ft (91 m) or less. Most stringing programs assume that the conductor spans the entire length between support structures, has the shape of a catenary or parabolic curve, and neglects the effect of insulators on the conductor sag.

Ice or wind loads applied transverse to the wire, or low temperatures increase the tension load in the wire beyond its original installation tension and should be considered when computing the wire tension load. These effects are discussed further in Sections 3.1.5 and 3.1.6.

With typical strain bus spans, the porcelain insulator weight may create a substantial portion of the sag and should not be neglected. Also, some older stringing programs assume that the conductor loading is uniform and do not have provisions for concentrated loads. Concentrated loads are typically created by taps (vertical jumpers) from the strain bus conductor to a lower bus conductor or electrical equipment. Depending on the tap location and magnitude, they may significantly affect the tension and sag of the conductor. A sag tension program that can model these conditions should be used for strain bus design.

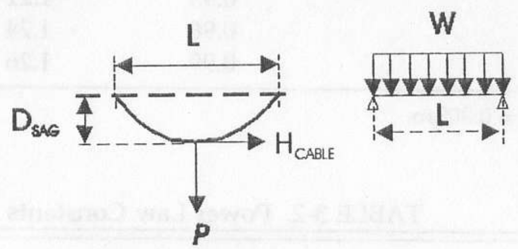
Taps should have adequate slack to allow unrestrained upward movement of the overhead conductor during cold temperatures. Otherwise, the tension in the conductor may become larger than the design tension on the structure, or the tap may transmit loads to the lower bus conductor or equipment that is not designed to resist this additional load. For medium to long strain bus spans with taps, the use of springs inline with the strain bus conductor can reduce the effects of variations in sag caused by temperature and loading conditions. Finite element programs can model the effects of insulators and concentrated tap loads.

3.1.4.1 Concentrated Loads on Strain Bus Systems. Taps create vertical concentrated loads on the strain bus conductors that can significantly increase wire tensions. Some computer programs that calculate wire tensions and sags use uniform wire loads and do not have provisions for concentrated loads. Dividing the concentrated load by the span length and adding it to the uniform wire load may underestimate the wire tension, particularly if the taps are located at midspan. A structural program with cable elements can properly calculate the wire tension for the concentrated loads. If this type of program is not available, the effects of concentrated loads can be approximated using the general cable theorem (Fig. 3-1).

The general cable theorem (Norris and Wilbur 1960) states, "At any point on a cable acted upon by vertical loads, the product of the horizontal component of cable tension and vertical distance from that point to the cable chord equals the bending moment which would occur at that section, if the loads carried by the cable were acting on a pin-supported beam of the same span length as that of the cable." This theorem is applicable to any set of vertical loads, whether the cable chord is horizontal or inclined.

For a concentrated load at midspan, the bending moment is $PL/4$ (where P is the concentrated load and L is the horizontal span length) and is equal to the product of the horizontal tension and the sag at midspan (Fig. 3-1). For a uniform load, the bending moment is $WL^2/8$, and it is also equal to the product of the horizontal tension and the sag at midspan. Therefore, $PL/4$ may be set equal to $WL^2/8$ and solved for W, which is the equivalent uniform load increase to account for the concentrated load. The equivalent uniform load, $W_{\text{concentrated load}}$, is then added to the conductor uniform load to calculate the tension and sag of the conductor. However, this example is only valid for concentrated loads that create the maximum bending moment at midspan.

3.1.4.2 Substation Dead-End Structures. The application of wire tension loads for dead-end structures should be chosen to allow for flexibility in routing transmission lines into and out of substations. This



$$M_{max} = D_{SAG} \times H_{CABLE} = WL^2/8 = PL/4$$

$$W_{CONCENTRATED\ LOAD} = (P)(2)/L$$

FIGURE 3-1. General Cable Theorem (Norris and Wilbur 1960).

flexibility can be accounted for by selecting a minimum line angle (such as 15 degrees), reasonable wind span for transverse loads, adequate low-point distances for vertical loads, and reasonable wire tensions for longitudinal loads. These loads act in either negative or positive directions.

Uplift loads can be applied to the substation dead-end structure when the wire attachment points of the first transmission line structure are higher in elevation than the attachment points of the substation dead-end structure. Design uplift loads can be specified for the dead-end structures. Normally, the uplift capacity of the members and connections is sufficient, assuming the vertical wire loads can act either up or down and additional uplift load cases are not necessary.

3.1.5 Extreme Wind Loads

Wind loads on substation structures, equipment, and conductors (bus and wire) should be applied in the direction that generates the maximum loading. For substation structures supporting wire loads, the longitudinal winds (in the direction of the wires) may also produce significant structure loading and should be considered in the load calculation.

The wind force can be determined using the following formula:

$$F = Qk_zV^2I_{FW}G_{RF}C_fA \qquad \text{(Eq. 3-1)}$$

where

F = wind force in the direction of wind (lb, N);
Q = air density factor, default value = 0.00256 (0.613SI), defined in Section 3.1.5.1;
k_z = terrain exposure coefficient, defined in Section 3.1.5.2;
V = basic wind speed, 3-s gust wind speed (mile/hour, m/s), defined in Section 3.1.5.3;
I_{FW} = importance factor, defined in Section 3.1.5.4;
G_{RF} = gust response factor (for structure and wire), defined in Section 3.1.5.5;
C_f = force coefficient, defined in Section 3.1.5.6; and
A = projected wind surface area normal to the direction of wind (ft^2, m^2).

The wind force calculated from Eq. 3-1 is based on the selection of appropriate values of wind speed, exposure coefficient, gust response factor, and force coefficient. These parameters are discussed in subsequent sections. For structures supporting wires, the wire tension corresponding to the wind loading should be calculated using the temperature that is most likely to occur at the time of the extreme wind loading events.

The wind loads recommended in this section are primarily based on the provisions of ASCE 7 (2005) and ASCE 74 (1991).

3.1.5.1 Air Density Factor. The air density factor, Q, converts the kinetic energy of moving air into the potential energy of pressure. For the standard atmosphere, the air density factor is 0.00256 (or 0.613 SI). The standard atmosphere is defined as sea level pressure of 29.92 in. of mercury (101.32 kPa) with a temperature of 59 °F (15 °C). The wind speed, V, in Eq. 3-1 should be expressed in terms of miles per hour when the constant 0.00256 is used and 0.613 when meters per second are used.

The numerical constant of 0.00256 should be used except where sufficient weather data are available to justify a different value. The air density factor varies as a function of altitude and temperature. Variations of the air density for other air temperatures and elevations that are different from the standard atmosphere are given in ASCE 74 (1991).

3.1.5.2 Terrain Exposure Coefficient. The exposure coefficient, k_z, modifies the basic wind speed to account for both terrain and height effects. Wind speed varies with height because of ground friction, and the amount of friction varies with ground roughness. The ground roughness is characterized by the various exposure categories described below.

3.1.5.2.1 Exposure Categories. Three exposure categories, B, C, and D, are recommended for use in this manual:

- **Exposure B.** This exposure is classified as urban and suburban areas, well-wooded areas, or terrain with numerous closely spaced obstructions of the size of single-family dwellings or larger. Use of this exposure category should be limited to those areas for which terrain representative of Exposure B prevails in the upwind direction for a distance of at least 2600 ft (792 m) or 20 times the height of the structure, whichever is greater.
- **Exposure C.** This exposure is classified as open terrain with scattered obstructions generally less than 33 ft (10 m) high. This category includes flat open country, grasslands, and shorelines in hurricane-prone regions.
- **Exposure D.** This exposure is classified as flat, unobstructed coastal areas exposed directly to wind flowing over open water (excluding shorelines in hurricane-prone regions) for longer than 5000 ft (1524 m) or 20 times the height of the structure, whichever is greater. Shorelines in exposure D include inland waterways, the Great Lakes, and coastal areas of California, Oregon, Washington, and Alaska. This exposure should apply to those structures exposed to the wind coming from over the water. Exposure D extends inland from the shoreline a distance of 600 ft (200 m) or 20 times the height of the structure, whichever is greater.

Values of the terrain exposure coefficient k_z are listed in Table 3-1 for heights up to 100 ft (30.5 m) above ground. Values for k_z not shown in Table 3-1, and for heights greater than 100 ft (30.5 m), can be determined using Eq. 3-2.

$$k_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha} \quad \text{for } 15 \le z \le z_g \qquad \text{(Eq. 3-2)}$$

where z = the effective height at which the wind is being evaluated, z_g = gradient height (Table 3-2), and α = the power law coefficient for a 3-s gust wind (Table 3-2).

Effects of terrain on the wind force are significant. The appropriate exposure category must be selected after careful review of the surrounding

TABLE 3-1. Terrain Exposure Coefficient, k_z

Height above Ground z (feet)	k_z Exposure B	k_z Exposure C	k_z Exposure D
0–15	0.57	0.85	1.03
30	0.70	0.98	1.16
40	0.76	1.04	1.22
50	0.81	1.09	1.27
60	0.85	1.13	1.31
70	0.89	1.17	1.34
80	0.93	1.21	1.38
90	0.96	1.24	1.40
100	0.99	1.26	1.43

Note: 1 foot = 0.305 m.

TABLE 3-2. Power Law Constants

Exposure Category	α Power Law Coefficient for a 3-s Gust Wind	z_g Gradient Height in feet (m)
B	7.0	1200 (366)
C	9.5	900 (274)
D	11.5	700 (213)

terrain. Exposure C should be used unless the design engineer has determined with good engineering judgment that other exposures are more appropriate. The power law constants (α and z_g) for the different terrain categories are shown in Table 3-2.

3.1.5.2.2 ***Effective Height.*** The effective height, z, is used for selection of the terrain exposure coefficient, k_z, the wire gust response factor, G_{WRF}, and the structure gust response factor, G_{SRF}. Sections 3.1.5.5.1 and 3.1.5.5.2 define the location for the effective height for the structure and wire.

3.1.5.3 Basic Wind Speed. The basic wind speed, used in Eq. 3-1, is the 3-second gust wind speed at 33 ft (10 m) above ground in a flat and open country terrain (Exposure Category C) and associated with a 50-year return period. ASCE 7 (2005) basic wind speed maps are shown in Figs. 3-2a and 3-2b.

In certain regions in the country, such as mountainous terrain, topographical characteristics may cause significant variations of wind speed over short distances (ASCE 7 2005). In these regions, local meteorological data should be collected to establish the design wind speed (Abild et al. 1992; Hoskings and Wallis 1987; Peterka 1992; Wang 1991). General guidelines of developing local wind data are recommended in ASCE 74 (1991).

Hurricane wind speeds associated with the 50-year return period are incorporated in the design wind speed map of ASCE 7 (2005). Additional hurricane wind speed maps are available in ASCE 7 (2005) for the western and eastern Gulf of Mexico coastlines and the mid- and Northern Atlantic coastlines.

3.1.5.4 Importance Factor for Basic Wind Speed. The return period, also called the mean recurrence interval, is approximately the reciprocal of the annual probability of occurrence. A 50-year mean recurrence interval (return period) signifies approximately a 2% annual probability of wind loading that will exceed or equal the design value (importance factor, $I_{FW} = 1.0$). For substation structures that require a higher level of reliability, a 100-year mean recurrence interval may be desirable. The importance factors and their relationship to the return period are listed in Table 3-3.

The selection of the importance factor provides a method of adjusting the level of structural reliability. The use of an importance factor equal to 1.0 does not imply that the structures are not important. Rather, it represents a good understanding of the probabilities of failure and required structural reliability. It is the owner's responsibility to select the appropriate importance factor for their substation structures.

3.1.5.5 Gust Response Factor. The gust response factor, G_{RF}, accounts for the dynamic effects of gusts on the wind response of structures. The gust response factors provided in this manual are based on ASCE 7 (2005) and Davenport's (1979) wind load model.

The Davenport equations for gust response factors were originally developed based on 10-min. average wind speed. To convert these equations to 3-s gust wind, a constant, k_v, was introduced to account for the ratio of 3-s gust wind speed to 10-min. average wind speed in open country (Exposure C) at the 33-ft (10 m) reference height. This ratio (k_v) is equal to 1.43, based on Durst's conversion (ASCE 7 2005, Fig. C6-4).

Other theories have been developed (Deaves and Harris 1978) to address issues involving wind speed conversion. They often yield different results. This is particularly true for Exposures B and D, where only limited historical data are available under extreme wind events. With all the uncertainties related to wind and wind measurements, it is recommended that a single numerical constant for the value of k_v under all exposure categories be used. The 1.43 was developed for terrain exposure category C, but because at this time there are no equivalent values for categories B and D, it was determined that the k_v value of 1.43 will provide acceptable gust response factors for application to substation structures.

3.1.5.5.1 ***Structure Gust Response Factor.*** The structure gust response factor, G_{SRF}, is used for computing the wind loads acting on substation structures and on the insulator and hardware assemblies attached to the structures. This factor accounts for the response of the substation structure to the gust wind.

The structure gust response factor, G_{SRF}, for equipment support structures is based on ASCE 7 (2005) for rigid structures. Rigid, nonwire-supporting structures for wind response are defined as structures with a fundamental frequency of 1 Hz or greater. For these structures, the structure gust response factor, G_{SRF}, can be assumed as a constant value of 0.85.

The structure gust response factors, G_{SRF}, for wire-supporting structures (dead-end and line termination structures) and flexible, nonwire-supporting structures (with fundamental frequency less than 1 Hz) are based on Davenport's (1979) wind load model. The gust response factors used in this manual for these structure types do not include the dynamic resonant response of the structure or wires. The dynamic response of the wires and structures to wind gusts may result in amplification of the wind loads that tend to offset any spatial reduction. With flexible structures, this amplification factor may need to be included as shown in the Davenport equations (1979).

The wire-supporting structure gust response factor can be calculated using Eq. 3-3. The above-ground height (h) of the substation structure should be used in this equation. A factor of 0.67 is included in Eq. 3-3 (E_s)

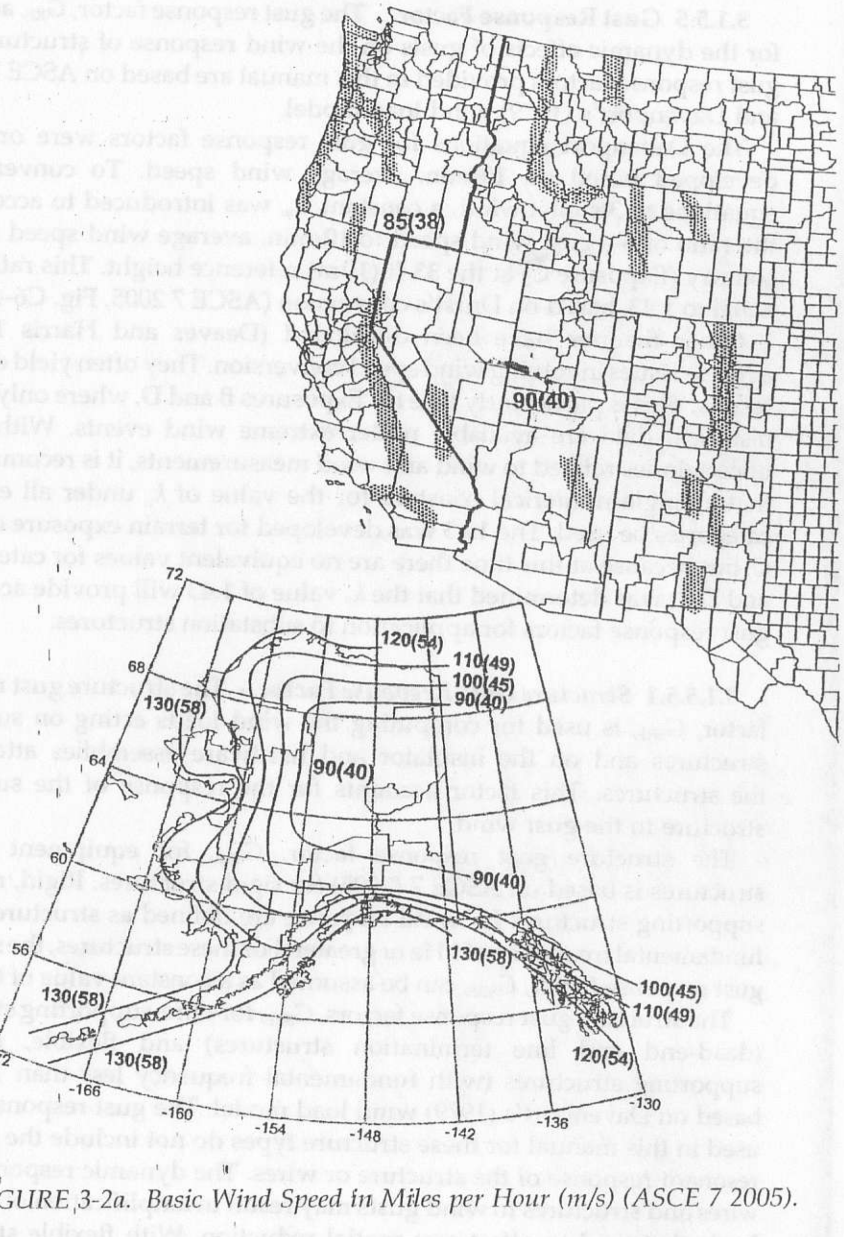
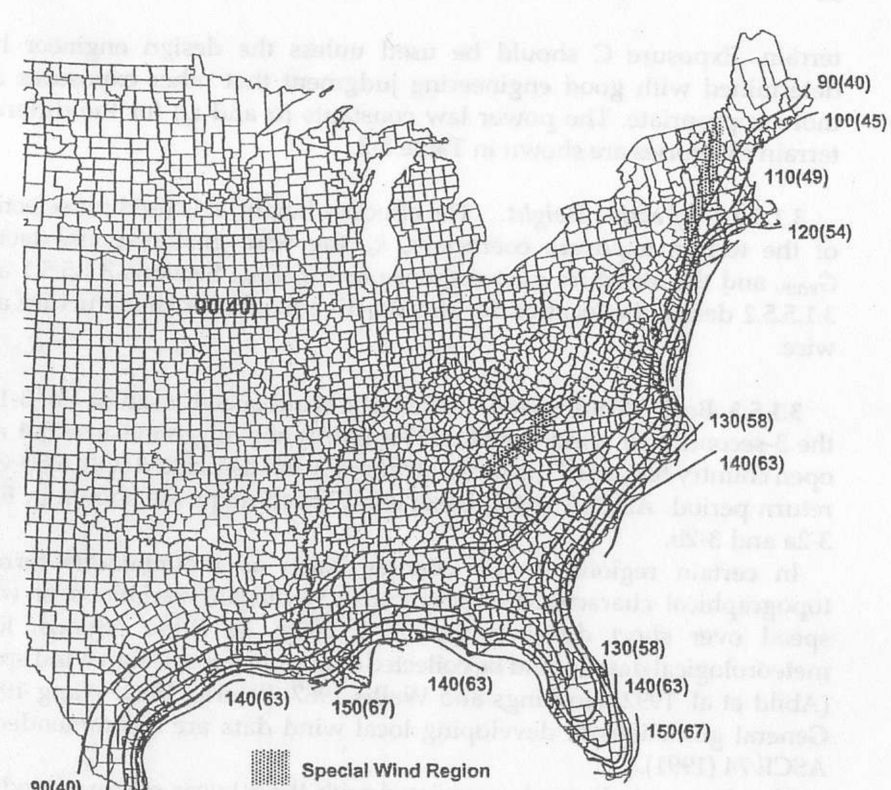


FIGURE 3-2a. Basic Wind Speed in Miles per Hour (m/s) (ASCE 7 2005).



Location	V mph	(m/s)
Hawaii	105	(47)
Puerto Rico	145	(65)
Guam	170	(76)
Virgin Islands	145	(65)
American Samoa	125	(56)

Notes:
1. **Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10 m) above ground for Exposure C category.**
2. **Linear interpolation between wind contours is permitted.**
3. **Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.**
4. **Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.**

FIGURE 3-2b. Basic Wind Speed in Miles per Hour (m/s) (ASCE 7 2005).

TABLE 3-3. Importance Factors (I_{FW}) for Basic Wind Speed

Mean Recurrence Interval	50 Years	100 Years
Importance factor	1.00	1.15

to simplify the calculation by assuming the average wind force is equal to the force calculated at 2/3 of the structure height, defined as the structure effective height.

$$G_{SRF} = \left(1 + 3.6(\varepsilon) E_S (B_S)^{0.5}\right) / k_v^2 \quad \text{(Eq. 3-3)}$$

where

E_S = $4.9\,(k_{10})^{1/2}\,(33/(0.67)(h))^{1/\alpha_{10}}$;

ε = 0.75 for wire-supporting structures (dead-end and line termination structures) (Table 3-4a);

ε = 1.00 for flexible structures (nonwire-supporting, <1 Hz) (Table 3-4b);

TABLE 3-4a. Structure Response Factor, G_{SRF}, for Wire-Supporting Structures, $\varepsilon = 0.75$

Height (ft)	Exposure B	Exposure C	Exposure D
≤33	1.17	0.96	0.85
>33 to 40	1.15	0.95	0.84
>40 to 50	1.12	0.94	0.84
>50 to 60	1.08	0.92	0.83
>60 to 70	1.06	0.91	0.82
>70 to 80	1.03	0.89	0.81
>80 to 90	1.01	0.88	0.81
>90 to 100	1.00	0.88	0.80

Notes: Equation 3-3 can be used to calculate G_{SRF} values. 1 foot = 0.305 m.

TABLE 3-4b. Structure Response Factor, G_{SRF}, for Flexible Structures, Nonwire-Supporting, <1 Hz, $\varepsilon = 1.0$

Height (ft)	Exposure B	Exposure C	Exposure D
≤15	1.59	1.20	1.02
>15 to 33	1.48	1.15	0.99
>33 to 40	1.37	1.11	0.96
>40 to 50	1.33	1.08	0.95
>50 to 60	1.28	1.06	0.94
>60 to 70	1.25	1.05	0.93
>70 to 80	1.22	1.03	0.92
>80 to 90	1.19	1.02	0.91
>90 to 100	1.17	1.00	0.90

Notes: Equation 3-3 can be used to calculate G_{SRF} values. 1 foot = 0.305 m.

B_S = $1/(1 + 0.375(h/L_{10}))$;

h = total above-ground height of the structure in feet;

k_v = 1.43;

L_{10} = turbulence scale (Table 3-5);

k_{10} = surface drag coefficient (Table 3-5); and

α_{10} = power law coefficient (10-min. average wind) (Table 3-5).

Equation 3-3 was developed from the Davenport (1979) equation by neglecting the resonant response terms for the structure and converting to a 3-s gust wind speed. This simplified approach is applicable for most practical wire-supporting substation structure types (dead-end and line termination structures). Tables 3-4 give the structure gust response factor for terrain exposure categories B, C, and D. The values in these tables were determined using a k_v of 1.43 for all terrain exposure categories. The G_{SRF} values for the heights at ≤15 ft and ≤33 ft are determined at the heights of 15 ft and 33 ft. The G_{SRF} values for the table rows with height ranges (e.g., >33 to 40 ft) are based on the average G_{SRF} value within the specified range.

3.1.5.5.2 Wire and Strain Bus Gust Response Factor. Substation structures supporting overhead wire and strain bus (both will be referred to as wire) should include a wire gust response factor (G_{WRF}) for determining the wind loads acting on the wire. This factor accounts for the response of the wire system to the gust wind.

The G_{WRF} is a function of the exposure category, design wind span between structures, and the effective height. The wire gust response factor can be calculated using Eq. 3-4. In Eq. 3-4, the effective height for an overhead wire is the height above the ground to the wire attachment point.

$$G_{WRF} = \left(1 + 2.7 E_W (B_W)^{0.5}\right) / k_v^2 \quad \text{(Eq. 3-4)}$$

TABLE 3-5. Exposure Category Constants for G_{SRF} and G_{WRF} Determination

Exposure Category	Surface Drag Coefficient k_{10}	Power Law Coefficient α_{10} (10-min average)	Turbulence Scale L_{10} (ft)
B	0.010	4.5	170
C	0.005	7	220
D	0.003	10	250

Note: 1 foot = 0.305 m.

where

$E_W = 4.9\,(k_{10})^{1/2}\,(33/h)^{1/\alpha_{10}}$;
$B_W = 1/(1 + (0.8L/L_{10}))$;
H = wire and strain bus effective height (ft);
k_v = 1.43;
L = wire horizontal span length (ft);
L_{10} = turbulence scale (Table 3-5);
k_{10} = surface drag coefficient (Table 3-5); and
α_{10} = power law coefficient (10-min. average wind) (Table 3-5).

Equation 3-4 was developed from the Davenport (1979) equation by neglecting the wire resonant response terms and converting to a 3-s gust wind speed. This simplified approach is applicable for most practical wire installations supported on substation structures. Tables 3-6a, 3-6b, and 3-6c give the wire gust response factor for terrain exposure categories B, C, and D. Equation 3-4, with the appropriate exposure category constants shown in Table 3-5, can be used to obtain the G_{WRF} for exposure categories B, C, and D for heights over 100 ft (30.5 m) and wind spans equal to or greater than 750 ft (229 m). The G_{WRF} values for the heights at ≤33 ft are determined at the average height between the ground level and 33 ft. The remaining G_{WRF} values are based on the lower range height value. A 50-ft span is used to calculate the average value for wire span (L) less than 100 ft. The lower wire span length within the given range is used to determine the G_{WRF} values.

The values in these tables were determined using a k_v of 1.43 for all terrain exposure categories. Davenport's equations (1979) for gust response factors were originally developed based on 10-min. average wind speed.

TABLE 3-6a. Wire Gust Response Factor, G_{WRF}, Exposure B

Wire Effective Height (ft)	Wire Span, L (ft): $L < 100$	$100 \le L < 250$	$250 \le L < 500$	$500 \le L < 750$
≤33	1.17	1.11	1.00	0.90
>33 to 40	1.07	1.02	0.93	0.84
>40 to 50	1.05	1.00	0.91	0.83
>50 to 60	1.02	0.98	0.89	0.81
>60 to 70	1.00	0.96	0.87	0.80
>70 to 80	0.98	0.94	0.86	0.79
>80 to 90	0.97	0.93	0.85	0.78
>90 to 100	0.95	0.92	0.84	0.77

Notes: Equation 3-4 should be used to calculate G_{WRF} for wire/strain bus effective height over 100 ft or spans 750 ft and greater. 1 foot = 0.305 m.

TABLE 3-6b. Wire Gust Response Factor, G_{WRF}, Exposure C

Wire Effective Height (ft)	Wire Span, L (ft): $L < 100$	$100 \le L < 250$	$250 \le L < 500$	$500 \le L < 750$
≤33	0.95	0.92	0.86	0.79
>33 to 40	0.91	0.88	0.82	0.76
>40 to 50	0.90	0.87	0.81	0.75
>50 to 60	0.89	0.86	0.80	0.75
>60 to 70	0.88	0.85	0.80	0.74
>70 to 80	0.87	0.84	0.79	0.73
>80 to 90	0.86	0.83	0.78	0.73
>90 to 100	0.85	0.83	0.78	0.73

Notes: Equation 3-4 should be used to calculate G_{WRF} for wire/strain bus effective height over 100 ft or spans 750 ft and greater. 1 foot = 0.305 m.

TABLE 3-6c. Wire Gust Response Factor, G_{WRF}, Exposure D

Wire Effective Height (ft)	Wire Span, L (ft): $L < 100$	$100 \le L < 250$	$250 \le L < 500$	$500 \le L < 750$
≤33	0.85	0.82	0.77	0.72
>33 to 40	0.82	0.80	0.75	0.71
>40 to 50	0.81	0.79	0.75	0.70
>50 to 60	0.80	0.78	0.74	0.70
>60 to 70	0.80	0.78	0.74	0.70
>70 to 80	0.79	0.78	0.73	0.69
>80 to 90	0.79	0.77	0.73	0.69
>90 to 100	0.79	0.77	0.73	0.69

Notes: Equation 3-4 should be used to calculate G_{WRF} for wire/strain bus effective height over 100 ft or spans 750 ft and greater. 1 foot = 0.305 m.

To convert these equations to 3-s gust wind, a constant, k_v, was introduced to account for the ratio of 3-s gust wind speed to 10-min. average wind speed in open country (exposure C) at the 33-ft (10-m) reference height. This ratio (k_v) is equal to 1.43, based on Durst's conversion (ASCE 7 2005, Fig. C6-4).

3.1.5.6 Force Coefficient. The force coefficient, C_f, in the wind force formula, Eq. 3-1, is the ratio of the resulting wind force per unit area in the direction of the wind to the applied wind pressure. This coefficient takes into account the effects of the structural member's wind characteristics: shape, size, orientation with respect to the wind, solidity, shielding, and surface roughness. It is also referred to as a drag coefficient, pressure coefficient, or shape factor.

The ratio of a member's length to its diameter (or width) is known as the aspect ratio. Shorter members have lower force coefficients than longer members of the same shape. The force coefficients given in the tables of this chapter are applicable to members with aspect ratios greater than 40. Adjustment factors for members with aspect ratios less than 40 may be used to determine a corrected C'_f that is substituted for C_f in Eq. 3-1, as follows (ASCE 74 1991):

$$C'_f = (c)(C_f) \qquad \text{(Eq. 3-5)}$$

where c is the correction factor from Table 3-7 and C_f is the force coefficient from Table 3-8 and Table 3-9.

The term *yawed wind* is used to describe winds whose angle of incidence with a structure is other than perpendicular. The maximum effective wind on square base lattice structures occurs at a yaw angle of slightly less than 45 degrees (Bayar 1986; BEAIRA 1935). These wind loads are typically 12% to 15% greater than the calculated perpendicular wind loads.

An important factor that influences the force coefficient of lattice truss structures is the solidity ratio of the frame. The force coefficient for the total structure depends on the airflow resistance of individual members and on the airflow pattern around the members. For lattice structures that are less than 200 ft (61 m) high, the solidity ratios for the various tower panels over the height of the transverse and longitudinal faces may be averaged to simplify the wind load calculation. Solidity ratio, Φ, is defined as the ratio of the area of all members in the windward face to the area of the outline of the windward face of a latticed structure.

When two members are placed in line with the wind, such as in a lattice structure, the leeward frame is partially shielded by the windward frame. The shielding factor is influenced by the solidity ratio, spacing between frames, and yaw angle.

Tables 3-8 and 3-9 give the force coefficients recommended by ASCE 74 (1991). Table 3-10 gives correction factors for converting the C_f values in Table 3-8 to C_f values for round-section members:

$$C_f(\text{round}) = C_f\,(\text{flat-sided members}) \times C_c\,(\text{correction factor})$$

These values are assembled from the latest boundary-layer wind-tunnel and full-scale tests and from previously available literature. Other force

TABLE 3-7. Aspect Ratio Correction Factors

Aspect Ratio	Correction Factor (c)
0–4	0.6
4–8	0.7
8–40	0.8
>40	1.0

TABLE 3-8. Force Coefficients, C_f, for Normal Wind on Latticed Structures with Flat-Sided Members

	Force Coefficient	
Solidity Ratio Φ	Square Cross-Sectional Structures	Triangular Cross-Sectional Structures
<0.025	4.00	3.6
0.025–0.44	4.1–5.2 Φ	3.7–4.5 Φ
0.45–0.69	1.8	1.7
0.70–1.00	1.3–0.7 Φ	1.0 + Φ

TABLE 3-9. Force Coefficients, C_f, for Structural Shapes, Bus, and Surfaces Commonly Used in Tubular Structures

Member Shape	Force Coefficient
Structural shapes (average value)	1.6
Bus: Rigid and flexible	1
Circular	0.9
Hexadecagonal (16-sided polygonal)	0.9
Dodecagonal (12-sided polygonal)	1
Octagonal (8-sided polygonal)	1.4
Hexagonal (6-sided polygonal)	1.4
Square or rectangle	2

TABLE 3-10. Correction Factor, C_c, for Normal Wind on Latticed Structures with Round-Section Members

Solidity Ratio Φ	Correction Factor C_c
<0.30	0.67
0.30–0.79	0.47 + 0.67 Φ
0.80–1.00	1

coefficients can be used where justified by experimental data. Additional background information on force coefficients can be found in ASCE 7 (2005) and ASCE 74 (1991).

3.1.5.7 Application of Wind Forces to the Structures. The wind forces determined by Eq. 3-1 using the recommended force coefficients of this manual have accounted for both the windward and leeward faces, including shielding. The wind forces calculated on a complete lattice truss system can be distributed to the panel points of the structure without further consideration.

Equipment or portions of a structure can be partially shielded from the wind by an adjacent item. This phenomenon is known as wind shading or wind shielding. A slight change in the wind direction usually eliminates or reduces the amount of wind shielding. For these reasons, the effects of wind shielding are normally not considered for substation structures.

Where the lattice truss systems in a structure, or individual tubular shaft members of an H-frame structure, are separated, the windward and leeward faces should be considered as each being individually exposed to the calculated wind force with appropriate force coefficients.

3.1.6 Combined Ice and Wind Loads

The most common ice load is glaze ice. The ice load maps shown in Figs. 3-3a through f were published in ASCE 7 (2005). These maps show uniform radial thickness of glaze ice with concurrent 3-s gust wind due to freezing rain at 33 ft (10 m) above the ground for a 50-year mean recurrence interval.

In certain geographical areas, other types of ice loads, such as rime or in-cloud ice, wet snow, and hoarfrost may become important in the design of substation structures. For information on nonglaze ice loads, meteorological and engineering studies can be conducted to properly account for nonglaze ice loads in design practice.

The design engineer is responsible for selecting from available data the most appropriate ice thickness to use for the location of the facility being designed. Utilities can conduct icing studies, with the assistance of a consulting meteorologist with ice expertise, to develop more accurate ice loading for substation sites in their service areas (Hoskings and Wallis 1987; Jones 1996a; Jones 1996b; Wang 1991).

To calculate ice loads to be used for the design of substation structures, glaze ice is assumed to weigh 57 lb/ft^3 (8.95 kN/m^3). Rime ice can accumulate to a much larger thickness than glaze ice, but its density is usually lower than glaze ice and may be assumed to weigh 40 lb/ft^3 (6.29 kN/m^3).

3.1.6.1 Ice-Sensitive Substation Structures. Designers do not need to consider ice loads when designing every structure or structural component. Considerations should be given to only ice-sensitive structures. In addition, ice loads may be applied to only selected components in ice-sensitive structures. For example, in dead-end structure design, the ice load on the conductor is included in design, but the ice load on the structure is often neglected. Ice-sensitive structures are structures for which the load effects from atmospheric icing control the design of part or all of the structural system. Typically in a substation, ice-sensitive structures include equipment and rigid bus conductors and supports.

3.1.6.2 Importance Factor. The return period, also called the mean recurrence interval, is approximately the reciprocal of the annual probability of occurrence. A 50-year mean recurrence interval signifies approximately a 2% annual probability of ice thickness that will exceed or equal the design ice thickness (importance factor, $I_{FI} = 1.0$). For substation structures that require a higher level of reliability, a higher recurrence interval, e.g., a 100-year mean recurrence interval, may be desirable. The ice thickness importance factors and their relationship to the return period are listed in Table 3-11. The 3-s gust wind speed used in combination with the ice obtained from Figs. 3-3a through f will have an importance factor of 1.0 for both the 50 and 100 mean recurrence intervals, per ASCE 7 (2005).

The selection of the ice thickness importance factor provides a method of adjusting the level of structural reliability. The use of an ice thickness importance factor equal to 1.0 does not imply that the structures are not important. Rather, it represents a good understanding of the probabilities of failure and required structural reliability. The owners must select the appropriate ice thickness importance factor for their substation structures.

3.1.7 Earthquake Loads

Earthquake loading is an environmental loading condition that, based on the specific site and substation structure characteristics, may govern design in certain regions. Designers should be aware of unusual soil

TABLE 3-11. Importance Factors for Combined Ice and Wind

Mean Recurrence Interval	50 Years	100 Years
Ice thickness importance factor, I_{FI}	1.00	1.25
Concurrent wind load importance factor, I_{FWI}	1.00	1.00

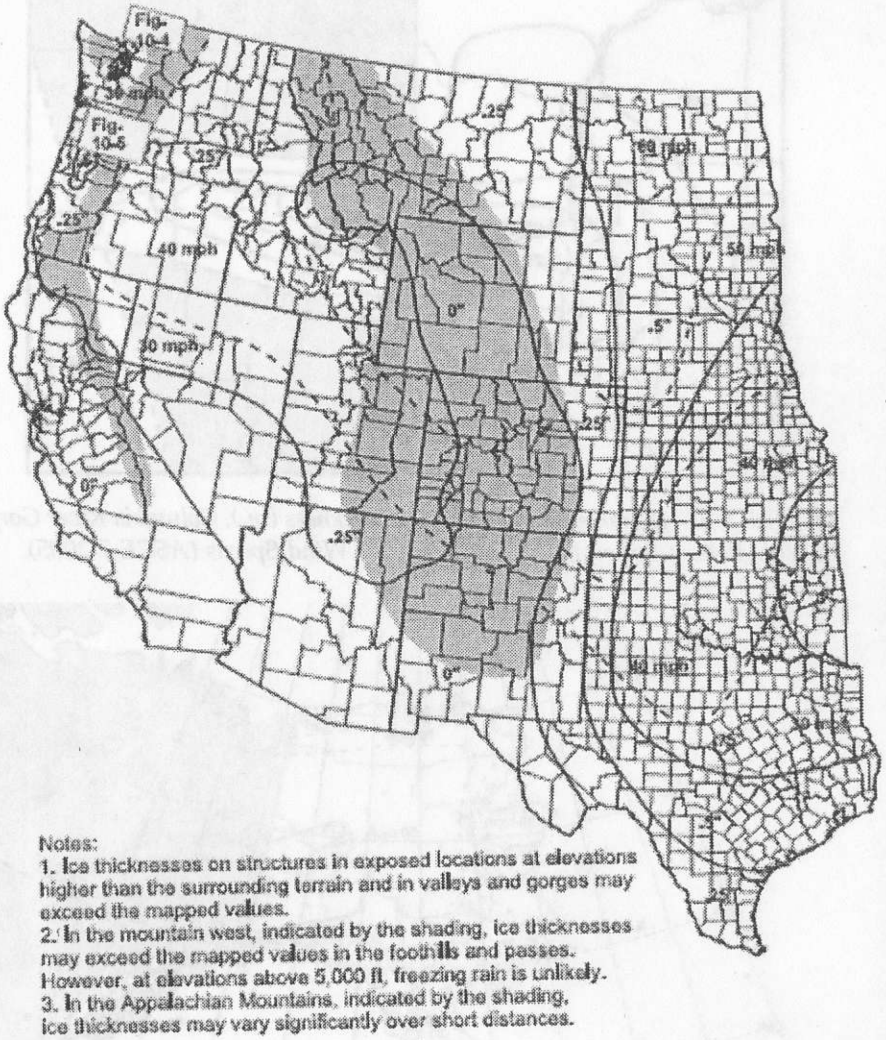


FIGURE 3-3a. *Extreme Radial Glaze Ice Thickness (in.), Western United States (except Pacific Northwest), 50-Year Return Period with Concurrent 3-s Wind Speeds (ASCE 7 2005).*

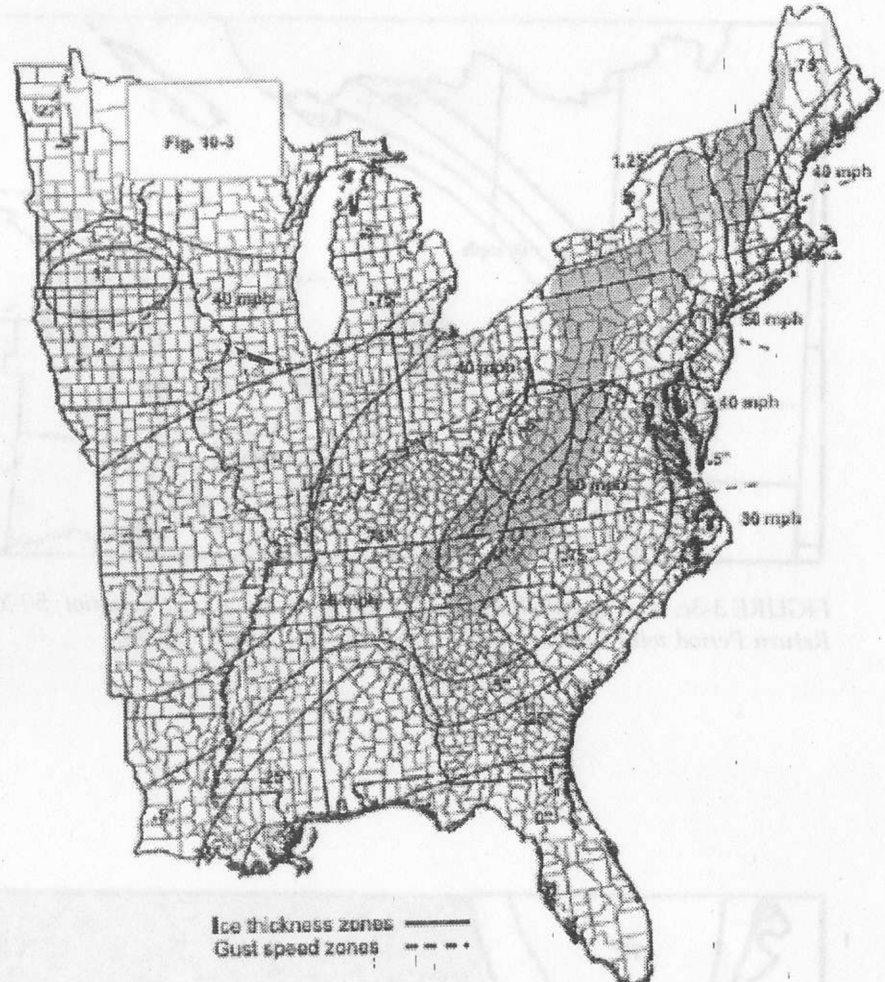


FIGURE 3-3b. *Extreme Radial Glaze Ice Thickness (in.), Eastern United States, 50-Year Return Period with Concurrent 3-s Wind Speeds (ASCE 7 2005).*

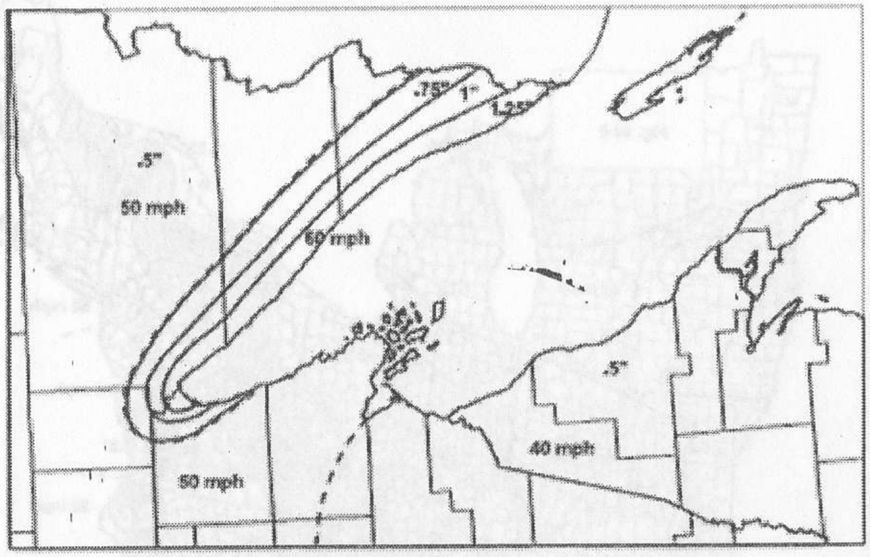


FIGURE 3-3c. Extreme Radial Glaze Ice Thickness (in.), Lake Superior, 50-Year Return Period with Concurrent 3-s Wind Speeds (ASCE 7 2005).

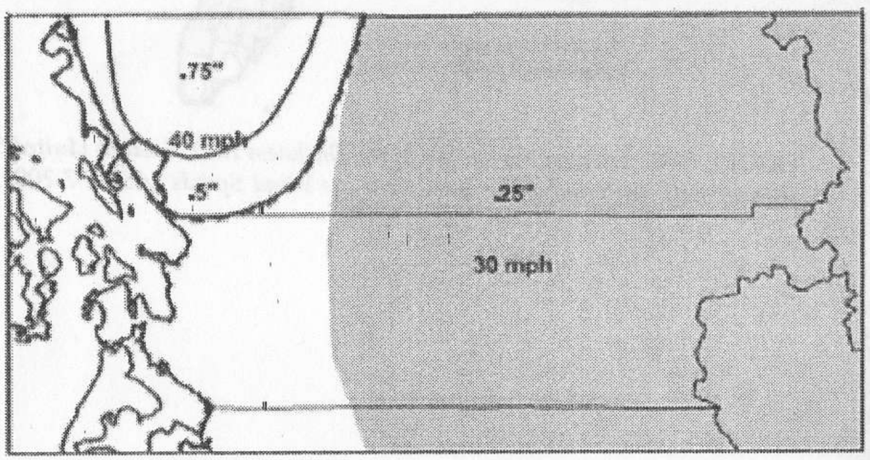


FIGURE 3-3d. Extreme Radial Glaze Ice Thickness (in.), Fraser Valley, 50-Year Return Period with Concurrent 3-s Wind Speeds (ASCE 7 2005).

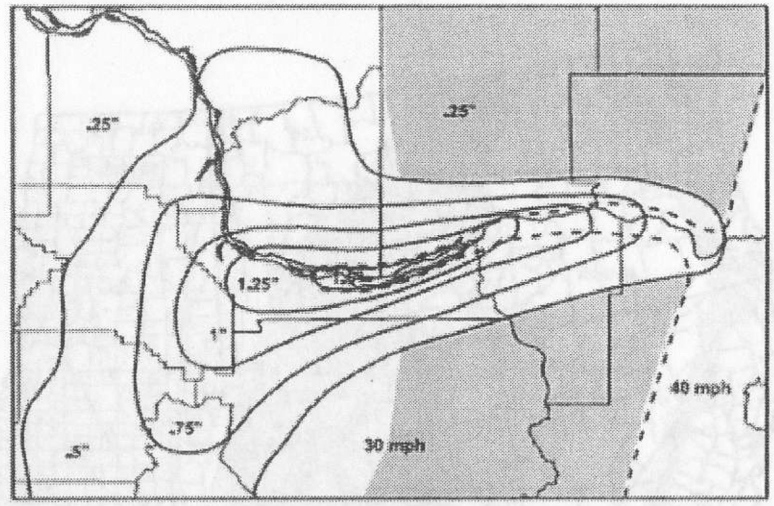


FIGURE 3-3e. Extreme Radial Glaze Ice Thickness (in.), Columbia River Gorge, 50-Year Return Period with Concurrent 3-s Wind Speeds (ASCE 7 2005).

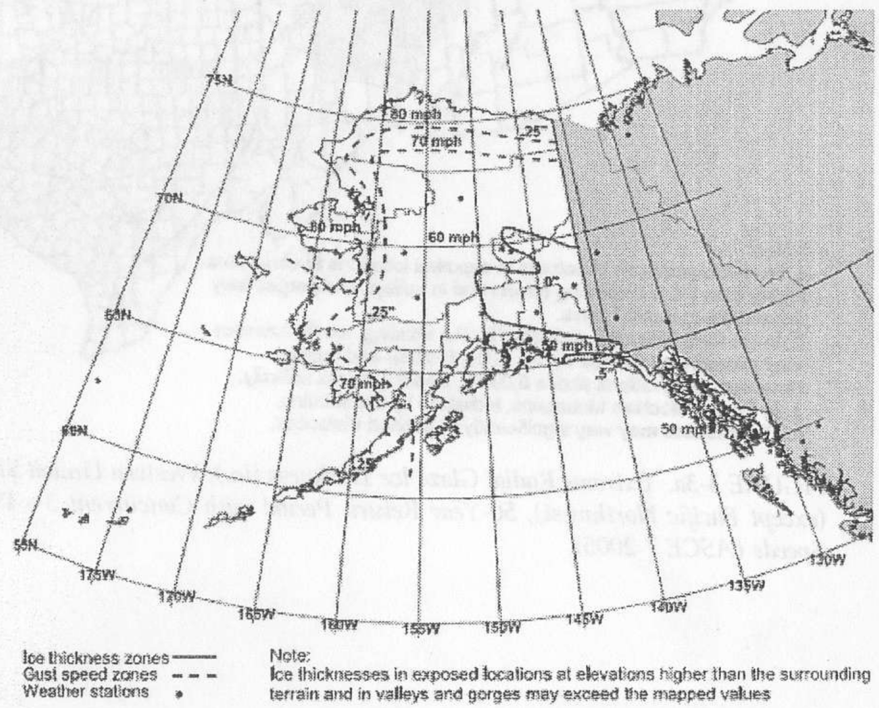


FIGURE 3-3f. Extreme Radial Glaze Ice Thickness (in.), Alaska, 50-Year Return Period with Concurrent 3-s Wind Speeds (ASCE 7 2005).

conditions, soil structure interaction, and the potential of modified response due to an intermediate structure.

Earthquake loading is not considered in combination with extreme wind or ice loading but may be considered in combination with operating or short-circuit loading, if these loads can occur during an earthquake. This manual assumes that the earthquake load case is applied during the condition of zero wind, no ice, and 60°F (15.6°C). The substation owner should determine if it is appropriate to combine earthquake loads with other load cases.

The vertical ground acceleration used in combination with the horizontal base shear should be 80% of the design horizontal ground acceleration. Friction forces due to gravity loads shall not be considered to provide resistance to seismic forces.

Electrical connections to equipment should have adequate slack in the connections between equipment. Guidance for the amount of electrical connection slack can be found in IEEE 693 (2005) and IEEE 1527 (2006).

The following simplified procedure for determining the seismic design load is based on the National Earthquake Hazard Reduction Program (NEHRP) Provisions (FEMA 450 2004).

Figure 3-4 shows the U.S Geological Survey (USGS) relative seismic hazard map. Spectral response acceleration maps obtained from the USGS website can be used to determine the design acceleration level.

The ground motion spectral response accelerations obtained from the maps are adjusted for design applications. The spectral response

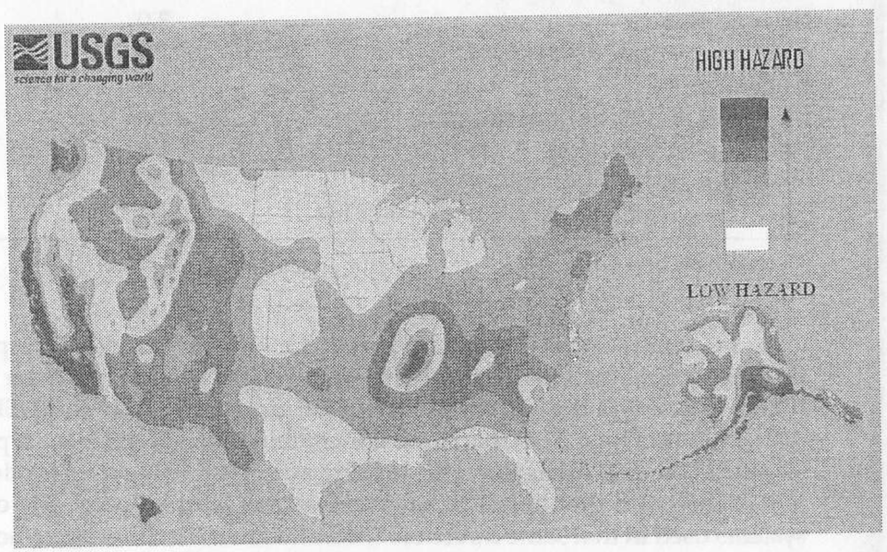


FIGURE 3-4. Relative Seismic Hazard Map (USGS).

acceleration obtained from the 0.2-s map is referred to as S_s (short periods) and the 1.0-s map values are referred to as S_1.

The mapped ground motion spectral response acceleration values (S_S and S_1) are adjusted for the following soil site classes:

A, hard rock,
B, rock,
C, very dense soil or soft rock,
D, stiff soil,
E, soft soil, and
F, very poor soil.

The different soil site classes are based on shear wave velocity, standard penetration resistance, and undrained shear strength data in the upper 100 ft (30 m) of the soil profile. These soil site characteristics are defined in FEMA 450 (2004). Using the soil site class and the S_s and S_1 values, the acceleration-based site coefficient, F_a (at a 0.2-s period) (Table 3-12), and the velocity-based site coefficient, F_v (at a 1.0-s period) (Table 3-13), are selected.

TABLE 3-12. Site Coefficient F_a

Site Class	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	[a]	[a]	[a]	[a]	[a]

[a]Site-specific geotechnical investigation and dynamic site response analyses are required.
Note: Use straight-line interpolation for intermediate values of S_S.

TABLE 3-13. Site Coefficient F_v

Site Class	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	[a]	[a]	[a]	[a]	[a]

[a]Site-specific geotechnical investigation and dynamic site response analyses are required.
Note: Use straight-line interpolation for intermediate values of S_1.

The soil site coefficients, F_a and F_v, and mapped ground motion spectral response accelerations, S_s and S_1, are used in Eqs. 3-6 and 3-7 to obtain the spectral response accelerations S_{DS} and S_{D1}.

$$S_{DS} = (2/3)(F_a)(S_s) \quad \text{(Eq. 3-6)}$$

$$S_{D1} = (2/3)(F_v)(S_1) \quad \text{(Eq. 3-7)}$$

The substation structure vibration period (in seconds), T, is used to adjust the 1-s spectral response acceleration to obtain the design spectral response acceleration, S_a. Equations 3-8 and 3-9 are used to determine the design spectral response acceleration S_a. The maximum value of S_a should be used to determine the seismic design force:

$$S_a = S_{DS} \quad \text{(Eq. 3-8)}$$

For substation structure periods $T > (S_{D1}/S_{DS})$, use Eq. 3-9:

$$S_a = S_{D1}/T \quad \text{(Eq. 3-9)}$$

3.1.7.1 Structure Earthquake Loads. An equivalent lateral force procedure may be used for calculation of the seismic design force, F_E, using Eq. 3-10:

$$F_E = (S_a/R)W(I_{FE})(I_{MV}) \quad \text{(Eq. 3-10)}$$

where
F_E is the seismic design force, lateral force applied at the center of gravity of the structure or component,
R is the structure–response modification factor (Section 3.1.7.3),
I_{FE} is the importance factor for earthquake loads (Section 3.1.7.2),
W is the dead load (including all rigidly attached equipment and 50% of the weight of attached wire),
S_a is the design spectral response acceleration (Eq. 3-8 or 3-9), and
I_{MV} is 1.0 for dominant single mode behavior or 1.5 when multiple vibration modes are considered by the designer.

Multiple vibration modes should be considered when the designer determines that they are significant to the structure response. The contribution of multiple modes can be accounted for by setting I_{MV} to 1.5.

The directions of application of the seismic design force should produce the most critical load effects. The combination of member stresses and

forces obtained from loads applied in orthogonal directions should be combined using the recommendations in Section 5.6.2.1 of Chapter 5.

3.1.7.2 Importance Factor. The importance factors, I_{FE}, recommended by this manual are:

Structures and equipment essential to operation	1.25
Anchorage for structures and equipment essential to operation	2.0
All other structures and equipment	1.0
All other anchorages	1.5

The selection of the appropriate importance factor (I_{FE}) is the responsibility of the design engineer. The importance factors, I_{FE}, specified in this section are the recommended values for I_p, used in IEEE 693 (2005) for foundation design.

3.1.7.3 Structure–Response Modification Factor. The structure–response modification factor, R, is based on the lateral force resisting system of the structure. Recommended substation structure R values are listed below:

Structure or Component Type	*USD*	*ASD*
Moment-resisting steel frame	3.0	4.0
Trussed tower	3.0	4.0
Cantilever support structures	2.0	2.7
Tubular pole	1.5	2.0
Steel and aluminum bus supports	2.0	2.7
Station post insulators	1.0	1.3
Rigid bus (aluminum and copper)	2.0	2.7
Structures with natural frequency >25 Hz	1.3	1.7

Note: See Section 6.7.1.

It is conservatively recommended to use a structure–response modification factor of 4 for ASD (allowable stress design) and 3 for USD (ultimate strength design) for most trussed structures. ASD and USD are defined in Section 6.2 of Chapter 6. For combinations of different types of structural systems along the same loading axis, the R value used for design in that direction should not be greater than the least value of any of the systems used in that same direction. Other R values for different structural systems and materials can be found in FEMA 450 (2004).

Use of *R* factors greater than 3 (USD) implies that inelastic energy dissipation mechanisms in the structure are permitted to develop. In order to count on those values of *R*, the structure should be designed and detailed in a manner that allows the development of such mechanisms before instabilities (e.g., buckling) or weaker nonductile failure modes occur.

3.1.8 Short-Circuit Loads

Short-circuit currents produce electromagnetic fields that cause forces on the bus conductors and potentially on the equipment. The bus conductors, insulators, and supports should be strong enough to resist these forces.

The forces imparted to the bus structure by short-circuit current depend on conductor spacing, magnitude of short-circuit fault current, type of short circuit, and degree of short-circuit asymmetry. Other factors to be considered are support flexibility and rigid bus configuration (e.g., corner and end effects).

The primary electrical control system can recognize a fault and open the circuit in 2–6 cycles (0.03–0.10 s) provided the circuit breaker and relays are operating properly. The backup control system normally takes 10–30 cycles (0.17–0.5 s) to open the circuit. If a fault interrupts in the 2–6 cycle range, the rigid bus conductors and supports may not have appreciable structural response to the fault. If the primary control system fails, the backup system is required to operate, and the rigid bus conductors and support system have sufficient time to structurally respond to the fault. The inertia of the rigid bus conductors and supports can be overcome, creating deflections and forces. Short-circuit loads should be considered in design.

Short-circuit forces can be calculated using the equations in IEEE 605 (2006). Based on fault current tests, IEEE 605 (2006) equations give conservative results. The electrical engineer should be responsible for determining the short-circuit fault current.

3.1.9 Construction and Maintenance Loads

During construction and maintenance operations, it is sometimes necessary for workers to be supported from a component of the structure or to impose forces on the structure while pulling or hoisting equipment. Occupational Safety and Health Administration (OSHA) requirements for worker safety should be reviewed. An IEEE standard applicable to substation structures, IEEE 1307 (2004) is available for determining minimum worker safety loads. Each structure should be evaluated for the likelihood of such loads. The design engineer should determine whether

other unusual loads will be applied during construction and maintenance operations, and if so, their magnitude. These loads are usually not combined with other extreme climatic loads because it is unlikely that workers will perform construction or maintenance during severe weather, and construction and maintenance loads are specified with higher load factors. Additional information on construction and maintenance considerations is discussed in Chapter 10.

3.1.10 Wind-Induced Oscillations

Relatively constant low-speed winds may occasionally produce an oscillating motion in structures. This phenomenon is called wind-induced vortex shedding. Wind-induced oscillations are typically unpredictable and are mitigated when they develop. The oscillating motion can become severe in structures, such as lightning masts, when no shield wires are attached. Damping devices (external or internal), such as a heavy steel chain encased in a rubber hose, suspended inside the pole can be effective in reducing such motions. Design considerations for wind-induced vortex shedding are discussed in Section 6.10.2 of Chapter 6.

3.1.11 Loading Criteria for Deflection Limitations

Where the substation owner has not developed specific loading conditions for deflection analysis, the load conditions from Sections 3.1.11.1, 3.1.11.2, and 3.1.11.3 may be used. A load factor of 1.0, applied to the dead weight, is used with the deflection load cases. The owner or design engineer should determine if additional loads (such as wire tension or earthquake) should be applied in combination with the recommended deflection load cases. The deflection limitations in Section 4.1 of Chapter 4 are to be used with the wind or ice and wind loads in this section.

3.1.11.1 Wind Load for Deflection Calculations. A 5-year mean recurrence interval peak gust wind speed should be used to calculate wind load associated with deflection criteria for substations located outside hurricane zones. Hurricane wind zone loads used to determine deflections should be determined by the owner. Table 3-14 provides wind load conversion factors that should be used in conjunction with the extreme wind load obtained from Eq. 3-1.

TABLE 3-14. Wind Deflection Load Conversion Factors

	5-Year Mean Recurrence
Wind deflection load conversion factor	0.78

TABLE 3-15. Ice Thickness Deflection Load Conversion Factors

	5-Year Mean Recurrence
Ice thickness deflection conversion factor	0.50
Wind deflection load conversion factor	1.00

3.1.11.2 Ice and Wind Combined Load for Deflection Calculations. A 5-year mean recurrence interval peak ice thickness should be used to calculate the ice load associated with deflection criteria. Table 3-15 provides ice load conversion factors that should be used in conjunction with extreme ice thickness obtained from Figs. 3-3a through 3-3f. The combined wind speed should be the value shown on Figs. 3-3a through 3-3f.

3.1.11.3 Other Deflection Considerations. If the electrical equipment is expected to operate during extreme winds, then the unfactored extreme wind should be used for deflection calculations. If the electrical equipment is expected to operate during extreme icing, then unfactored extreme icing loads should be used for deflection calculations.

Loads resulting from bus short circuits and earthquakes should not be considered in deflection analysis. Both loading conditions are short in duration, extreme events that require structural capability but not necessarily operational capability. Given the difficulty in predicting deflections under dynamic conditions and the limited need for equipment operation during the event, deflection analysis for these conditions would be difficult, imprecise, and of questionable use.

3.1.12 National Electrical Safety Code Loads

The National Electrical Safety Code (NESC 2007), Section 16, Paragraph 162.A, requires that substation structures supporting facilities (wires) that extend outside the substation fence should comply with the loading and strength sections of the NESC (2007). NESC (2007) is a minimum safety code and is not intended as a design specification.

3.1.13 State and Local Regulatory Loads

State and local regulatory loads, such as California's General Order 95 (2006), should be reviewed for application to substation structures and transmission lines entering a substation.

3.2 APPLICATION OF LOADS

The following are recommended guidelines regarding the application of the structure-loading criteria presented in this manual.

The following loading conditions should be considered for designing substation structures:

- NESC (2007) (and other state or local regulatory codes), Sections 3.1.12 and 3.1.13;
- extreme wind, Section 3.1.5;
- combined ice and wind, Section 3.1.6;
- earthquake, Section 3.1.7;
- short-circuit (combined with other load conditions when appropriate), Section 3.1.8;
- construction and maintenance, Section 3.1.9; and
- equipment operating loads, Section 3.1.2.

The following loading conditions should be considered for checking substation structure deflections:

- wind, Section 3.1.11.1;
- combined ice and wind, Section 3.1.11.2; and
- equipment operating loads, Sections 3.1.2 and 3.1.11.3.

Table 3-16 lists substation structure loading conditions that have the potential to control the design of the structure types listed. These load

TABLE 3-16. Basic Loading Conditions

Loading Conditions	Wire-Loaded Substation Structures	Switch and Interruption Supports	Rigid Bus Supports	Other Equipment Supports
NESC (2007)[a]	Y	N	N	N
Extreme wind	Y	Y	Y	Y
Combined ice and wind	Y	Y	Y	Y
Earthquake	Y	Y	Y	Y
Short circuit	N	Y	Y	N[b]
Construction and maintenance	Y	Y	Y	Y
Equipment operation	N	Y	N	Y
Deflection	Y	Y	Y	Y

[a]NESC (2007) or other state or local regulatory codes that may apply (i.e., General Order 95 (2006)).
[b]Short-circuit loads should be considered if the design engineer determines that this load effect is significant, such as rigid bus connected equipment.

conditions, as well as other important load conditions identified by the design engineer, should be selected based on the structure type, equipment, function, site location, and structural reliability required.

3.3 LOAD FACTORS AND COMBINATIONS

The methods for estimating loads, especially those for weather-related events, are mostly based on statistical models. These models, although scientifically correct, have limitations on the precision of their prediction. To ensure structural reliability, load factors are introduced to compensate for this uncertainty.

The load factors recommended in this section are selected based on the unique characteristic of typical electrical substation structures. The recommended load factors are different from those in other documents (such as ASCE 7 (2005), ACI 318 (2005) or AISC (2005a) that address loads mainly related to building-like structures.

Electrical transmission line grids are distributed systems with multiple redundancies. Typically, multiple looped paths are provided from the generation sources to the point of service. The operational reliability of the transmission line system is generally addressed by grid looping and extra redundancies. Thus, the reliability of the electrical grid typically does not rely on one individual structure. For this reason, load factors can be reduced without compromising the reliability of the system.

Unlike failures of building-type structures, the failure of a substation structure represents a low hazard to utility personnel. In fact, almost all substations are uninhabited. Thus, in the event of weather-related extreme load, electrical substations are usually unoccupied. For substation structures supporting wires that extend outside the substation fence, NESC (2007) provides requirements for public safety.

Substation structural configurations are simple enough that dead weight can easily be taken into account, unlike that for building-type structures. The nature of substation structures also prevents the likelihood of converting these structures into other functions or uses. Thus, design engineers can calculate the dead load with reasonable accuracy.

In the case of short-circuit loads, many theories and experiments have proven that the magnitude of this load event is typically much less than what has been predicted. The short-circuit loads are also short in duration. When combined with other extreme events, the structures most likely receive little effect from short-circuit loads.

TABLE 3-17. Ultimate Strength Design Cases and Load Factors

Load Cases	Load Factors and Combinations
Case 1	$1.1\,D + 1.2\,WI_{FW} + 0.75\,SC + 1.1\,T_W$
Case 2	$1.1\,D + 1.2\,I_W I_{FI}{}^a + 1.2\,W_I I_{FWI}{}^b + 0.75\,SC + 1.1\,T_W$
Case 3	$1.1\,D + 1.0\,SC + 1.1\,T_W$
Case 4	$1.1\,D + 1.25\,E$ (or E_{FS})$I_{FE} + 0.75\,SC + 1.1\,T_W$

[a]The importance factor for ice is applied to the thickness.
[b]The importance factor for wind with ice, I_{FIW}, is 1.0 with wind speeds from Figures 3-3a to f.
Notes: Replace load factor 1.1 on D with 0.9 for cases in which dead load is counted on to resist other applied loads. For allowable stress design, the load factors should equal 1.0.

This discussion addresses the parameters that influence the selection of the recommended load factors for substation structures presented in this manual. The load factors should be used with all material-based design specifications in conjunction with the nominal resistance factors.

Table 3-17 shows suggested design load cases, combinations, and minimum load factors to be use for substation structures. The individual load components are the following:

- D = structure and wire dead load;
- W = extreme wind load (F, from Eq. 3-1, without I_{FW});
- W_I = wind load in combination with ice;
- I_W = ice load in combination with wind;
- E = earthquake load (F_E, from Eq. 3-10, without I_{FW});
- E_{FS} = earthquake load reactions from first support imposed on the remainder of the structure (without I_{FE});
- T_W = horizontal wire tension for the appropriate wind and temperature condition;
- SC = short-circuit load; and
- I_F = importance factors (I_{FW}, I_{FI}, I_{FWI}, and I_{FE}).

The general format of the load case equations is the combination of load factors times the calculated loads, such as 1.1 × dead load. Load factors are used to account for the uncertainties in the estimation of the loads. A load factor of 1.0 does not imply that the structure is not structurally reliable.

Rather, it represents a good understanding of the uncertainty in the load determination.

A particular structure may not have all the individual load components listed in the load combination equations. The design engineer should determine whether a load case or load combination is appropriate. The combining of short-circuit loads with other loads (e.g., wind, ice, and earthquake) should be considered, and the owner should determine the level of short-circuit load used in combination with other loads. Table 3-17 does not imply that only these four load cases are adequate for the design of a substation structure. Variations of these or other load cases may be required to account for conditions, e.g., wind direction or short-circuit fault location.

3.4 ALTERNATE DESIGN LOADS AND LOAD FACTORS

Design loads, cases, combinations, and load factors other than those prescribed in this chapter should be substantiated by experimental or analytical investigations and by sound engineering judgment.

3.5 SERVICEABILITY CONSIDERATIONS

Serviceability is preserving the function, maintainability, durability, and appearance of substation structures and electrical equipment under normal usage. Serviceability should ensure that operational disruptions to the substation during normal, everyday use are rare and that operational disruption to the substation after extreme wind and ice loading events is kept to a minimum.

Types of excessive events that can affect substation structural behavior and may impair serviceability are the following:

1. local damage (e.g., yielding, buckling, or cracking) that may require excessive maintenance or lead to corrosion;
2. deflection or rotation that may affect structural appearance, structural function, or the operation of electrical equipment;
3. excessive vibration created by wind;
4. excessive temperature that may affect the strength of aluminum and copper; and
5. impact load created during an earthquake, when relative displacements between adjacent pieces of electrical equipment are not accommodated.

Factors of safety for allowable stress design (ASD) and load factors with strength reduction factors for ultimate strength design (USD) are used to control excessive local damage. Specifying limiting deflections and service loads controls excessive deflection and vibration. Limiting the operating temperature controls excessive temperatures. In the past, these guidelines have provided satisfactory structural performance, except for some cases of wind-induced vortex shedding vibration of hollow tubular members. These wind-induced vibrations are discussed in Sections 3.1.10 in this chapter and 6.10.2 in Chapter 6.

For structures supporting electrical equipment, the manufacturer's recommendations for service loads should be followed. Alternatively, the owner should consult or specify the anticipated loading requirement to the manufacturer.

For dead-end structures, rigid bus structures, and ground masts, serviceability criteria, in addition to deflection limits specified in Chapter 4, should consist of the following:

1. The minimum phase to ground clearance for conductors and bus systems should be maintained during high wind conditions. Flexible conductors can have significant horizontal movements in high wind conditions.
2. The minimum vertical electrical clearances should be maintained for conductors, bus systems, and overhead ground wires during ice conditions. Overhead ground wires are particularly susceptible to large vertical sags in ice conditions.
3. The minimum vertical clearances should be maintained under maximum operating temperature conditions for conductors and bus systems.
4. High-temperature operations may lead to aluminum material strength reductions.
5. Coating (e.g., galvanizing or paint) of structural components should be designed to protect the component from corrosion.
6. Provisions should be made to allow for the expansion and contraction of conductors, overhead ground wires, and rigid bus conductors from varying temperatures. The upward vertical movements of the conductors should not be restrained by taps. See Section 3.1.4 for additional information.

3.6 EXAMPLES OF APPLICATION OF LOAD CASES AND LOAD FACTORS

The following examples are provided to demonstrate basic concepts of calculating the loads presented within this manual. Not all the necessary calculations or the potential loads that could be considered are shown in these examples.

3.6.1 Example Load Cases for a Dead-End Structure (Fig. 3-5) and a 69-kV Disconnect Switch Support Structure (Fig. 3-6)

Load Case 1	Extreme wind 90 mph basic wind speed, 3-s gust, Exposure C 50-year mean recurrence Load factors from Table 3-17
Load Case 2	Combined ice and wind 40 mph wind and 1.0 in. of ice 50-year mean recurrence Load factors from Table 3-17
Load Case 3	Short circuit 15-kA symmetrical Load factors from Table 3-17
Load Case 4	Earthquake $S_S = 0.590$ and $S_1 = 0.186$ Soil profile D Load factors from Table 3-17
Other loads	Construction and maintenance loads None considered for this example See Section 3.1.9 Equipment operating loads None considered for this example See Section 3.1.2
NESC	Heavy, Grade B (dead-end structure) 4 lb/ft² wind and 0.5-in. radial ice Load factors from NESC

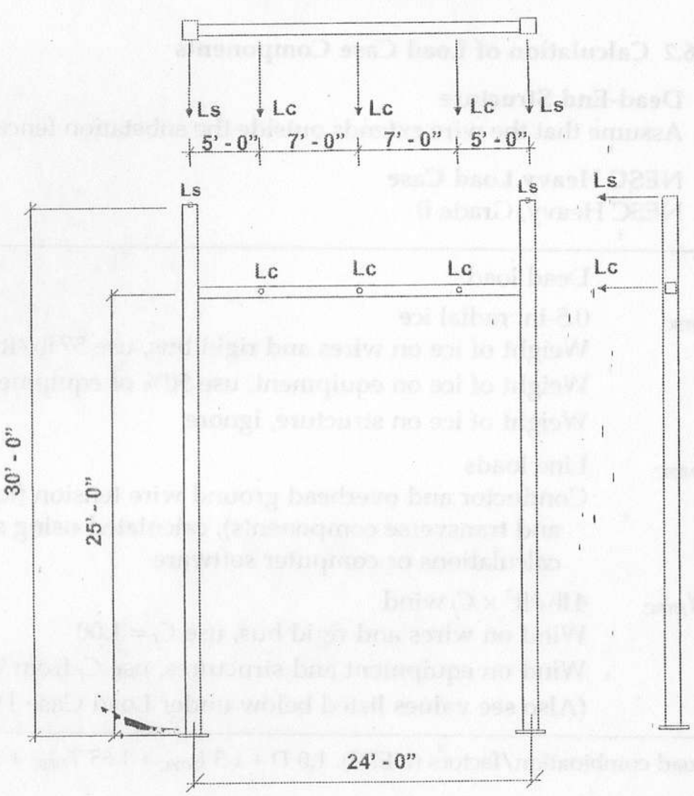


FIGURE 3-5. Dead-End Structure.

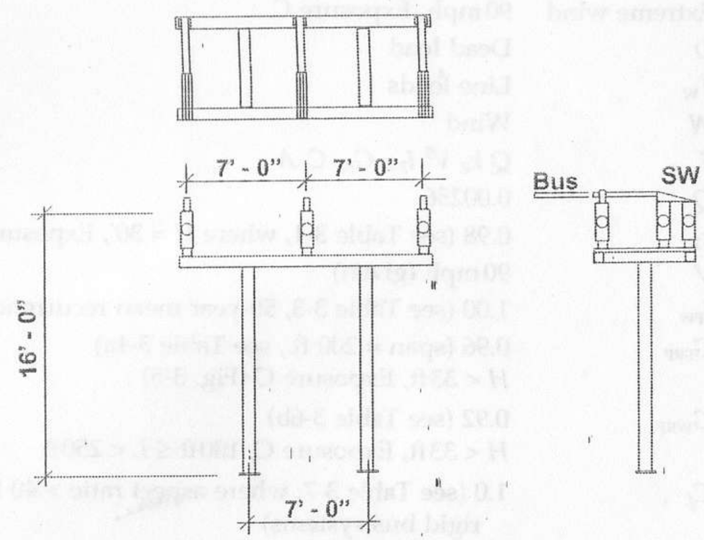


FIGURE 3-6. A 69-kV Disconnect Switch Support Structure.

3.6.2 Calculation of Load Case Components

Dead-End Structure

Assume that the wire extends outside the substation fence.

NESC Heavy Load Case

NESC Heavy, Grade B

D	Dead load
I_{NESC}	0.5-in. radial ice Weight of ice on wires and rigid bus, use 57 lb/ft^3 Weight of ice on equipment, use 50% of equipment weight Weight of ice on structure, ignore
T_{NESC}	Line loads Conductor and overhead ground wire tension (longitudinal and transverse components), calculated using sag-tension calculations or computer software
W_{NESC}	4 lb/ft^2 × C_f wind Wind on wires and rigid bus, use C_f = 1.00 Wind on equipment and structures, use C_f from Table 3-9 (Also see values listed below under Load Case 1)

Load combination/factors (NESC): $1.0\,D + 1.5\,I_{NESC} + 1.65\,T_{NESC} + 2.50\,W_{NESC}$

Extreme Wind Load Case 1

Extreme wind	90 mph, Exposure C
D	Dead load
T_W	Line loads
W	Wind
F	$Q\, k_Z\, V^2\, I_{FW}\, G_{RF}\, C_f\, A$
Q	0.00256
k_Z	0.98 (see Table 3-1, where H = 30′, Exposure C
V	90 mph (given)
I_{FW}	1.00 (see Table 3-3, 50-year mean recurrence interval)
G_{SRF}	0.96 (span = 200 ft., see Table 3-4a) H < 33 ft, Exposure C (Fig. 3-5)
G_{WRF}	0.92 (see Table 3-6b) H < 33 ft, Exposure C, 100 ft ≤ L < 250 ft
C_f	1.0 (see Table 3-7, where aspect ratio > 40 for wires and rigid bus systems)
C_f	0.9 (see Table 3-9 for circular equipment shapes)
C_f	2.0 (see Table 3-9 for square structural shapes)

Dead-End Structure

F	$= 0.00256 \times 0.98 \times 90^2 \times 1.00 \times 0.96 \times C_f \times A$ $= 19.5 \times C_f \times A$ $= 19.5 \times 1.0 \times A = 19.5\,\text{lb/ft}^2 \times A$ wires and rigid bus systems $= 19.5 \times 0.9 \times A = 17.6\,\text{lb/ft}^2 \times A$ circular equipment shapes $= 19.5 \times 2.0 \times A = 39.0\,\text{lb/ft}^2 \times A$ square structural shapes

I_{FW} = 1.0: Extreme wind importance factor (50-year mean recurrence).

69-kV Disconnect Switch Support Structure

Assume a rigid structure equipment support (Section 3.1.5.5.1, paragraph 2).

G_{SRF}	= 0.85
F	$= 0.00256 \times 0.98 \times 90^2 \times 1.00 \times 0.85 \times C_f \times A$ $= 17.27 \times C_f \times A$ $= 17.27 \times 1.0 \times A = 17.27\,\text{lb/ft}^2 \times A$ wires & rigid bus systems $= 17.27 \times 0.9 \times A = 15.55\,\text{lb/ft}^2 \times A$ circular equipment shapes (post insulators) $= 17.27 \times 2.0 \times A = 34.55\,\text{lb/ft}^2 \times A$ square structural shapes

I_{FW} = 1.0: Extreme wind importance factor (50-year mean recurrence).
Load combination/Factors: $1.1\,D + 1.2\,WI_{FW} + 0.75\,SC + 1.1\,T_W$ (from Table 3-17).

Combined Ice and Wind, Case 2

Combined wind and ice	40 mph wind + 1.00-in. radial ice
D	Dead load
T_W	Line loads
I_W	Ice Weight of ice on wires, use 57 lb/ft^3 Weight of ice on equipment, use 100% of equipment weight Weight of ice on structure, ignore
I_{FI}	1.0 (50-year mean recurrence)

Dead-End Structure

F	$= 0.00256 \times 0.98 \times 40^2 \times 1.00 \times 0.96 \times C_f \times A$
	$= 3.85 \times C_f \times A$
	$= 3.85 \times 1.0 \times A = 3.85\,\text{lb/ft}^2 \times A$ wires and rigid bus systems
	$= 3.85 \times 0.9 \times A = 3.47\,\text{lb/ft}^2 \times A$ circular shapes (post insulators)
	$= 3.85 \times 2.0 \times A = 7.70\,\text{lb/ft}^2 \times A$ square structural shapes

69-kV Disconnect Switch Support Structure

F	$= 0.00256 \times 0.98 \times 40^2 \times 1.00 \times 0.85 \times C_f \times A$
	$= 3.41 \times C_f \times A$
	$= 3.41 \times 1.0 \times A = 3.41\,\text{lb/ft}^2 \times A$ wires and rigid bus systems
	$= 3.41 \times 0.9 \times A = 3.07\,\text{lb/ft}^2 \times A$ circular equipment shapes
	$= 3.41 \times 2.0 \times A = 6.82\,\text{lb/ft}^2 \times A$ square structural shapes

Load combination/Factors: $1.1\,D + 1.2\,I_W\,I_{FI} + 1.2\,W_I I_{FW} + 0.75\,SC + 1.1\,T_W$ from Table 3-17.
$I_{FWI} = 1.0$.

Short-Circuit Load, Case 3

$F_{SC} = 3.596\gamma I_{SC}{}^2/(10^7\,D)$	IEEE 605 (2006), applicable equation and design parameters to be recommended by the electrical engineer
	$\gamma = 1.00$ phase–phase
	$I_{SC} = 15\,\text{kA} = 15{,}000\,\text{A}$
	$D = 7\,\text{ft}$
$F_{SC} = 3.596 \times 1 \times (15{,}000\,\text{A})^2/(10^7 \times 7\,\text{ft})$	
$F_{SC} = 11.6\,\text{lb/ft}$	

Load combination/Factors: $1.1\,D + 1.0\,SC + 1.1\,T_W$ from Table 3-17.

Earthquake Load, Case 4

$F_a = 1.33$	From Table 3-12, Site Class D, $S_S = 0.590$ by interpolation
$F_v = 2.06$	From Table 3-13, Site Class D, $S_1 = 0.186$ by interpolation
$S_{DS} = 2/3\,F_a\,S_S$	From Eq. 3-6
	$S_{DS} = 2/3 \times 1.33 \times 0.590$
	$S_{DS} = 0.523$
$S_{D1} = 2/3\,F_v\,S_1$	From Eq. 3-7
	$S_{D1} = 2/3 \times 2.06 \times 0.186$
	$S_{D1} = 0.255$
$S_a = S_{DS} = 0.523$	for $T < S_{D1}/S_{DS}$ from Eq. 3-8
$S_a = S_{D1}/T = 0.255/T$	for $T > S_{D1}/S_{DS}$ from Eq. 3-9
For these examples, assume $S_a = S_{DS}$ controls	
$S_a = S_{DS} = 0.523$	
$F_E = (S_a/R)\,W\,I_{FE}\,I_{MV}$	From Eq. 3-10
$F_E = (0.523/2.0) \times W \times 1.25 \times 1.0$	
$F_E = 0.33\,W$	
Where R	$= 2.0$ for cantilever support (USD)
I_{FE}	$= 1.25$ for essential structures and equipment
I_{MV}	$= 1.0$, assume significant single-mode response

Load combination/Factors: $1.1\,D + 1.25\,E$ (or E_{FS}) $I_{FE} + 0.75\,SC + 1.1\,T_W$ from Table 3.17.

3.6.3 Dead-End Structure (Fig. 3-5), Calculation of Structure Loads

3/8″ High Strength Steel, Overhead Ground Wire (OHGW)

L_s	= 200-ft span
Weight	= 0.273 lb/ft
Diameter	= 0.375 in.

795 MCM ACSR/Phase

L_c	= 200-ft span
Weight	= 1.094 lb/ft
Diameter	= 1.108 in.
Insulators	= Seven suspension insulators (15 lb each)
Wind area	Estimate 0.42 ft² each
Structural members	Square hollow structural section

Calculation of Loads

NESC Heavy Load Case

D — $D_S = 0.273\,\text{lb/ft} \times 200\,\text{ft}/2 = 28\,\text{lb}$
$D_C = 1.094\,\text{lb/ft} \times 200\,\text{ft}/2 + 7$ insulators $\times 15\,\text{lb/insulator} = 215\,\text{lb}$
D_{STR}

I — $I_S = 57\,\text{lb/ft}^3\ ((\Pi(1.375^2 - 0.375^2)/4)\ \text{in.}^2 \times 200\,\text{ft}/2/(12\,\text{in./ft})^2) = 55\,\text{lb}$
$I_C = 57\,\text{lb/ft}^3\ ((\Pi(2.108^2 - 1.108^2)/4)\ \text{in.}^2 \times 200\,\text{ft}/2/(12\,\text{in./ft})^2) + 0.5 \times 7$ insulators $\times 15\,\text{lb/insulator} = 153\,\text{lb}$
$I_{STR} = 0$

T — $T_S = 939\,\text{lb}$ from sag–tension calculations
$T_C = 998\,\text{lb}$ from sag–tension calculations

W — $W_S = 4\,\text{lb/ft}^2 \times 1.375\,\text{in.} \times 200\,\text{ft.}/2/12\,\text{in./ft} = 46\,\text{lb}$
$W_C = 4\,\text{lb/ft}^2 \times 2.108\,\text{in.} \times 200\,\text{ft}/2/12\,\text{in./ft} + 4\,\text{lb/ft}^2 \times 0.9 \times 7$ insulators $\times 0.42\,\text{ft}^3 = 81\,\text{lb}$
$W_{STR} = 4\,\text{lb/ft}^2 \times 2.0 \times A = 8\,\text{lb/ft}^2 \times A_{STR}$

Load combination/Factors (NESC): $1.0\,D + 1.5\,I_{NESC} + 1.65\,T_{NESC} + 2.50\,W_{NESC}$

Extreme Wind Load Case 1

D — $D_S = 0.273\,\text{lb/ft} \times 200\,\text{ft}/2 = 28\,\text{lb}$
$D_C = 1.094\,\text{lb/ft} \times 200\,\text{ft}/2 + 7$ insulators $\times 15\,\text{lb/insulator} = 215\,\text{lb}$
D_{STR}

T_W — $T_S = 522\,\text{lb}$ from sag–tension calculations
$T_C = 857\,\text{lb}$ from sag–tension calculations

F — $F_S = 19.5\,\text{lb/ft}^2 \times 0.375\,\text{in.} \times 200\,\text{ft}/2/12\,\text{in./ft} = 61\,\text{lb}$
$F_C = 19.5\,\text{lb/ft}^2 \times 1.108\,\text{in.} \times 200\,\text{ft}/2/12\,\text{in./ft} + 17.6\,\text{lb/ft}^2 \times 7$ insulators $\times 0.42\,\text{ft}^3 = 232\,\text{lb}$
$F_{STR} = 39.0\,\text{lb/ft}^2 \times A_{STR}$

Extreme wind importance factor: $I_{FW} = 1.0$.
SC: N/A.
Load factors: $1.1\,D + 1.2\,W\,I_{FW} + 0.75\,SC + 1.1\,T_W$

Combined Ice and Wind Load Case 2

D — $D_S = 0.273\,\text{lb/ft} \times 200\,\text{ft}/2 = 28\,\text{lb}$
$D_C = 1.094\,\text{lb/ft} \times 200\,\text{ft}/2 + 7$ insulators $\times 15\,\text{lb/insulator} = 215\,\text{lb}$
D_{STR}

I_W — $I_S = 57\,\text{lb/ft}^3\ ((\Pi(2.375^2 - 0.375^2)/4)\ \text{in.}^2 \times 200\,\text{ft}/2/(12\,\text{in./ft})^2) = 171\,\text{lb}$
$I_C = 57\,\text{lb/ft}^3\ ((\Pi(3.108^2 - 1.108^2)/4)\ \text{in.}^2 \times 200\,\text{ft}/2/(12\,\text{in./ft})^2) + 1.0 \times 7$ insulators $\times 15\,\text{lb/insulator} = 368\,\text{lb}$
$I_{STR} = 0$

Ice importance factor: $I_{FI} = 1.0$.

T_W — $T_S = 1501\,\text{lb}$ from sag–tension calculations
$T_C = 1511\,\text{lb}$ from sag–tension calculations

F_I — $F_S = 3.85\,\text{lb/ft}^2 \times 2.375\,\text{in.} \times 200\,\text{ft}/2/12\,\text{in./ft} = 76\,\text{lb}$
$F_C = 3.85\,\text{lb/ft}^2 \times 3.108\,\text{in.} \times 200\,\text{ft}/2/12\,\text{in./ft} + 3.47 \times 7$ insulators $\times 0.42\,\text{ft}^3 = 110\,\text{lb}$
$F_{STR} = 7.70\,\text{lb/ft}^2 \times A_{STR}$

W (wind with ice importance factor): $I_{FWI} = 1.0$ (per ASCE 7 2005).
SC: N/A.
Load factors: $1.1\,D + 1.2\,I_W\,I_{FI} + 1.2\,W_I\,I_{FWI} + 0.75\,SC + 1.1\,T_W$

Short-Circuit Load Case 3
Not applicable to this structure.

Earthquake Load Case 4

D — $D_S = 0.273\,\text{lb/ft} \times 100\,\text{ft} = 28\,\text{lb}$
$D_C = 1.094\,\text{lb/ft} \times 100\,\text{ft} + 7$ insulators $\times 15\,\text{lb/insulator} = 215\,\text{lb}$
D_{STR}

T_w — $T_S = 232\,\text{lb}$ from sag–tension calculations
$T_C = 454\,\text{lb}$ from sag–tension calculations

F_E — $F_{ES} = 0.33 \times 28\,\text{lb} = 9\,\text{lb}$
$F_{EC} = 0.33 \times 215\,\text{lb} = 71\,\text{lb}$
$F_{ESTR} = 0.33\,W$

SC: N/A.
Load factor: $1.1\,D + 1.25\,E$ (or E_{FS}) $I_{FE} + 0.75\,SC + 1.1\,T_w$

The designer would apply these loads and load combinations to the structure. Then, using the appropriate analysis method, the member stresses are checked. The deflection load cases would be developed, and the deflection limits of Chapter 4 checked.

3.6.4 A 69-kV Disconnect Switch Support Structure (Fig. 3-6), Calculation of Structure Loads

69-kV Disconnect Switch
Weight = 500 lb/phase
Wind area = estimate, 10 ft^2

Rigid Bus 3″ Standard Bus Size (sps) AL (with 266.8 MCM ACSR damper)
Span = 30 ft
Diameter = 3.5 in.
Weight = 2.991 lb/ft = (bus wt = 2.621 lb/ft + damper wt = 0.370 lb/ft)
Structural members: Square hollow structural section.

Extreme Wind Load, Case 1

D (dead load)	D_{SW} = 500 lb/phase D_{bus} = 2.991 lb/ft × (30 ft/2) = 45 lb/phase $D_{structure}$ T_w (wire tension): N/A.
F (wind load)	F_{SW} = 15.55 lb/ft^2 × 10 ft^2 = 156 lb/phase F_{bus} = 17.27 lb/ft^2 × (3.50 in. × 30 ft/2/(12 in./ft)) = 76 lb/phase $F_{structure}$ = 34.55 lb/ft^2 × $A_{Structure}$
Short-circuit load	SC_{bus} = 11.6 lb/ft × 30 ft/2 = 174 lb

Extreme wind importance factor: I_{FW} = 1.0.
Load combination/Factors: 1.1 D + 1.2 $W(I_{FW})$ + 0.75 SC

Combined Ice and Wind Load Case 2

D (dead load)	D_{SW} = 500 lb/phase D_{bus} = 45 lb/phase $D_{structure}$
I (ice load)	I_{bus} = 57 lb/ft^3 ((Π(5.50^2 − 3.50^2)/4) in.^{2} × 30 in./2/(12 in./ft)^2) = 84 lb/phase $I_{structure}$ = 0
F (wind load in combination with ice)	F_{SW} = 3.07 lb/ft^2 × 10 ft^2 = 31 lb/phase F_{bus} = 3.41 lb/ft^2 × (5.50 in. × 30 ft/2/(12 in./ft)) = 23 lb/phase $F_{structure}$ = 6.82 lb/ft^2 × $A_{structure}$
Short-circuit load	SC_{bus} = 8.1 lb/ft × 30 ft/2 = 122 lb

Ice importance factor: I_{FI} = 1.0.
T_w (wire tension): T_w = N/A.
Wind with ice importance factor: I_{FWI} = 1.0 (per ASCE 7 2005).
Load combination/Factors: 1.1 D + 1.2 I_W I_{FI} + 1.2 W_I I_{FIW} + 0.75 SC.

Earthquake Load Case 3

D_3 (dead load)	D_{SW} = 500 lb/phase D_{bus} = 45 lb/phase $D_{structure}$
F_E (earthquake load)	F_{ESW} = 0.33 × 500 lb = 165 lb/phase F_{Ebus} = 0.33 × 45 lb = 15 lb/phase F_{ESTR} = 0.33 W lb
Short-circuit load	SC_{bus} = 8.1 lb/ft × 30 ft/2 = 122 lb

T_w (wire tension): T_w = N/A.
Load combination/Factors: 1.1 D + 1.25 E (or E_{FS}) I_{FE} + 0.75 SC.

Short-Circuit Load Case 4
Load combination/Factors: 1.1 D + 1.00 SC_{bus} + 1.1 T_w.

The designer would apply these load combinations to the structure. Then, using the appropriate analysis method, the member stresses are checked. The deflection load cases would be developed, and the deflection limits of Chapter 4 would be checked.

CHAPTER 4
DEFLECTION CRITERIA

Deflection and rotation of substation structures and members can affect the mechanical operation of supported electrical equipment, reduce electrical clearances, and cause unpredicted stress in structures, insulators, connectors, and rigid bus conductors. For these reasons, structural deflections should be limited to magnitudes that are not detrimental to the mechanical and electrical operation of the substation.

The sensitivity of equipment to deflection of supporting structures varies considerably. Disconnect switches, with complex mechanical operating mechanisms, are highly susceptible to binding if the structure distorts from the installed geometry. Conversely, structures supporting only stranded bus conductors or overhead line dead-ends can withstand structure deflections without any effect on operation. Therefore, structures are classified for the purpose of applying deflection limitations that reflect the sensitivity of supported equipment.

Loading criteria for deflection limitations are recommended in Section 3.1.11 in Chapter 3.

4.1 STRUCTURE CLASSIFICATIONS AND DEFLECTION LIMITATIONS

4.1.1 Deflection Analysis and Criteria

4.1.1.1 Horizontal Members. For determination of maximum deflections, the span of a horizontal member is the clear distance between connections to vertical supporting members, or for cantilever members, the distance from the point of investigation to the vertical supporting member (Fig. 4-1).

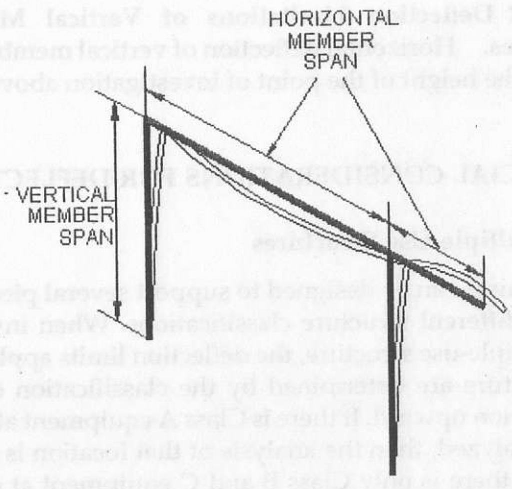


FIGURE 4-1. Span Definitions.

For horizontal members, the deflection is the maximum net displacement, horizontal or vertical, of the member relative to the member connection points. Deflection analysis typically does not include the foundation displacement or rotation.

4.1.1.2 Vertical Members. For determination of maximum deflections, the span of a vertical member is the vertical distance from the foundation support to the point of investigation on the structure. The deflection to be limited is the gross horizontal displacement of the member relative to the foundation support.

4.1.2 Class A Structures

Class A structures support equipment with mechanical mechanisms where structure deflection could impair or prevent proper operation. Examples are group-operated switches, vertical reach switches, ground switches, circuit-breaker supports, and circuit-interrupting devices. Equipment manufacturers should be consulted to determine if any specific structure deflection limits are required for their equipment.

4.1.2.1 Deflection Limitations of Horizontal Members in Class A Structures. Vertical deflection of horizontal members (Fig. 4-2) should not exceed 1/200 of the member span. Horizontal deflection of horizontal members should also not exceed 1/200 of the member span.

4.1.2.2 Deflection Limitations of Vertical Members in Class A Structures. Horizontal deflection of vertical members should not exceed 1/100 of the height of the point of investigation above the foundation.

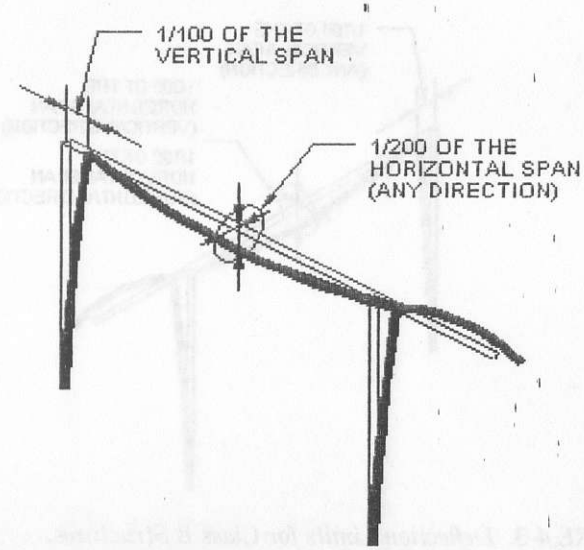


FIGURE 4-2. Deflection Limits for Class A Structures.

4.1.3 Class B Structures

Class B structures support equipment without mechanical mechanisms, but where excessive deflection could result in compromised phase-to-phase or phase-to-ground clearances or unpredicted stresses in equipment, fittings, or bus conductors. Examples are support structures for rigid bus conductors, surge arresters, metering devices (such as CTs, PTs, and CCVTs), station power transformers, hookstick switches or fuses, and wave traps. Equipment manufacturers should be consulted to determine if any specific structure deflection limits are required for their equipment.

4.1.3.1 Deflection Limitations of Horizontal Members in Class B Structures. Vertical deflection of horizontal members (Fig. 4-3) should not exceed 1/200 of the member span. Horizontal deflection of horizontal members should not exceed 1/100 of the member span.

4.1.3.2 Deflection Limitations of Vertical Members in Class B Structures. Horizontal deflection of vertical members should not exceed 1/100 of the height of the point of investigation above the foundation.

4.1.4 Class C Structures

Class C structures support equipment relatively insensitive to deflection or are stand-alone structures that do not support any equipment. Examples are support structures for flexible (stranded conductor) buses, masts for

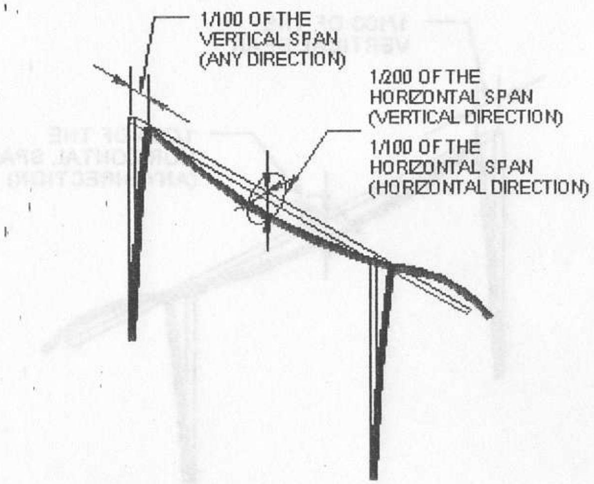


FIGURE 4-3. Deflection Limits for Class B Structures.

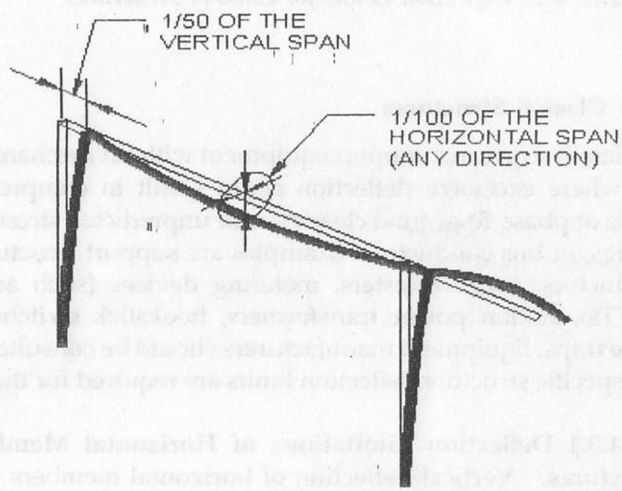


FIGURE 4-4. Deflection Limits for Class C Structures.

lightning shielding, and dead-end structures for incoming transmission lines. Deflection limitations for these structures are intended to limit P-delta stresses, wind-induced vibrations, and visual impact.

4.1.4.1 Deflection Limitations of Horizontal Members in Class C Structures. Vertical deflection of horizontal members (Fig. 4-4) should not exceed 1/100 of the member span. Horizontal deflection of horizontal members should not exceed 1/100 of the member span.

4.1.4.2 Deflection Limitations of Vertical Members in Class C Structures. Horizontal deflection of vertical members should not exceed 1/50 of the height of the point of investigation above the foundation.

4.2 SPECIAL CONSIDERATIONS FOR DEFLECTION ANALYSIS

4.2.1 Multiple-Use Structures

Structures can be designed to support several pieces of equipment that require different structure classifications. When investigating deflection of a multiple-use structure, the deflection limits applicable to any point on the structure are determined by the classification of the structure from that location upward. If there is Class A equipment at or above the location being analyzed, then the analysis of that location is governed by Class A limits. If there is only Class B and C equipment at or above the location being analyzed, then the analysis of that location is governed by Class B limits. If there is only Class C equipment at or above the location being analyzed, then the analysis of that location is governed by Class C limits. As an example, Fig. 4-5 shows a line dead-end structure (Class C) that also supports a switch (Class A) at a lower elevation. The switch platform and the vertical members from the foundation to the switch platform should meet Class A deflection criteria. Members associated with the line

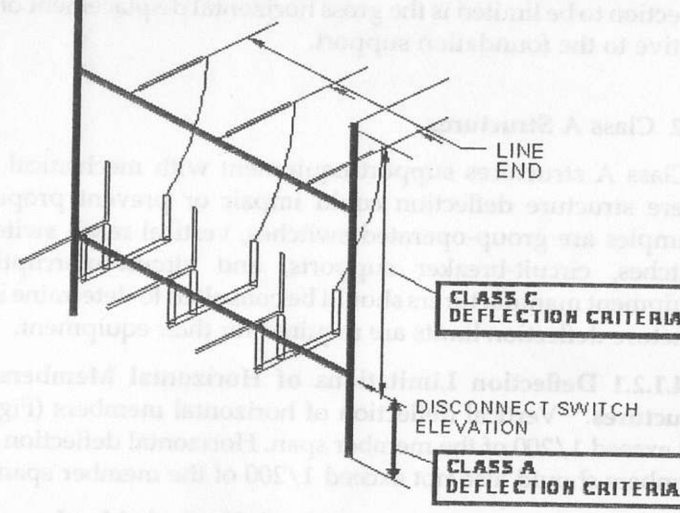


FIGURE 4-5. Multiple-Use Structures.

dead-end and vertical members from the switch elevation to the line dead-end should meet Class C criteria.

4.2.2 Rotational Limitation

Some equipment and rigid bus designs may be sensitive to rotation of supporting members in addition to the deflection of the member. Equipment manufacturers should be consulted as to any rotational limits that may be necessary to ensure reliable operation. Where an analysis is performed of the rigid bus and support system, the sensitivity of the system to support rotation should be investigated and limits determined if necessary.

4.2.3 Lightning Masts and Other Tall, Slender Structures

In certain cases, the structure type, design loads, and lower deflection limits for Class C structures can result in a flexible (low stiffness) structure. These structures can be subject to potentially damaging wind-induced oscillations. Such structures can be susceptible to fatigue cracking and failure. In addition to the specified static deflection limits, consideration should be given to the use of dampening devices or other techniques to minimize potential for damage. Methods include the use of internal cables or covered chains, external spoilers, or other means of interrupting the oscillations. Additional information on structural member vibrations is discussed in Sections 6.9.10 and 6.10.2 in Chapter 6.

TABLE 4-1. Summary of Structure Deflection Limitations

Maximum Structure Deflection as a Ratio of Span Length[a]				
		Structure Class		
Member Type	Deflection Direction	Class A	Class B	Class C
Horizontal[b]	Vertical	1/200	1/200	1/100
Horizontal[b]	Horizontal	1/200	1/100	1/100
Vertical[c]	Horizontal	1/100	1/100	1/50

[a]For loading criteria for deflection limitations, see Section 3.1.11 in Chapter 3.
[b]Spans for horizontal members should be the clear span between vertical supports, or for cantilever members, the distance to the nearest vertical support. Deflection should be the net displacement, horizontal or vertical, relative to the member support points.
[c]Spans for vertical members should be the vertical distance from the foundation connection to the point of investigation. Deflection should be the gross, horizontal displacement relative to the foundation support.

4.2.4 Rigid Bus Conductor Deflection Criteria

To obtain an acceptable appearance, it is recommended that the vertical deflection of rigid bus conductors (aluminum or copper tubing or shapes) be limited to 1/200 of the bus span. This criterion should be applied with the deadweight of the rigid bus, with dampers, and with no ice or wind. Vertical deflection of rigid bus conductors is measured from the attachment point of the bus conductor at the insulators to the midspan point.

4.3 SUMMARY

Table 4-1 summarizes the structure classes and associated deflection limits.

CHAPTER 5

METHOD OF ANALYSIS

5.1 OVERVIEW

The design of substation structures requires a knowledge of the equipment being supported by the structure, its operation, and electrical and safety codes.

Analysis, as used herein, is defined as the mathematical formulation of the behavior of a structure under load. The solution yields the calculated displacements, support reactions, and internal forces or stresses. The analysis of a structure begins by developing a model that defines the structural configuration, connection characteristics, support boundary conditions, and loading cases.

5.2 STRESS CRITERION VS. DEFLECTION CRITERION

An analysis of a structure generates both stresses and deflections, either of which may determine the adequacy of the structure.

A structure should provide sufficient capability to support its own weight and any combination of loads as specified in Chapter 3. The internal stresses under these loading cases should be at or below predetermined values specified in Chapter 6 or verified by physical tests.

A structure designed for strength may have excessive deflections. Excessive deflection of a structure may or may not in itself be detrimental, but the effects on substation components that are supported by the deflecting structure are frequently quite significant. Deflection may cause aesthetic displeasure to the viewer or preclude the equipment from performing its intended purpose. A design controlled by deflection is said to be based on serviceability. Some substation structures have rigorous deflection limitations, whereas others are not so restricted. The substation structure deflection criteria are specified in Chapter 4. These criteria are based on the function of the structure. Deflection may control the design of substation structures.

5.3 THE STRUCTURE MODEL

The analysis model is a set of mathematical equations to predict the performance of the structure. The accuracy of the analysis is only as good as the model. Thus, it is important to construct an adequate mathematical model to simulate the true behavior of the structure.

In developing a model of a structure, assumptions are made concerning the individual elements, their geometrical and mechanical properties, and the connections, loads, and foundation supports. These are discussed below.

5.3.1 Individual Members and Connections

A structure may consist of only a few members, as in the case of a steel pole, or a group of members, as in the case of a lattice tower. Members may be as simple as a two-dimensional truss element or as complex as a plate in bending in a finite element analysis. The choice of the element type depends on the structural geometric configuration and the desired load flow into each element. The choice of the element also defines the type of connection in the analysis model.

5.3.2 Truss Model

A truss has a geometric configuration consisting of elements forming triangles. The elements in a truss are assumed to be two force members providing load resistance by axial force only. The accuracy of the analysis using a truss model depends on the connections being detailed to connect the neutral axes of the truss elements and the end connections to simulate a pinned joint.

A truss model used in an analysis assumes that there is only axial load transmitted by and to the members. Care should be taken in the connection design to ensure these assumptions. Any eccentricities introduced by the connection will introduce moment into the member and could make the analysis inaccurate or cause the member to have induced bending stresses, which could cause an overstress condition.

A truss model implies that all the nodes are pinned. This assumption may sometimes need to be verified for main leg members, which in reality

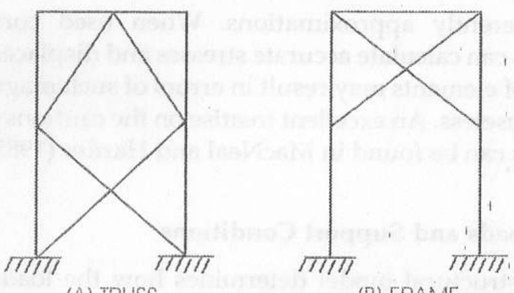


FIGURE 5-1. Truss and Frame Comparison.

are continuous elements throughout. For this reason, depending on the geometry of the structure, some shear forces could be transferred to the main leg members, therefore inducing bending moments that may need to be checked.

Subtle differences can significantly alter the behavior of a structure. For example, the structure shown in Fig. 5-1A could be analyzed as a truss and would not have significant bending moments. However, the structure in Fig. 5-1B may be governed by bending stresses due to the shear acting over the unbraced lower section of the leg.

5.3.3 Frame Model

The elements in a frame model provide resistance to load by forces and bending moments. If the frame model is three-dimensional, moments may be produced in three orthogonal directions. The connections in a frame model may be configured to resist or not resist moment (including torsion). The configuration of the connection is accounted for in a frame model by defining moment or force restraints. If moment or torsion is to be transferred, the connection should form a rigid connection between the elements of the cross section resisting the moment or torsion, i.e., in a wide flange, the flanges resist moment.

5.3.4 Finite Element Model

More complex elements are used in finite element analysis. The most common are plane stress or plane strain (membrane forces), plate in bending, and solid elements. Any or all of these elements may be combined with each other or included in a structure having truss or frame elements. Care should be taken when using finite element analysis because the displacement functions used in formulating the element's performance are inherently approximations. When used correctly, finite element analysis can calculate accurate stresses and displacements. However, poor choice of elements may result in errors of such magnitude as to render the results useless. An excellent treatise on the cautions of using finite element analysis can be found in MacNeal and Harder (1985).

5.3.5 Loads and Support Conditions

The structural model determines how the loads are applied and the types of support conditions. The joints of a truss element can only translate, whereas the joints of a frame element can translate and rotate. These are the degrees of freedom. A membrane and solid element have degrees of freedom in translation; a plate in bending has rotational degrees of freedom about axes within the plane of the element. The degrees of freedom define the limits of the loads and support conditions. A truss model can only have translational joint loads; a frame model can have translational and rotational joint loads. A force applied in the span of a truss element is not allowed in the model because it cannot be transferred to the joint without inducing bending in the element, which violates the assumptions defining a two-force member. The effect of these forces should be checked using an analysis of the member, taking into account the appropriate bending stiffness of the member end conditions. A frame can have loads applied in its span because the flexure strength of the element allows the span load to be transferred to the joints and thus into the rest of the frame.

The allowed loads on these finite elements are predetermined by the formulation of the code of the computer program being used and should be understood by the design engineer. The allowed joint loads correspond to the degrees of freedom, i.e., a translation degree of freedom allows a translation load.

The support conditions define boundary restraints of the structure. If the support is rigid, it is a known displacement field whose value is zero. A nonzero value defines a support movement. The allowed specified deflections are limited by the degrees of freedom of the joint providing the boundary condition or support of the structure. Not all degrees of freedom at a support need to be specified. Those not specified are unknown and are calculated in the analysis.

It is the design engineer's responsibility to ensure that the final physical structural configuration and the model agree. If the analysis used a truss model, the connections should be designed to provide truss action only or to have the line of action of the internal forces pass through a common point. If eccentricities are not accounted for in the analysis, they should be minimized in the physical design.

This manual defines the allowable deflections in substation structures. The structure model will provide the calculated deflection. If the model is assumed stiffer than the actual structure, the calculated deflections will be less than the actual deflections, and vice versa. Thus, the model should duplicate the stiffness of the actual structure for a valid check of the serviceability requirements.

5.4 STATIC ANALYSIS METHOD

5.4.1 Approximate Analysis

Many approximate analysis techniques are documented in structural analysis texts. Some approximate analyses are done based on the experience of the analyst, wherein simplifying assumptions are made and the stresses and deflections are calculated. The design is considered adequate if all strength and serviceability criteria are satisfied. Approximate analysis is not recommended for complex structural configurations. Approximate analyses have value in the preliminary design stage or as an independent cursory check of a computer analysis.

5.4.2 First-Order Elastic Analysis

In a first-order elastic analysis, equilibrium is formulated on the undeformed geometry. Such an analysis is performed using one of the classical structural analysis techniques such as statics, slope deflection, moment distribution, and matrix methods using the stiffness approach. The analysis assumes linear material behavior as an element is loaded and unloaded irrespective of the magnitude of deflection, load, or stress. The calculations in a first-order elastic analysis use the undeformed shape of the structure. Therefore, any amplification in moment or any other force due to the deformation of the structure is not accounted for.

In the case of internal bending moment, this is commonly referred to as the *P-delta amplification.* White and Hajjar (1991) point out that there are two P-delta effects. One is caused by joint translation commonly referred to as side sway, whereas the other is induced by the internal curvature of the bending member irrespective of the translation of the joints. Some design equations account for the P-delta effect on stresses. AISC (2005a) refers to this as an "amplification factor," and it is included in the equations for calculating the requirements of members subjected to both axial compression and bending stresses. Using equations to calculate stresses including the P-delta effect ensures that the stress levels are acceptable. These equations do not account for the added deflection produced by the P-delta secondary effect.

5.4.3 Second-Order Elastic Analysis

In a second-order elastic analysis or a geometric nonlinear analysis, equilibrium is formulated on the deformed configuration of the structure. This analysis is commonly thought of as including the amplification of moment due to the axial load × the deformation of the compression member, i.e., the P-delta amplification in bending moment. However, geometric nonlinear effects can show up in any structure. A lattice tower modeled as a three-dimensional truss has geometric nonlinear effects as the tower moves under load.

Including the geometric nonlinear effect is not as simple as reformulating the analysis using the deflected shape. As the structure deflects, internal forces are produced and need to be accounted for in the analysis of the deformed structure. Cook (1974) gives details of how to perform an analysis including the geometric nonlinear effects. Even so, this method only includes the effects of joint translation and not the internal curvature of the member. To include this in the analysis would require subdividing the individual members. In general, a lattice tower modeled as a truss model has negligible geometric nonlinear effects. A pole structure or cable structure may exhibit significant geometric nonlinear effects.

Some changes to design equations can account for the geometric nonlinear effects by reducing the allowable stress or increasing the force in the member. These changes should not be included if an analysis includes all the geometric nonlinear effects.

An analysis including the geometric nonlinear effects is most important in determining a realistic value for deflection. This analysis method should be considered for analyzing displacement-sensitive flexible substation (Class A) structures.

5.4.4 First-Order Inelastic Analysis

Material yield or nonlinear member performance is accounted for in a first-order inelastic analysis. A plastic analysis of a rigid moment resistant frame is a classical example of a first-order inelastic analysis (Beedle 1958). This technique is based on a structure being able to carry load beyond the elastic limit until it reaches its ultimate load through plastic deformation. It is applicable to indeterminate rigid-jointed frames, continuous beams, and in general structures stressed primarily in bending.

A determinate beam or frame under a given set of loads has one point at which the moment is maximum. If this load is increased, this moment increases and ultimately forms a plastic hinge. When the bending stress block reaches yield, a collapse mechanism has formed.

An indeterminate (hyperstatic) structure behaves differently than a determinate (isostatic) beam. It will not collapse until the number of plastic

hinges equals one plus the degree of indeterminacy. The concept of plastic analysis and design uses this reserve strength to produce more economical structures. The engineer uses plastic analysis to determine the load that will produce a plastic hinge mechanism of the structure and the internal moments used in plastic design. AISC (2005a) recognizes plastic analysis as an acceptable analysis technique. However, this manual does not recommend using plastic analysis.

Mueller et al. (1991) proposed a method to account for nonlinear member performance in the analysis of transmission towers. In it, the individual member performance is defined by a nonlinear member load vs. member deflection curve.

5.5 DYNAMIC ANALYSIS METHOD

5.5.1 Steady-State Analysis

In a steady-state analysis, the load {R} is of the form

$$\{R\} = \sin(\omega t)\{F\} \quad \text{(Eq. 5-1)}$$

where ω = circular frequency and t = time, and the equilibrium equations are the following:

$$[M]\{\ddot{y}\} + [K]\{y\} = \{R\} \quad \text{(Eq. 5-2)}$$

where {F} = loads, $[M]$ = mass, $\{y\}$ = displacement, $\{\ddot{y}\}$ = acceleration, and $[K]$ = structural stiffness.

The steady state assumes zero damping and requires the loading frequency of all loads to be the same.

5.5.2 Eigenvalue Analysis: Natural Frequencies and Normal Modes

In an eigenvalue analysis, free vibration is considered, and the structure is not subjected to external forces or support motion. The following equation describes this free vibration state:

$$[M]\{\ddot{y}\} + [K]\{y\} = 0 \quad \text{(Eq. 5-3)}$$

A solution is sought in the form of

$$\{y\} = \{a\}\sin(\omega t) \quad \text{(Eq. 5-4)}$$

where a_i is the amplitude of motion of the ith degree of freedom. Substituting Eq. 5-4 into Eq. 5-3 yields the following:

$$[[K] - \omega^2[M]]\{a\} = 0 \qquad \text{(Eq. 5-5)}$$

The nontrivial solution of Eq. 5-5 results in normal or natural modes of free undamped motion. The shapes are normal mode shapes or modal shapes, each of which has its natural frequency, ω.

5.5.3 Response Spectrum Analysis

A response spectrum is a plot of the maximum response (e.g., displacement, velocity, or acceleration) to a specified load function for all possible single degree-of-freedom systems. Typically, the abscissa of the spectrum is the natural frequency or period of the structural system, and the ordinate is the maximum response. Figure 5-2 shows the recommended normalized response spectrum that presents maximum amplitude for a single degree-of-freedom resonance response. This response spectrum is the IEEE 693 (2005) response spectrum normalized to 1.0 g. The response spectrum acceleration in Fig. 5-2 can be adjusted for a different zero period

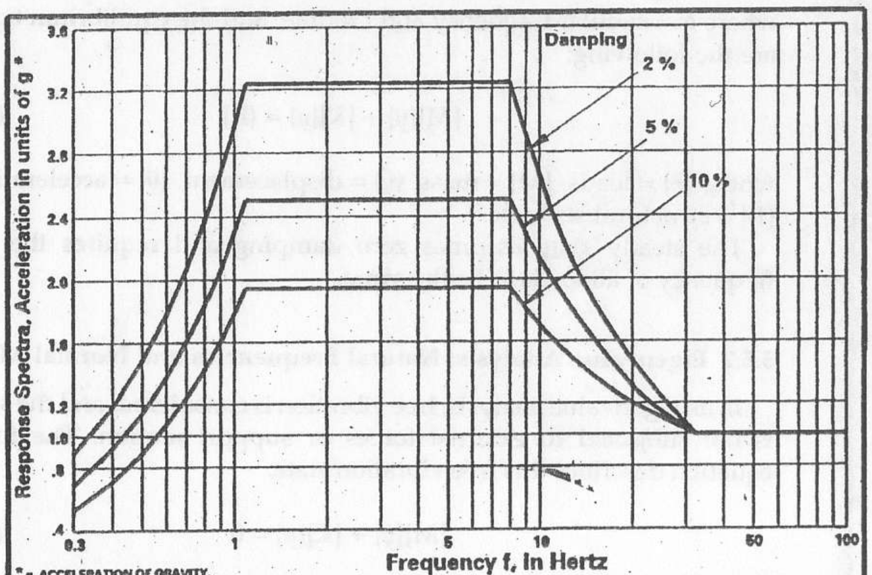


FIGURE 5-2. 1.0-g Normalized Response Spectrum. (Reprinted with permission from IEEE Std. 693, Recommended Practice for Seismic Design of Substation Structures, Copyright 1997, by IEEE. The IEEE disclaims any responsibility or liability resulting from the placement and use in the described manner)

acceleration, and the acceleration value can be determined by picking an acceleration value using the structure natural frequency obtained from an eigenvalue analysis. A site-specific response spectrum can also be developed per Section 6.7.5 of Chapter 6 and used for a response spectrum analysis.

5.6 RECOMMENDATION FOR AN ANALYSIS METHOD

5.6.1 Static Analysis

For some structures, such as simple bus supports, an approximate analysis (as in Section 5.4.1) may be adequate. When ASD is to be used to design the structure, a first-order elastic analysis (as in Section 5.4.2) should be used in analyzing the strength and deflection of the structure. The ASD equations account for the geometric nonlinear effects.

When USD is to be used for the structure design, a second-order elastic analysis (as in Section 5.4.3) should be used in analyzing the strength and deflection of the structure because the equations used in USD depend on the P-delta moments being included in the analysis. In AISC (2005), Chapter C states the requirement for the second-order analysis.

5.6.2 Dynamic Analysis

Structures supporting electrical equipment and the electrical equipment mounted on them should be analyzed together to model accurately the mass distribution and stiffness characteristics of the structure and the electrical equipment. This model may be used to perform a modal (eigenvalue) analysis to determine if the structure and equipment are rigid or flexible.

For simple beam structures, the fundamental natural frequency can be approximated from the equations shown in Table 5-1. Equation 5-6 is a suggested simplification for combining the individual fundamental frequencies of the structure and equipment to obtain an approximate system fundamental frequency.

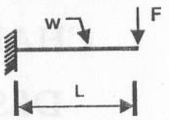
$$1/f^2 = 1/f_e^2 + 1/f_s^2 \qquad \text{(Eq. 5-6)}$$

where f = the equipment structure system frequency approximation, f_e = the frequency of equipment, and f_s = the frequency of the structure.

The natural frequency of the equipment-support structure system can be altered when its base plate is supported by anchor bolts with leveling nuts, as in Section 7.3.2.2 in Chapter 7. This possibility should be considered when evaluating the seismic behavior of the equipment-support structure system.

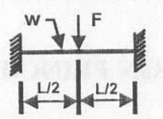
TABLE 5-1. Natural Frequency (cycles/s) Equations for Simple Beams

Cantilever Beam



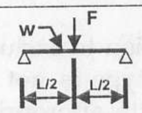
F(force)	w (force/Lt)	F and w
$f = \dfrac{3.13}{\sqrt{\dfrac{FL^3}{3EI}}}$	$f = \dfrac{3.89}{\sqrt{\dfrac{wL^4}{8EI}}}$	$f = \dfrac{3.13}{\sqrt{\dfrac{(F + 0.223wL)L^3}{3EI}}}$

Fixed End Beam



F(force)	w (force/Lt)	F and w
$f = \dfrac{3.13}{\sqrt{\dfrac{FL^3}{192EI}}}$	$f = \dfrac{3.52}{\sqrt{\dfrac{wL^4}{384EI}}}$	$f = \dfrac{3.13}{\sqrt{\dfrac{(F + 0.389wL^3)L}{192EI}}}$

Pinned End Beam



F(force)	w (force/Lt)	F and w
$f = \dfrac{3.13}{\sqrt{\dfrac{FL^3}{48EI}}}$	$f = \dfrac{3.52}{\sqrt{\dfrac{5wL^4}{384EI}}}$	$f = \dfrac{3.13}{\sqrt{\dfrac{(F + 0.486wL^3)L}{48EI}}}$

Notes: E = modulus of elasticity (kips/in.^{2}); I = moment of inertia (in.^{4}); w = uniform load (kips/in.); F = point load (kips); L = beam length (in.); and kips = 1000 pounds.

A static analysis may be used for rigid structures in substations. For this type of analysis, the structure and equipment should be designed to resist the forces resulting from an acceleration of the structure or equipment base equal to the maximum ground acceleration.

5.6.2.1 Earthquake Analysis. For flexible structures, a dynamic analysis using the response spectrum analysis method should be performed. A damping value of 2% should be used, unless a higher value is justified. The maximum modal response can be determined either by using the response spectrum shown in Fig. 5-2 adjusted for the selected ground acceleration, using S_{DS} and S_{D1} values and the FEMA 450 (2004) design response spectrum or a site-specific response spectrum (see Section 6.7.5 in Chapter 6). The total response should be determined by combining each modal response by the square root of the sum of the squares (SRSS) or the complete quadratic combination (CQC) methods.

The SRSS method assumes that the vibration modes are independent and therefore does not account for cross-coupling vibration mode effects. For structures with coupled modes of vibration, structures with unsymmetrical stiffness or mass, this can result in underestimating the dynamic response. When using the SRSS method, the response of vibration modes whose frequencies differ by less than 10% should be first summed using absolute values. The CQC method has the advantage of accounting for the cross-coupling effects of vibration modes. For both methods, a sufficient number of natural modes of vibration should be used such that total response does not increase by more than 10% with the addition of more modes. The acceleration, displacement, force, and moment response due to motion in the three orthogonal directions should be combined by SRSS.

Static coefficient analysis, as described in IEEE 693 (2005), may be used as an alternate method of analysis for simple flexible structures having one or two dominant modes. The peak acceleration obtained from the response spectrum is multiplied by 1.5 to account for multiple mode effects. The calculated response, loads, and stresses in each of the principle axes are combined using the SRSS method. This analysis technique allows a simpler procedure in return for added conservatism.

For systems in series, the structure and equipment should be considered rigid if the natural frequency exceeds 33 cycles/s (Hz).

5.6.2.2 Dynamic Analysis of Short-Circuit Events. It is sometimes necessary to model an entire rigid bus system. When rigid A-frames are used to connect two buses that are turned 90 degrees to one another but separated vertically, a dynamic analysis may be needed to determine the stresses in the bus and the supporting structures. The short-circuit dynamic forcing function should be obtained from an electrical engineer and applied to the bus conductor. The bus conductor, insulators, and support structures for the substation should be modeled in a finite element structural computer program. The short-circuit load plus any other static loads that may occur simultaneously should be applied to the model. The dynamic and static load events will produce deflections and stresses in the bus conductors, insulators, and support structures that should be lower than the allowable values for each component. Refer to Sections 3.1.8 in Chapter 3 and 6.9 in Chapter 6 for additional information on short-circuit design of substation structures.

CHAPTER 6

DESIGN

6.1 GENERAL DESIGN PRINCIPLES

Specific guidelines for member design and fabrication are not included in this manual. This manual refers to other documents for design guidelines and notes any exceptions to the referenced documents.

Load factors, load combinations, load cases, and deflection criteria specified in Chapters 3 and 4 should be used with referenced design codes or documents.

There is no intention to exclude any material or section types. If the material or section type is not addressed in this manual, the design engineer should use the appropriate design code or document.

6.2 DESIGN METHODS

Allowable stress design (ASD) is a method of proportioning structural members such that elastically computed stresses produced in the members by nominal loads do not exceed specified allowable stresses. ASD is also called working stress design.

Ultimate strength design (USD) is a method of proportioning structural members such that the computed forces produced in the members by the factored loads do not exceed the member design strength. USD is also called load and resistance factor design or LRFD.

USD and ASD are both acceptable for design of substation structures. Ultimate strength design is recommended.

ASD uses unfactored design loads and limits stress levels to a value that is less than the yield strength of the material. If ASD is used with the intention of meeting NESC (2007) (or local regulatory code) load requirements, loads should be increased to account for the NESC (2007) load factors. The 1/3 increase in the allowable stress for wind and seismic loading is not recommended for substation structures. The 1/3 increase provision was intended for building structures where the dead loads are the major contributing factor to the design of the structure and there are multiple paths (as in an indeterminate or hyperstatic system) for the load to be distributed within the structure. Most substation structures have only one path for the load to travel (this is called a determinate or isostatic system), and the dead load is typically a minor design factor.

Plastic analysis or design is not recommended for substation structures.

Structures that support conductors and overhead ground wires that extend outside the boundaries of the substation should meet or exceed the load and strength requirements of the NESC (2007) or local regulatory codes. For structures that are required to meet NESC (2007) load criteria, it is recommended that the ultimate strength design be used because the NESC code (2007) specifies load factors.

6.3 STEEL STRUCTURES

ASCE 10 (1997) is recommended for design of lattice structures using angles (see Section 6.3.1.1).

AISC (2005) is recommended for design of structures using standard structural shapes (see Section 6.3.1.2).

ASCE/SEI 48 (2005) is recommended for design of structures using hollow tubular member shapes (see Section 6.3.1.3).

6.3.1 Ultimate Strength Design

6.3.1.1 Lattice Angle Structures. ASCE 10 (1997) should be used for ultimate strength design of lattice structures. This standard uses factored design loads, linear material properties, and first- or second-order elastic analysis. This standard does not use strength reduction factors.

ASCE 10 (1997) has adjusted column equations for angles, which account for the effect of eccentricities in connections that are commonly used in lattice angle structures. The effects of flexural–torsional and torsional buckling are also included. For these reasons, ASCE 10 (1997) is recommended for designing lattice substation structures constructed using angle sections.

Steel lattice structures using angle sections can also be designed in accordance with AISC (2005). Load and resistance factored design (LRFD) should be used with factored design loads, linear material properties, LRFD member capacity reduction factors, and second-order elastic analysis.

LRFD, as with ASD (in Section 6.3.2.1), states that compression members should be loaded through the centroidal axis. When the loading is not through the centroidal axis, LRFD requires that the combined stress equation for bending and axial loads be used and that flexural–torsional and torsional buckling be considered for these members.

6.3.1.2 Standard Structural Shapes Other Than Angles. Examples of standard structural shapes other than angles are wide flanges, channels, tubes, pipes, and tee sections. LRFD should be used for design of these member shapes with factored design loads, linear material properties, LRFD member capacity reduction factors, and second-order elastic analysis.

To account for uniformly tapered open member shapes, an equivalent slenderness ratio can be calculated in the same manner as discussed in Section 6.3.2.2.

6.3.1.3 Hollow Tubular Member Shapes. Hollow tubular member shapes include 4-, 6-, 8-, and 12-sided polygonal sections and circular sections. These members should be designed and fabricated in accordance with ASCE/SEI 48 (2005).

The local buckling equations for polygonal sections in ASCE/SEI 48 (2005) identify the stress level for collapse of the section, rather than the stress level at which local buckling is initiated.

The local buckling equations for 8- and 12-sided members are based on bending tests. In these bending tests, one of the flats is initially loaded in uniform compression and becomes inelastic before the adjacent flats because the adjacent flats are at a lower, nonuniform stress level. If the section does not collapse, further increases in compressive stress are distributed to adjacent flats, giving the section additional postbuckling strength. If these sections were loaded with a uniform compressive stress over the entire cross section, there would not be a redistribution of stresses to adjacent flats because they are all loaded equally. Accordingly, the additional postbuckling strength does not exist, and the local buckling equations in ASCE/SEI 48 (2005) may overpredict the collapse strength of 8- or 12-sided members loaded in axial compression. For this reason, the local buckling equations in ASCE/SEI 48 (2005) for rectangular shapes with $F_a > 1\,\text{kip/in.}^2$ are recommended for 6-, 8-, and 12-sided polygonal sections with $F_a > 1\,\text{kip/in.}^2$.

ASCE/SEI 48 (2005) uses factored design loads, linear material properties, and a second-order geometric nonlinear analysis method

(P-delta effect is included). ASCE/SEI 48 (2005) also allows for uniformly tapered members.

LRFD requires second-order linear analysis. LRFD should be used with factored design loads, linear material properties, and LRFD member capacity reduction factors. To account for uniformly tapered members using LRFD, an equivalent slenderness ratio can be calculated in the same manner as given in Section 6.3.2.2.

ASCE/SEI 48 (2005) is the recommended method for design because the effective length factor, K; amplification factors to account for P-delta effects; and factors to account for tapered members do not have to be determined.

6.3.1.4 Local Buckling of Irregular Polygonal Shapes. The local buckling assumptions in ASCE/SEI 48 (2005) cover regular polygonal shapes of more than 4 sides where all the flats are the same width. For flats that bisect the neutral bending axis, the local buckling strength k factor can be increased over the k factor for flats in uniform compression. For structures that use an irregular polygonal shape and have high w/t ratios for the two flats that are on the neutral axis, the buckling stress of the long flats can be calculated using Eq. 6-1 (Bleich 1952).

$$\sigma_{cr} = \frac{\pi^2 E}{12(1-\upsilon^2)}\left(\frac{t}{w}\right)^2 k, \text{ but less than } F_y \qquad \text{(Eq. 6-1)}$$

where

σ_{cr} = critical buckling stress,
E = Young's modulus,
F_y = yield strength stress,
υ = Poisson's ratio,
t = plate thickness,
w = plate width in compression, and
k = the local buckling factor.

For flats that have tension on one corner and compression on the other corner, the width w can be taken as the distance from the corner in compression to the point along the flat that has zero compression. Then k is 7.7 for this condition. For flats where both corners are in compression, k is 4.0, as in ASCE/SEI 48 (2005), and the width w can be taken as the length of the flat between the actual inside bend radii or 4 times the thickness, as specified in ASCE/SEI 48 (2005).

Using this method, the stress at each corner of the polygon must be calculated for each load case and for the entire length of the member because the buckling strength is load case and position dependent.

6.3.2 Allowable Stress Design

AISC (2005) should be used for allowable stress design. ASD uses unfactored design loads, linear material properties, and first-order elastic analysis. The 1/3 increase in the allowable stress for wind and seismic loading is not recommended for substation structures (refer to Section 6.2).

6.3.2.1 Angles and Other Open Member Shapes. Allowable stress design in AISC (2005) states that compression members should be loaded through the centroidal axis. This requirement is rarely obtained in lattice structures because they commonly consist of angles that are bolted on one leg. When the loading is not through the centroidal axis, ASD requires that the combined stress equation for bending and axial loads be used. ASD also requires that flexural–torsional and torsional buckling be considered for these members.

To account for uniformly tapered open member shapes, an equivalent slenderness ratio can be calculated in the same manner as discussed in Section 6.3.2.2.

6.3.2.2 Hollow Tubular Member Shapes. Hollow tubular member shapes generally include 4-, 6-, 8-, or 12-sided polygonal sections and circular sections.

To account for uniformly tapered members using ASD, an equivalent slenderness ratio can be used as given by the following equation:

$$\left(\frac{KL}{r}\right)_{eq} = \frac{KL}{(P^* r_0)^{1/2}} \qquad \text{(Eq. 6-2)}$$

where

r_0 = radius of gyration at the small end;
P^* = the coefficient that accounts for the effect of the taper;
K = the effective length factor; and
L = the member unbraced length.

Values of P^* are given in Gere and Carter (1962). The procedure gives conservative results in the inelastic range of buckling. This happens because all cross sections of the member do not become inelastic simultaneously, as in a column of uniform cross section. Instead, the cross section at the small end, where the axial stress is largest, is the first to become inelastic. Then, if the member does not buckle at this load, adjoining cross sections become successively inelastic as the load increases, until the buckling load is reached. This equation does not apply if sudden changes in cross-sectional properties occur, as in a member composed of segments of different thickness.

A second-order geometric nonlinear analysis method (including the P-delta effect) may be used with the following exceptions to ASD:

1. Because the P-delta effect is included, and therefore the amplification factor, $C_m/(1 - f_a/F'_e)$, should not be included in the combined axial compression and bending equations.
2. Because the elastic stability is checked in a second-order analysis, F_a (allowable uniform compression stress) and F_b (allowable bending stress) in the combined axial compression and bending equations should be equal to the lower of 0.6 F_y (F_y = minimum yield strength) or the allowable local buckling stress as defined by ASD.

6.4 CONCRETE STRUCTURES

Concrete structures are designed to accommodate cracking behavior. In a corrosive environment, water may be absorbed into the open cracks and corrode the reinforcing steel. Typical substation concrete structures should provide enough concrete cover to protect the reinforcing steel. For structural members subjected to sustained flexure loading, such as dead-end structures, it may be desirable to allow no tensile stress along the member cross section under everyday loading conditions. This zero-tension criterion will prevent cracks from staying open under normal situations and will preclude the reinforcing steel from corroding.

6.4.1 Reinforced Concrete Structures

Reinforced concrete structures should be designed and constructed in accordance with ACI 318 (2005). This code uses the ultimate strength design method with factored design loads, linear material properties, and second-order elastic analysis. Member capacity reduction factors should be used as specified in this code.

6.4.2 Prestressed Concrete Structures

Prestressed concrete structures should be designed and constructed in accordance with PCI MNL-120 (2004). This handbook uses the ultimate strength method with factored design loads, linear material properties, and second-order elastic analysis. Member capacity reduction factors should be used as specified in PCI MNL-120 (2004).

6.4.3 Prestressed Concrete Poles

The prestressed concrete pole type structures, either static cast or spun cast, should be designed and constructed in accordance with ASCE-PCI (1997). This guideline uses the ultimate strength design method and, in general, follows all ACI and PCI recommendations.

6.5 ALUMINUM STRUCTURES

The Aluminum Association's design criteria for aluminum structures is recommended in this manual. Additional information on aluminum structure design is available in ASCE (1972), Mooers (2006), and Kissell and Ferry (1995).

6.5.1 Ultimate Strength Design

Aluminum structures should be designed and fabricated in accordance with the Aluminum Association (2005) design manual, using stresses for bridges and similar structures.

6.5.2 Allowable Stress Design

Aluminum structures should be designed and fabricated in accordance with the Aluminum Association (2005) design manual, using stresses for bridges and similar structures.

6.5.3 Aluminum with Dissimilar Materials

Aluminum corrodes when in contact with dissimilar materials, such as steel, wood, or concrete. These dissimilar materials have a different pH than that of aluminum. Aluminum functions best when in contact with material having a pH range of 5 to 9.

6.5.3.1 Steel. Aluminum surfaces to be placed in contact with steel should be given one coat of a zinc chromate primer complying with Federal Specification TT-P-645B (1990), or the equivalent, or one coat of a suitable nonhardening joint compound that can exclude moisture from the joint during prolonged service. Additional protection can be obtained by applying the joint compound in addition to the zinc chromate primer. The zinc chromate paint should be allowed to dry before the parts are assembled.

Aluminum surfaces to be placed in contact with stainless, aluminized, hot-dip galvanized, or electrogalvanized steel need not be painted.

6.5.3.2 Wood. Aluminum surfaces to be placed in contact with wood should be given a heavy coat of an alkali-resistant bituminous paint before installation. The paint should be applied in the condition in which it is received from the manufacturer without the addition of any thinner.

6.5.3.3 Concrete. Aluminum should not come into contact with wet concrete. Aluminum reacts with the alkaline constituents of the cement and generates hydrogen gas. The hydrogen gas will cause expansion of the mortar and reduce the concrete's compressive strength.

Aluminum base plates should be separated from concrete foundations by mounting the base plate on galvanized steel anchor bolts and leveling nuts above the concrete or by placing a galvanized steel plate between the aluminum plate and the concrete.

Where the surface of concrete in contact with aluminum is subjected to moisture entrapment, the aluminum surface should be treated at the installation site as specified in Section 6.5.3.2.

6.6 WOOD STRUCTURES

6.6.1 Ultimate Strength Design

Wood structures and poles should be designed and constructed in accordance with IEEE 751 (1991) and NESC (2007). IEEE 751 (1991) describes a probabilistic and a deterministic method for designing wood structures. ANSI-O5.1 (2002) and O5.1c (2004) can be used for wood pole stresses with the NESC (2007) 0.65 strength factor. Additional design information can be found in NDS (2005).

6.6.2 Allowable Stress Design

Wood structures and poles should be designed and constructed using NDS (2005).

6.7 SEISMIC DESIGN GUIDELINES

Seismic events can interrupt the delivery of power in several ways. Low-level ground shaking may trip equipment relays without any long-term damage and may require remote or manual reenergizing of relays. Moderate-level ground shaking may cause minor reparable equipment damage. Major-level ground shaking may cause equipment damage, destruction, or both.

Each electrical installation (substation or switchyard) should be evaluated based on its relative criticality to the owner's power system. Installations or specific equipment defined as critical or essential are those that are vital to power delivery and cannot be bypassed in the system or are undesirable to lose because of economic effects. Equipment that can be bypassed for short-term emergency operations is considered nonessential.

The designers of structures should consider the functional equipment needs with respect to calculated elastic displacements, material stresses, and plastic deformation. Functional needs include mechanical operations, such as opening and closing linkages, in addition to electrical functions, such as power circuit-breaker internal operation.

Displacements caused by the seismic events, either temporary or permanent, among components of different seismic response potential should not impair the performance of the mounted equipment, cause secondary induced stress, reduce the required electrical clearance, or cause other safety hazards.

Connections between equipment and components and their effects on one another require specific attention. Rigid electrical bus connections between equipment that restricts seismic-induced displacements may cause equipment damage.

All components should be designed to withstand stresses caused by the earthquake loading in Section 3.1.7, calculated by modal analysis or measured during seismic qualification testing. For static analysis, vertical accelerations equal to 80% of the horizontal acceleration should be combined with the horizontal acceleration in a direction that produces the most severe stresses.

For lattice dead-end structures that do not support electrical equipment, wind, ice, and wire tension loads usually control the design, when compared to earthquake loads. One situation where this may not be the case is when the dead-end structure supports the full transmission line wire tensions and the wind load effects are smaller than the seismic load effects. There has not been a documented case of failure for these types of structures because of earthquake loads.

Additional recommendations for seismic design of rigid bus systems are provided in Section 6.9.9.

6.7.1 Structures That Support Electrical Equipment

Seismic design of structures that support electrical equipment should satisfy the requirements of IEEE 693 (2005). A structure defined by IEEE 693 (2005) as a *first support* is the structural element on which the equipment is mounted. The first support can be a pedestal supporting a cantilever-type piece of equipment, such as a surge arrester. The first support can also be a structural member (component) within a support structure. An example of a structural member first support is a beam that the equipment, such as a reactor, is mounted on in a steel rack, box-type, structure. In this case, the equipment and its first support (i.e., the beam on which the equipment is mounted) is designed and qualified according to IEEE 693 (2005). The steel rack structure, as a complete system with the equipment and first support, is designed according to the requirements of this substation structure design manual. An R value of 1.0 is recommended for equipment reactions, obtained from an IEEE 693 (2005) qualification, applied to the first support for the design of the supporting structure (e.g., the rack, pedestal, or lattice structure). All other loads on this structure can use the appropriate R values as recommended in Section 3.1.7.3 of Chapter 3.

6.7.2 Structures Not Covered by IEEE 693

Structures not included in IEEE 693 (2005) include dead-end structures, rigid bus structures, strain bus structures, cable bus structures, and shielding masts. These structures should be designed to withstand the stresses using the equivalent lateral force procedure in Section 3.1.7.

6.7.3 Member Connections

Member connection bolts should not be less than 0.5 in. (13 mm) in diameter.

For USD, the stresses for connection bolts should be limited to the yield strength or the proof load of the bolt. For welds, the stresses should be as specified in ASCE/SEI 48 (2005).

6.7.4 Shake Table Testing

Shake table testing for equipment and their supports should be in accordance with IEEE 693 (2005). The equipment and equipment supports should be mounted during the test in a manner that simulates the intended service mounting.

6.7.5 Modal Analysis of Structures

The structure should be modeled as an assemblage of discrete structural elements interconnected at a finite number of points called nodes (for finite element analysis). The number and location of elements and nodes should be such that an adequate representation of the real system is obtained. The models should represent the equipment and structure system as it is mounted in service. The maximum modal response should be determined using an input motion described using the response spectrum in Section 5.6.2.1 or a site-specific response spectra, or the response spectra calculated using S_{DS} and S_{D1} in Section 3.1.7 of Chapter 3.

IEEE 693 (2005) and FEMA 450 (2004) provide requirements for determining these response spectra. The design engineer is responsible for selecting the appropriate design response spectrum.

6.7.6 Ultimate Strength Design

The stresses in members and connections should be in accordance with Section 6.3.1.

6.7.7 Allowable Stress Design

The stresses in members and connections should be in accordance with Section 6.3.2.

6.8 BASE PLATE DESIGN

This section provides a method to determine the plate thickness for a base plate on leveling nuts. It is conservative to use this procedure for base plates mounted on concrete. Although the design of anchor bolts is discussed in Chapter 7, it is important to know that the number of anchor bolts will affect the determination of the base plate thickness. In general, a greater number of small bolts will allow the use of thinner base plates than a fewer number of larger bolts. However, when the total installed cost of the foundation and anchor bolts is considered, a slightly thicker base plate with fewer bolts may prove to be the most economical choice because of reduced construction costs. ASCE/SEI 48 (2005) also contains methods for base plate design.

Figure 6-1 shows some base plate connections that may be used in substation structures.

6.8.1 Determination of Bolt Loads

The anchor bolt setting plan is determined by the geometry of the column, the loads imposed on the column, and the proper clearance between the nuts and the column. Assuming that the base plate behaves as an infinitely rigid body, the load in bolt i, BL_i can be calculated by the following formula:

$$BL_i = \left(\frac{P}{A_{BC}} + \frac{M_x y_i}{I_{BCx}} + \frac{M_y x_i}{I_{BCy}} \right) A_i \quad \text{(Eq. 6-3)}$$

where

P = total vertical load at the base of the column;

M_x = the base moment about the x axis;

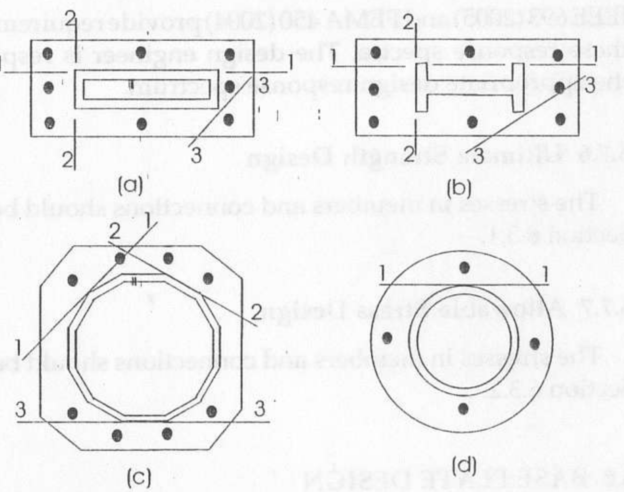


FIGURE 6-1. Examples of Base Plates.

M_y = the base moment about the y axis;

x_i, y_i = x and y distances of bolt i from reference axes;

A_i = the net area of bolt i;

$A_{BC} = \sum_{i=1}^{n} A_i$ = the total bolt cage area;

$I_{BCx} = \sum_{i=1}^{n} (A_i y_i^2 + I_i)$ = the bolt cage inertia about the x axis;

$I_{BCy} = \sum_{i=1}^{n} (A_i x_i^2 + I_i)$ = the bolt cage inertia about the y axis;

n = the total number of bolts; and

I_i = the moment of inertia of bolt i.

Because I_i is often small, it may be omitted when calculating I_{BCx} and I_{BCy}. Figure 6-2 illustrates the application of Eq. 6-3 to determine bolt loads.

6.8.2 Determination of Base Plate Thickness

A common design procedure for base plates assumes that bolt loads produce a uniform bending stress, F_b, along the effective portion of bend lines located at the face of the column. Each bend line is characterized by the following:

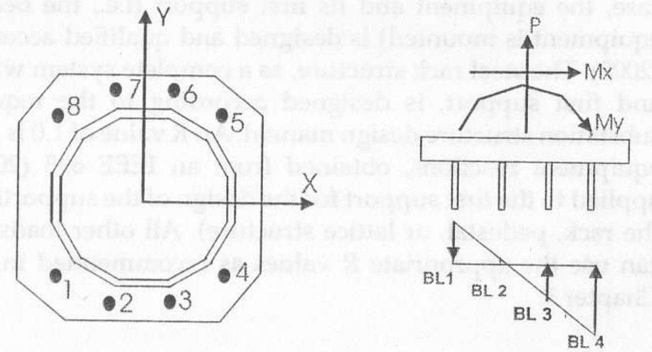


FIGURE 6-2. Bolt Load Calculation.

- k = the number of bolt load BL_i's contributing moment along the bend line;
- c_i = the shortest distance from bolt i to the bend line; and
- b_{eff} = the length of the bend line (depending on the shape of the column, the shape of the base plate, and k).

Figure 6-1 suggests some possible bend lines in various types of base plates. The most difficult task in the analysis of the base plate is the determination of the proper effective lengths of bend lines (b_{eff}) used to calculate the bending stress. Many times, b_{eff} should be limited to ensure that the bend line will be effectively loaded. One method to calculate b_{eff} assumes that the anchor bolt reactions are resisted at a bend line that is tangential to the column. The effective length of this bend line (b_{eff}) is assumed to be limited by the distance between the projected length of the first and last bolt acting on the bend line plus the sum of the perpendicular distances from these extreme bolts to the bend line. Manufacturers have used similar methods for many years and have had successful verification of this approach through full-scale testing. ASCE/SEI 48 (2005) can also be used to design base plates.

The bending stress F_{PL} for an assumed bend line can be calculated by the formula

$$F_{PL} = \left(\frac{6}{b_{eff} t^2}\right)(BL_1 c_1 + BL_2 c_2 + \ldots + BL_k c_k) \quad \text{(Eq. 6-4)}$$

where t is the base plate thickness. The minimum base plate thickness is determined by keeping F_{PL} below the yield stress F_y (if the USD method is used) or the allowable stress F_b (if the ASD method is used). Equation 6-4 can be rewritten as

$$t_{\min} = \sqrt{\left(\frac{6}{b_{\text{eff}}(F_y \text{ or } F_b)}\right)(BL_1c_1 + BL_2c_2 + \ldots + BL_kc_k)} \quad \text{(Eq. 6-5)}$$

6.9 RIGID BUS DESIGN

Rigid bus design should be approached as a system requiring both an electrical engineer and a design engineer. The electrical engineer should be responsible for selecting the electrical parameters, including the minimum size bus required for ampacity (current-carrying capacity), insulators, hardware, electrical clearances, and determining the short-circuit fault current. The design engineer should be responsible for selecting support locations and structural analysis and design of the rigid bus conductors, insulators, and support structures. Ultimate strength design is the preferred method for rigid bus design. Additional design considerations for rigid bus conductors can be found in IEEE 605 (2006).

6.9.1 Structural Analysis

A-frames often connect a high bus conductor to a low bus conductor. The connection should be made using flexible cables. This connection enables a more simplified analysis of each bus conductor (high and low) individually. If flexible cables are not used, the entire system of rigid bus conductors, insulators, and structural supports should be modeled. The short-circuit force, when applicable, should be applied simultaneously in two directions.

For multiple structures supporting continuous spans of rigid bus conductors, the design of the insulator and structure may be modeled and designed as an entire system of structures, rather than designed as a typical single structure. For a continuous-beam model, the horizontal and vertical deflection of the bus conductors results in smaller deflections, whereas the bus wind loads on the insulators are higher.

The design engineer should determine boundary conditions for bus fittings and determine whether a simple or complex analysis is required.

6.9.2 Structure Design

Use the structural loads from Chapter 3 and design the structure in accordance with Section 6.3.1.

6.9.3 Rigid Bus Shapes and Materials

Round tubular shapes are considerably more rigid than other structural shapes of the same ampacity. These shapes are efficient structurally and electrically, and their larger diameter helps to minimize corona at higher voltages.

A round tubular bus shape is applicable at all voltage levels. Other structural shapes, such as uniform angle bus conductor (UABC) and integral web channel bus (IWCB) are sometimes used at voltages under 138 kV.

Copper and aluminum are the two materials that have been historically used for bus conductors. Aluminum is more economical. Copper bus has been used in high-current situations or when working with an existing copper bus conductor. Properties of common bus materials can be found in IEEE 605 (2006).

Materials for which chemical composition and minimum physical properties are not supplied should not be used (e.g., structural tubes with welded seams).

The strength capacity of rigid bus conductors should be based on first yield of the cross section, unless limited by deflection criteria or instability of the element. Welding of aluminum results in a reduction of strength properties in the zone close to the weld (see Section 6.10.5.3). The strength resistance factor is 1.0 for USD.

6.9.4 Porcelain Station Post Insulators

Porcelain station post insulators are assemblages generally consisting of end fittings, a bonding medium, and a porcelain body. Whereas all of the components of the insulator can be sources of failure, the focus of most investigations is the porcelain body.

Porcelain insulators are manufactured 1 to 3 standard deviations above the rated strength. The porcelain insulator is coated with glaze, approximately 3 mils (7.62 mm) thick, to obtain the rated cantilever strength. Glaze has a higher coefficient of thermal expansion and preloads the porcelain in compression. Compression strength is much higher than the tensile strength. Typically, porcelain station post insulators have relatively good axial compression characteristics in comparison to the cantilever strength. Accordingly, bending strength is normally a significant design parameter.

For bending strength, manufacturers specify a cantilever rating. The cantilever rating is the maximum horizontal load that can be applied at the top of the insulator with the insulator base fixed. Manufacturers typically recommend multiplying the cantilever rating by a strength resistance factor of 0.4 for working or allowable loads. The moment capacity at the base of the insulator is equal to the product of the strength resistance factor, the cantilever rating, and the insulator height. The insulator height does not include the distance between the top of the insulator and the centerline of the bus conductor.

The strength resistance factor of 0.4 should be increased to provide similar insulator selections between ultimate strength and working strength (allowable stress) designs. A strength resistance factor of 0.5 is recommended for ultimate strength design. This factor is 25% higher than the 0.4 strength resistance factor and, when used with recommended load factors, should provide similar insulator selections.

6.9.5 Composite Station Post Insulators

The materials that provide the structural strength of composite insulators are much different from porcelain, but more importantly, the failure mechanism is different for composite and porcelain insulators. Therefore, the design criteria for composite station posts are different from those for porcelain. The two major reasons for this are the following:

1. Composite insulators are much more ductile than porcelain insulators.
2. Composite insulators have a fiberglass core and elastomer sheds. The strength is provided by the fiberglass core. Porcelain insulators have a porcelain core and porcelain sheds. The strength of porcelain is isotropic, whereas composite's strength depends on the orientation of the glass fibers in the fiberglass. The glass fibers may be longitudinal or in a woven pattern.

Two major ratings are provided for composites. They are the specified mechanical load (SML) and the maximum mechanical load (MML). For rigid bus station post insulators, these ratings are based on a cantilever load. The SML is the value at which failure begins and no damage is observed. The SML is used in conjunction with short-term ultimate loads. The MML is the value at which the fiberglass remains in the elastic range and can sustain this value for long duration loads. Fifty percent of the SML is used for allowable stress design (ASD). Fifty percent of the SML value is used for ultimate strength design (USD), with a strength resistance factor of 1.0. Annex R of IEEE 693 (2005) provides additional information on composite insulators.

6.9.6 Couplers

Internal welded bus couplers consist of a short tube inside of the rigid bus conductors, which permits the connection of individual bus lengths

by welding. The design stress level should be reduced within 1 in. (2.54 cm) of the weld. Whenever possible, the rigid bus coupler should be located at a low-stress location along the bus. If a coupler can obtain 100% capacity of the rigid bus, the rigid bus splice can be located anywhere along the bus.

Currently, couplers are selected from catalogs that may not give dimensional data or alloy type. The design engineer should contact the manufacturer to obtain the alloy and dimensional properties of the selected coupler (see Section 6.9.3).

6.9.7 Fittings

Bolted-type bus support fittings permit easier installation than welded-type fittings and do not require a reduction in stress level because of welding. However, welded-type fittings provide a better path for current flow.

6.9.8 Thermal Expansion Considerations

The length of a bus conductor changes when its temperature changes. The coefficient of thermal expansion for aluminum is 0.0000128 per °F (0.000023 per °C). Most manufacturers specify the gap dimension for a 68 °F (20 °C) installation temperature. The expansion-fitting gap, *D*, Fig. 6-3, should be selected for an operating temperature range determined by the utility's engineer, considering the variations and tolerances in the bus and fittings. During a fault current, the operating temperature could reach 200 °F (93 °C). The gap may have to be offset to the right or left of the center of the expansion fitting if the bus lengths on either side are not equal.

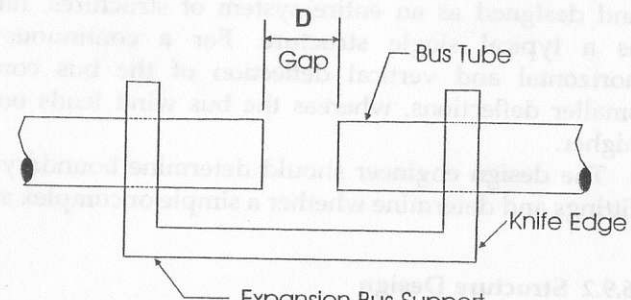


FIGURE 6-3. Rigid Bus Expansion-Fitting Example.

6.9.9 Rigid Bus Seismic Considerations

The following seismic issues should be addressed:

1. Expansion fittings may create impact loads because the thermal expansion-fitting gap is too small to allow for relative displacements at the top of insulators. Conductors may also pull out of the expansion fittings during an earthquake.
2. Cast bus fittings may be brittle and lack sufficient strength to resist earthquake loads. Forged fittings should be considered.
3. Depending on boundary conditions at the top of the insulator and the direction of the earthquake loads, moments may be developed at the top of the insulator.
4. If a segment of conductor is supported with fixed and slip fittings and the ground acceleration is parallel to the conductor, the insulators and supports connected to the fixed fittings require sufficient strength to resist the earthquake force created by the entire mass of the conductor segment.
5. Additional porcelain strength is obtained by increasing the cross-sectional size. High-strength insulators have a corresponding increase in weight, which for earthquake loads may offset the increase in strength.
6. Flexible connections between rigid bus conductors and electrical equipment should be considered to reduce the transfer of seismic forces. Information on flexible bus connections can be found in IEEE 1527 (2006) and IEEE 605 (2006).
7. Catenary-hung flexible conductors (strain bus or jumper), used in place of rigid bus conductors, can generate significant dynamic loads during an earthquake. These dynamic loads can exceed the static loads.

6.9.10 Rigid Bus Vibration Considerations

Rigid bus systems may be subject to vibratory loads, which, in conjunction with a state of stress, can lead to failure or damage. The oscillating loads tend to be characteristic of alternating current systems and may be generated by the bus itself or by the equipment to which it is attached. Because of the time dependency of such failures, the nominal steady-state operating condition is typically of concern.

It is considered good practice to decouple rigid bus conductors from known sources of vibration (e.g., properly designed flexible connectors), to avoid details and fabrication methods that are conducive to high-stress concentration factors, and to design so as to minimize loads at nominal conditions.

Rigid bus systems can also be susceptible to wind-induced vibration. A stranded aluminum conductor is sometimes placed inside the bus tube to help dampen these vibrations. IEEE 605 (2006) provides guidance concerning vibrations induced by wind and alternating currents.

6.10 SPECIAL DESIGN CONSIDERATIONS

When designing a substation structure, the design engineer may encounter some special situations that are unique to utility structures. This section provides some guidance on these areas.

6.10.1 Structures for Air Core Reactors

The magnetic field of an air core reactor induces currents in adjacent metallic structures or components that can cause heating under normal conditions and forces under short-circuit conditions. This heating does not adversely affect the reactors or inductors but may lead to degradation of adjacent equipment or structures and may possibly create a safety problem by making structures too hot to touch.

Air core reactors are usually mounted on aluminum, fiberglass, or reinforced-concrete structures. The following criteria are recommended for design of these structures. The manufacturer's recommendations should always be considered.

1. For concrete structures, use fiberglass reinforcement within $D/2$ of the reactor (D is the diameter of the reactor) and nonmagnetic stainless steel anchor bolts to connect the reactor insulators to the structure, when the anchor bolts are within $D/2$ of the reactor. When the anchor bolts are outside of $D/2$ of the reactor, steel anchor bolts may be used. Unlike steel reinforcement, fiberglass does not possess a plateau in the stress–strain curve and should be designed using an allowable stress design method. ACI 440.1R (2006) provides guidance on design with fiberglass reinforcing.
2. Aluminum structures should not be within $D/2$ of the reactor.
3. Closed loops in metal should not be within D of the reactor. A closed loop is any metal structure that allows circulating currents to flow in the outer area. Examples of closed loops are chain link fences, metal structures connected to the ground mat, and two-way slab reinforcement that is not separated by nonmagnetic material in concrete foundations. If closed loops are within D of the reactor, the manufacturer should be consulted for recommendations.
4. Closed loops can be broken by taping the crossing touching parts with electrical or other nonconductive tapes.

5. Care should be taken when components made from dissimilar metals come into contact with one another.

If the air core reactor is not mounted on a structure, the reactor should be enclosed by a fence for personnel safety.

6.10.2 Wind-Induced Vortex Shedding

Wind-induced vortex shedding occurs when turbulent vortices are shed alternately from one side, then the other side, of a tubular member. As each vortex is shed, it pulls the member to that side. The result is alternating forces perpendicular to the wind direction.

Vortex shedding requires relatively steady or laminar winds to produce sustained oscillations. Normally, the amplitude of these oscillations is small, but it may be greatly increased when the frequency of the vortex-shedding oscillations is close to one of the natural frequencies of the structure. When this occurs, the structure is susceptible to fatigue failure. The fatigue failure should occur at locations with the greatest rigidity, which are generally the member connections.

Long, slender structures can have a natural frequency that can be excited by winds of approximately 15 mile/h (6.71 m/s). These wind speeds can be expected almost anywhere. However, only isolated cases of vortex excitation of substation structures has been reported. This fact is probably because conductors, insulators, and ground wires significantly increase the mechanical damping and the wind is too turbulent for periodic vortex formation.

Analytical procedures for calculating structure response to vortex excitation are not yet practical for design use.

There are three approaches to controlling vortex excitation of strain bus structures and ground masts:

1. Increase the stiffness of the member. This can be accomplished by using larger sections, bracing between members, or adding guy wires.
2. Increase the damping of the member. This can be accomplished by hanging a chain inside a vertical member that has a weight approximately equal to 5% of the member weight.
3. Add spiral strakes along the length of a member. These strakes suppress the periodicity of the vortex formation and reduce the correlation between the aerodynamic forces acting along the length of the member. The pitch, spacing, number, and length of spoilers along the member's length are derived from wind tunnel tests and are contained in Scruton (1963) and ASCE (1961).

6.10.3 Galvanizing Steel Considerations

Members and connections should be designed and detailed to allow for proper drainage and venting during the galvanizing process. The American Galvanizer's Association has two pamphlets (1984 and 2001), which show proper design and detailing practices. ASTM A123/A123M (2002) and ASCE/SEI 48 (2005) should also be consulted.

Tubular structures present special challenges for galvanizing, and failure to address proper design and detailing for galvanizing practices may lead to structural failures because of internal corrosion. In addition to the drainage and venting requirements in the above references, the design engineer should avoid the design of complex built-up members with interior stiffeners that impede the free flow of molten zinc and make interior inspection difficult or impossible. Preferably, all members should be able to be galvanized in one single dip. If it is impossible to do a single dip, the design engineer should ensure that the fabricator and galvanizer are qualified to use the double-dip process without flux becoming entrapped beneath a layer of zinc in the double-dipped area.

6.10.4 Painted or Metallized Steel Considerations

Tubular steel members to be painted or metallized should have their interiors sealed because there is usually no effective method to apply the surface coating to the interior of the member or to inspect it once it is in place.

6.10.5 Member Connection Design

The design of the connections between members is an important part of the overall structure design. The external forces on the structure should be transferred through the members by axial, shear, torsion, and moment forces into the foundation with properly designed connections that mirror the assumptions made in the theoretical model. If a member is assumed to be fixed against rotation, the connection should be rigid enough to provide rotational stiffness.

6.10.5.1 Bolted Connections in Steel. When bolts are used to connect members, they should be placed in shear or in tension. Placing the bolt shaft in bending should be avoided. The section modulus of the threaded shaft is small and that can result in high bending stresses. If tension and shear are combined in a connection, interaction equations should be followed in the selection of the bolts. AISC (2005), which covers both ASD and LRFD, can be used.

6.10.5.2 Welded Connections in Steel. Members are usually connected by combinations of flat plates welded together. These flat plates are sized for weak axis bending after finding the appropriate bending plane and plate bending moment. In Blodgett (1966), Section 6.6-2 limits the effective plate-bending plane for weak axis bending due to a point load to a value of 12 times the plate thickness. AISC (2005), which covers both ASD and LRFD, can also be used.

6.10.5.3 Welded Connections in Aluminum. Care should be taken when designing welded connections for aluminum alloy structures. The heat-affected zone of the weld can have allowable stresses that are about one half that of a properly heat-treated alloy. The heat-affected zone of the weld can possibly revert to the stress allowable of the untempered aluminum alloy if the welding is not done properly. If at all possible, the welded joints should be heat-treated after welding to restore the strength properties lost in the welding process. Additional information on welding aluminum is available in AWS D1.2/D1.2M (2003).

6.10.5.4 Concrete Structure Connections. Substation structures can be made of precast or cast-in-place concrete elements. Bolts are generally used when members other than concrete are to be connected to a concrete member or when two precast concrete members are to be connected to each other. If a new member is to be installed on an existing concrete structure using concrete anchors, the anchor manufacturer's specifications and limitations should be followed.

When connecting two or more concrete members that are not one continuous placement of concrete, the concrete elements can be connected by welding embedded angles or plates. The embedded angles and plates should be designed to develop the connection loads through proper concrete embedment. Care should be used when welding is done on these embedded steel angles to prevent the spalling, or splintering, of concrete.

6.10.5.5 Connections in Wood Structures. The connections designed for wood structures should follow the guidelines in IEEE 751 (1991). Additional design information can be found in NDS (2005).

6.10.6 Weathering Steel Structures

ASTM A242/A242M (2004), ASTM A588/A588M (2005), and ASTM A871/A871M (2003) state that atmospheric, corrosion-resistant, high-strength, low-alloy steels can be used uncoated in most atmospheres. As the bare steel is exposed to the normal atmosphere, a tightly adherent oxide layer forms on the surface, which protects the steel from further corrosion. The proper design, detailing, fabrication, erection, and maintenance practices for the application of such steels should be observed to achieve the benefits of the enhanced atmospheric corrosion resistance property.

The oxides from the weathering steel structures may be deposited, probably because of the water runoff, on the underhung support insulators. There is no direct evidence that this staining will provoke an electrical flashover. However, the stain can roughen the insulator surface and elevate the level of contamination accumulation. Water on the stained surface also tends to remain longer than on unstained insulators. These, combined with other meteorological factors, such as coastal salt spray and abundant humidity, may create a favorable condition for electrical flashover. It may be advisable to increase the leakage distance for the insulators to prevent such an incidence.

The design and detailing of weathering steel connections should be done in accordance with ASCE/SEI 48 (2005). Large edge distances and bolt spacing can cause joint failure over long periods of time when the two connecting surfaces become filled with products of corrosion and expand.

6.10.7 Guyed Substation Structures

Guyed structures are infrequently used for substation structures. Substation yards typically have limited space for positioning guys. A geometric nonlinear analysis should be used for such structures to simulate accurately the forces in the members and guy (see Chapter 5). Particular care should be used when checking for overall structure buckling. Local plate buckling may be a problem on hollow polygonal compression members. ASCE 91 (1997) provides guidance for the design of this structure type.

CHAPTER 7
CONNECTIONS TO FOUNDATIONS

The many types of electrical substation structures have a wide range of ground line reactions. The foundations used depend on the different types of soil present as well as the design engineer's preferences. These foundations can be slabs on grade, spread footings, drilled shafts, or piling with pile caps. The foundation should be designed to transmit the structure and equipment forces to the supporting soil or rock. These forces include bearing, uplift, shear, and overturning moment. The foundation design should include factors of safety as determined by the design engineer for all loads. Buried structural steel within the substation yard should have appropriate design provisions for corrosion and electrical grounding.

Different types of anchorage are used to connect substation structures to their foundations. The most common means of transferring structure reactions to the foundations is by anchor bolts. This type of anchorage provides a good transfer of load. Anchor bolts can be headed bolts or a straight length of deformed reinforcing bar. Cast-in-place headed bolts are the recommended anchor bolt type, over hook bolts. Headed anchors can be designed to provide a ductile failure mode, whereas hook bolts have been reported to fail by anchor pullout in a brittle mode. Headed anchor bolts confined by the foundation reinforcement increase the structural capacity of the anchorage system.

Stub angles and direct-embedded structures can also be used in substations; these methods of transferring loads from the foundation to the structure to the foundation are adequately addressed in ASCE 10 (1997) and ASCE/SEI 48 (2005). Under certain conditions, anchorage may need to be installed in existing foundations (postinstalled) using drilled concrete anchors.

The approach for the design of anchor bolts in this chapter is based on ultimate strength design (USD). Design loads should include applicable load factors. All equipment anchorage assemblies, including anchor bolts, should be designed for loads resulting from the analysis or from tests, whichever is applicable.

7.1 ANCHOR MATERIALS

ASTM F1554 (2004) and bolts manufactured from ASTM A36/A36M (2005) steel rods (with nuts acting as the bolt head) are recommended for foundation anchor bolts. This type of steel is readily available, cost-effective, and ductile.

The term "anchor bolt" is used in this manual to describe the connection of the structure to the foundation. The latest AISC (2005) steel manual has adopted the term "anchor rod" to describe this item, which is a headed rod cast into the concrete foundation. However, the term "anchor bolt" is used in ASTM F1554 (2004) and throughout the industry.

The use of ductile anchorage is especially critical in seismic areas where larger, ductile anchor bolts are preferred to smaller, less ductile, high-strength anchor bolts. Accordingly, it is recommended that anchorages for seismic applications use ASTM A36/A36M (2005) steel material.

The material used for anchor bolts made from deformed reinforcing bars should be in accordance with ASTM A615/A615M (2006) or ASTM A706/A706M (2006). When high-tensile-strength, deformed reinforcing bars are used, they should be obtained with a minimum Charpy-V notch requirement of 15 ft-lb (20.3 N-m) at −20 °F (−28.9 °C) when tested in the longitudinal direction. Only one grade of bar should be used for each diameter to prevent the possibility of errors in the field during construction.

Table 7-1 shows the properties of the recommended anchor bolt materials. Other anchor bolt material may be used when higher strength

TABLE 7-1. Anchor Material Properties

Material ASTM	Yield Strength F_y (kip/in.^{2})	Ultimate Strength F_u (kip/in.^{2})
ASTM A36/A36M (2005)	36	58
ASTM F1554 (2004), Grade 36	36	58–80
ASTM F1554 (2004), Grade 55	55	75–95
ASTM F1554 (2004), Grade 105	105	125–150
ASTM A615/A615M (2006), Grade 60	60	90
ASTM A615/A615M (2006), Grade 75	75	100
ASTM A706/A706M (2006), Grade 60 (weldable)	60	80

Note: 1 kip/in.^{2} = 6.89 MPa.

is required, such as ASTM A449 (2004), or other bolting material that will provide acceptable ductile behavior. When high-strength bolts are used, they should be suitable for galvanizing. Anchor bolts should not be less than 0.75 in. (1.91 cm) in diameter, unless the allowable stresses exceed the applied stresses by not less than a factor of 2.

Other materials with higher strengths or other desirable properties may be used when circumstances dictate, but the material supplier should be consulted for availability, compatibility, and any special design considerations associated with that material.

Steel anchorage used in an outdoor substation application should have a protective coating applied to resist corrosion. The bolt projection plus a minimum of 6 in. (15.24 cm) should be hot-dip galvanized in accordance with ASTM A153/A153M (2005) or ASTM B695 (2004) for Class 50, Type I.

7.2 ANCHOR ARRANGEMENTS

The two most commonly used arrangements to anchor a structure to the foundation are base plate supported by anchor bolts with leveling nuts (Fig. 7-1), and anchor bolts with the base plates on concrete or grout (Fig. 7-2).

Washers or plates can be used to ensure adequate load transfer from the nut to the leveling plate.

7.2.1 Base Plates Supported by Anchor Bolts with Leveling Nuts

The use of leveling nuts to support substation structures has several advantages. This method eliminates the need for close tolerance work on

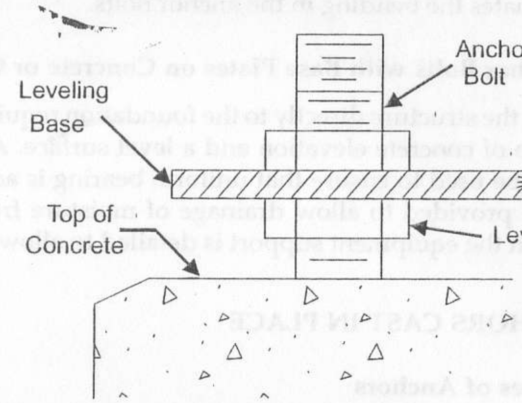


FIGURE 7-1. *A Base Plate Supported by Anchor Bolts with Leveling Nuts.*

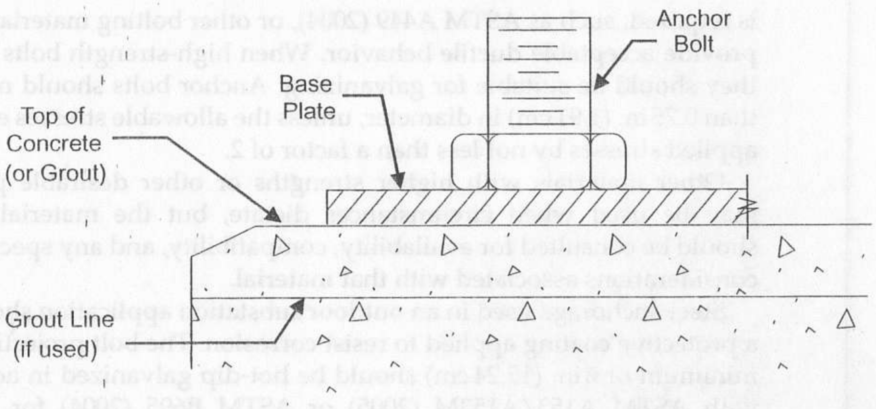


FIGURE 7-2. *Anchor Bolts with a Base Plate on Concrete or Grout.*

the foundation elevation and trueness of surface while providing flexibility for the structure alignment because of fabrication tolerances and bus erection. Leveling nuts also keep the base plate from resting in any standing water on the foundation. The disadvantage of using a leveling base is that the shear and axial loads have to be resisted entirely by the anchor bolts. These combined stresses can require larger anchor bolts and thicker base plates for heavily loaded structures. Therefore, in some cases the leveling base arrangement may not be the best alternative.

If the leveling base and anchor bolts have been designed to support the base plate on the leveling nuts, then it is not necessary to install nonshrink grout under the base plate. If grout is installed, the design procedures in Section 7.2.2 may be used. The use of grout is beneficial in cases where large shear loads are present, such as in seismically active areas. The grout provides a better shear transfer between the base plate and the foundation and eliminates the bending in the anchor bolts.

7.2.2 Anchor Bolts with Base Plates on Concrete or Grout

Bolting the structure directly to the foundation requires close tolerances for the top of concrete elevation and a level surface. A bed of nonshrink grout can be used to ensure that uniform bearing is achieved. Provisions should be provided to allow drainage of moisture from under the base plate when the equipment support is detailed to allow such drainage.

7.3 ANCHORS CAST IN PLACE

7.3.1 Types of Anchors

The three types of anchors most commonly used are headed bolts, deformed reinforcing bar bolts, and hook bolts (Fig. 7-3).

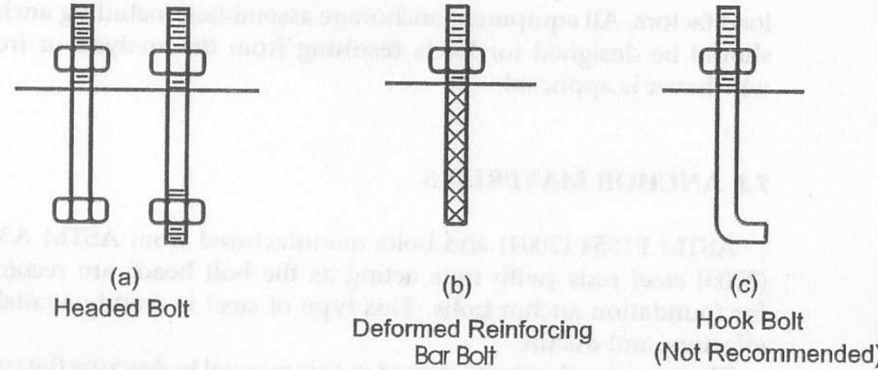


FIGURE 7-3. *Types of Anchors.*

7.3.1.1 Headed Bolts. The preferred method for anchor bolt systems is the use of headed anchors. These bolts include those with a bolt head or nut where tensile forces are resisted by a pullout shear cone. Embedded steel plates instead of a bolt head or nut is generally not recommended because it tends to create an inherently weak plane, which in turn reduces the capacity. This type of bolt requires adequate edge distance to prevent pullout or lateral bursting failure. When nuts are used at the bottom of a threaded rod, it may be necessary to consider methods to prevent rotation of the embedded nut during construction. Three methods for preventing rotation are deforming the bottom threads, pressing the bottom nut, and tack welding the bottom interface between the nut and threads (not recommended with high-tensile-strength bars, > 100 kip/in.^{2}). Care should be taken to make sure that concrete has set adequately before loosening or tightening top nuts because the head can easily be turned in freshly placed concrete.

7.3.1.2 Deformed Reinforcing Bar Bolts. Deformed reinforcing bars used as anchor bolts are typically No. 18 jumbo bars that are larger in diameter and heavier than the standard No. 18 reinforcing bar specified in ASTM A615/A615M (2006) or ASTM A706/A706M (2006). The larger diameter allows for full-depth threads to be cut into the deformed bar. The tensile forces are transmitted to the concrete through shear developed between the bar deformations and the concrete. These bolts are typically used for anchoring structures with large loads.

7.3.1.3 Hook Bolts. Smooth-bar hook bolts are not recommended because of less predictable behavior in tension tests.

7.3.2 Design Considerations for Anchor Steel

The method used to determine the area of steel required to transfer the load from the structure base plate to the foundation depends on the base plate arrangement. One arrangement is the base plate bearing directly on the concrete or bed of nonshrink grout and held in place by nuts that are tightened on top of the base plate. This arrangement transfers a portion of the shear directly to the foundation through the friction between the base plate and the concrete. Another arrangement is the base plate supported by leveling nuts, which transfers the shear to the concrete by the side-bearing pressure of the anchor bolt.

7.3.2.1 Anchor Bolts with Base Plate on Concrete or Grout. The area of steel required for tension and shear are additive (ACI 349 2001). Therefore, the following equations are used to determine the required minimum tensile area of steel (A_s):

$$A_s = A_a + A_v \qquad \text{(Eq. 7-1)}$$

The area required for tension load (A_a) is determined by

$$A_a = \frac{P_u}{F_{dt}} \qquad \text{(Eq. 7-2)}$$

where

P_u = the ultimate tension load per bolt (in kips or kN);
F_{dt} = design tensile strength (in kip/in.^{2} or MPa); = ϕF_y or 0.8 F_{ut} (whichever is less);
ϕ = 0.9;
F_y = the minimum specified yield strength of anchor steel (in kip/in.^{2} or MPa); and
F_{ut} = the minimum specified tensile strength of anchor steel (in kip/in.^{2} or MPa).

The area required for the shear load (A_v) is determined by

$$A_v = \frac{V_u - (\mu)(P_{cm})}{[(\phi)(F_y)]} \text{ for compression} \qquad \text{(Eq. 7-3a)}$$

$$A_v = \frac{V_u}{[(\phi)(F_y)]} \text{ for uplift} \qquad \text{(Eq. 7-3b)}$$

where

V_u = anchor bolt shear (in kips or kN);
P_{cm} = the minimum compression load on the base plate (in kips or kN) (Note: If base plate is subjected to net uplift, friction is zero);
μ = the static coefficient of friction between the base plate and the concrete (Fig. 7-4):

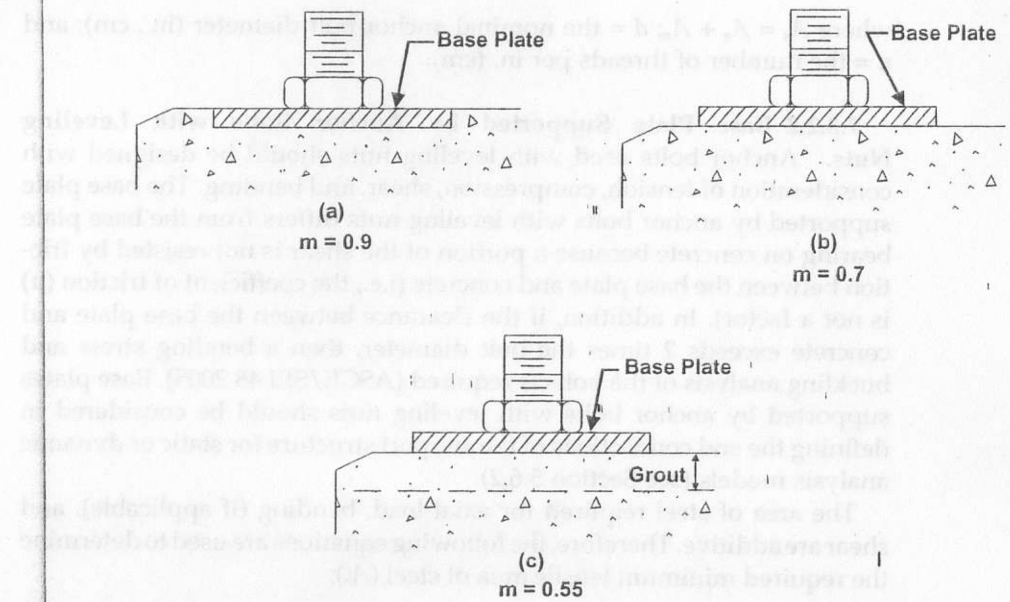


FIGURE 7-4. Coefficient of Friction (μ) Values for Various Conditions.

a. μ = 0.9 for concrete or grout against steel plate with the contact plane a full plate thickness below the concrete surface. The plate should remain in full contact with sound concrete during the service life of the equipment.
b. μ = 0.7 for concrete or grout placed against steel plate with the contact plane coincidental with the concrete surface.
c. μ = 0.55 for grouted conditions with the contact plane between the grout and the steel plate above the concrete surface.

ϕ = 0.85; and
F_y = minimum specified yield strength of anchor steel (in kip/in.^{2} or MPa).

The actual stress area (A'_s) for an anchor bolt, taking into account the area reduction from the threads, is determined by

$$A_s = \left(\frac{\pi}{4}\right)\left[d - \left(\frac{0.974}{n}\right)\right]^2 \qquad \text{(Eq. 7-4)}$$

Therefore, by setting $A'_s = A_s$, the required nominal anchor bolt diameter (d) can be determined by

$$d = (2)\left(\frac{A_s}{\pi}\right)^{1/2} + \left(\frac{0.974}{n}\right) \qquad \text{(Eq. 7-5)}$$

where $A_s = A_n + A_v$; d = the nominal anchor bolt diameter (in., cm); and n = the number of threads per in. (cm).

7.3.2.2 Base Plate Supported by Anchor Bolts with Leveling Nuts. Anchor bolts used with leveling nuts should be designed with consideration of tension, compression, shear, and bending. The base plate supported by anchor bolts with leveling nuts differs from the base plate bearing on concrete because a portion of the shear is not resisted by friction between the base plate and concrete (i.e., the coefficient of friction (μ) is not a factor). In addition, if the clearance between the base plate and concrete exceeds 2 times the bolt diameter, then a bending stress and buckling analysis of the bolts is required (ASCE/SEI 48 2005). Base plates supported by anchor bolts with leveling nuts should be considered in defining the end connections of the support structure for static or dynamic analysis models (see Section 5.6.2).

The area of steel required for axial load, bending (if applicable), and shear are additive. Therefore, the following equations are used to determine the required minimum tensile area of steel (A_s):

$$A_s = A_a + A_b + A_v \quad \text{(Eq. 7-6)}$$

where A_a = the area required for axial load (tension or compression) (in.^{2}, cm^2), A_v = the area required for shear (in.^{2}, cm^2), and A_b = the area required for bending (in.^{2}, cm^2).

Include A_b in Eq. 7-6 only when the clearance between the bottom of the base plate and the concrete exceeds 2× the diameter of the bolt.

The required area for the axial load (A_a) is determined the same as shown in Section 7.3.2.1.

When the clearance between the base plate and the concrete exceeds 2× the bolt diameter, the shear can induce a significant bending moment that should be considered (Fig. 7-5a). The shear can be assumed to act on a rigid frame consisting of the base plate as the beam and the anchor bolts as the columns. Using moment distribution with typical relative stiffness ratios of column to beam gives a conservative point of inflection (zero moment) at a point approximately 0.625× the distance from the top of the concrete to the bottom of the base plate (Fig. 7-5b).

Therefore, the bending in the anchor bolts can be determined by the following equation:

$$M_u = \left(\frac{5}{8}\right)(h)(V_u) \quad \text{(Eq. 7-7)}$$

where

M_u = the ultimate bending moment per bolt (in.-kips, mm-kN);

h = the distance from the top of the concrete to the bottom of the base plate (in., mm); and

V_u = the ultimate shear per bolt (kips, MPa)

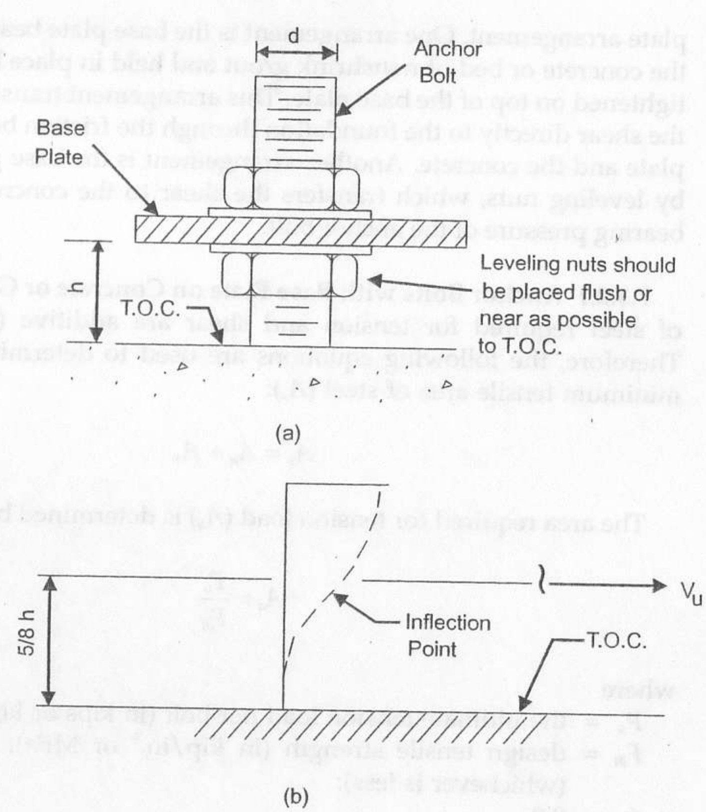


FIGURE 7-5. (a) Typical Leveling Nut Arrangement with (b) Rigid Frame Diagram.

The required area for bending (A_b) can be determined by

$$A_b = \left\{\pi\left[\frac{5hV_u}{2\phi F_y}\right]^2\right\}^{1/3} \quad \text{(Eq. 7-8)}$$

This equation is derived from the following relationships:

$$\phi F_y = \frac{M_u}{S_b}$$

where

ϕ = 0.9;

$M_u = \left(\frac{5}{8}\right)(h)(V_u)$; and

$S_b = (A_b^3/\pi)^{1/2}/4$

The required area for shear (A_v) is determined by

$A_v = V_u/[(\phi)(F_y)]$ and

$\phi = 0.85$.

The actual stress area (A'_s) for a bolt, taking into account the area reduction from the threads, is given by Eq. 7-4. Therefore, by setting $A_s = A'_s$, the required nominal bolt diameter (d) can be determined by Eq. 7-9:

$$d = (2)\left(\frac{A_s}{\pi}\right)^{1/2} + \left(\frac{0.974}{n}\right) \qquad \text{(Eq. 7-9)}$$

where

$A_s = A_a + A_b + A_v$;

d = the nominal bolt diameter (in., cm); and

n = the number of threads per in. (cm).

7.3.3 Design Considerations for Concrete

The capacity of a cast-in-place anchor bolt is limited by either the capacity of the steel anchor bolt or the concrete. In Section 7.3.2, the strength limitations of the anchor bolts were discussed. In this section, the concrete limitations are considered.

7.3.3.1 Tensile Capacity of Concrete. The procedure in Appendix D of ACI 318 (2005) is adequate for the type of structures covered by this manual. In ACI 318 (2005), Appendix D.4.2.2, limits on headed anchor bolt diameter are based on the extent of the test database that was used in developing the Appendix D provisions. Further testing is being done on large-diameter headed bolts, and the findings will be incorporated into future versions of ACI 318. Therefore, ACI 318 (2005), Appendix D, contains the recommended design procedure for determining the concrete tensile capacity of all anchor bolts using the loads and load combinations from this manual.

7.3.3.2 Design of Side Cover Distance for Tension. Appendix D of ACI 318 (2005) should be used with the loads and load combinations from this manual to determine the side cover distance for tension.

7.3.3.3 Design of Side Cover Distance for Shear. Appendix D of ACI 318 (2005) should be used with the loads and load combinations from this manual to determine the side cover distance for shear.

7.3.3.4 Anchor Bolt Embedment Length. The embedment for anchor bolts should resist pullout from the applied tension and at the same time transfer this tension to the foundation reinforcement.

The deformed bar anchor bolt resists pullout by the mechanical anchorage of the deformations, which prevent the longitudinal movement of the bar with respect to the concrete. Headed anchor bolts transfer the structure loads to the foundation by the bearing of the embedded nut or bolt head on the concrete. The design procedure used to determine the required anchor bolt embedment or development length is provided by ACI 318 (2005).

In addition to providing adequate anchor bolt embedment, the foundation vertical reinforcement should be developed by the anchor bolt, whether headed or deformed anchor bolts are used (Fig. 7-6). The required development length (l_d) of the vertical reinforcement in drilled shafts and spread footings is provided by ACI 318 (2005).

7.3.3.5 Concrete Punch Out from Anchor Bolts. When designing anchorages in reinforced concrete using headed smooth-bar anchor bolts, caution should be taken when the anchor bolts are in compression. This condition exists when the base plate of the structure rests on a leveling nut on the anchor bolt. The proper development length of the vertical reinforcing steel should be maintained both above and below the headed end of the anchor bolt. If headed anchor bolts are used in a reinforced concrete slab, the thickness of the slab should be great enough to prevent punch out through the bottom of the slab. For additional information, consult ACI 355.1 (1997) or ACI 318 (2005).

7.3.3.6 Localized Bearing Failure. Headed anchor bolts made from ASTM A36/A36M (2005) steel can usually be used without an

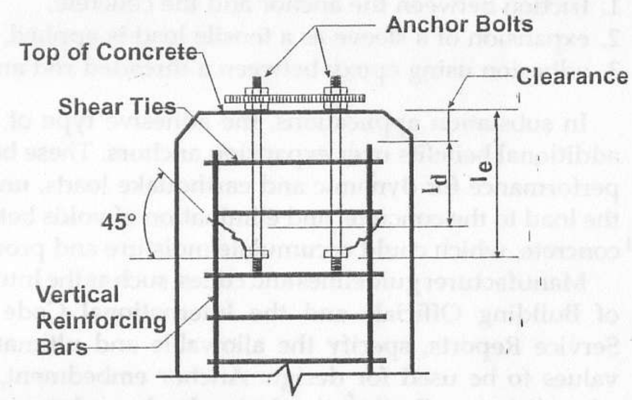


FIGURE 7-6. Development Length of Vertical Reinforcing Steel in Drilled Shafts and Spread Footings.

anchor-bearing plate, but higher strength bolt materials may need an anchor-bearing plate to avoid localized bearing failure. The following equation was adapted from ASCE (1997) and can be used to determine if an anchor plate is needed:

$$A_{plate} = A_{bolt} + A_s(0.11)\left(\frac{f_y}{f_c'}\right) \quad \text{(Eq 7-10)}$$

where

A_{plate} = the required plate or bolt head area (in.², cm²);
A_{bolt} = the nominal bolt shaft area (in.², cm²);
A_s = the tension area required for the anchor bolt from Eq. 7-6 (in.², cm²);
f_y = the bolt steel yield strength (kip/in.², MPa); and
f_c' = the concrete compressive strength (kip/in.², MPa).

If the calculated value for A_{plate} is smaller than the area of the nut or bolt head, then a bearing plate is not required.

7.4 DRILLED CONCRETE ANCHORS INSTALLED IN EXISTING CONCRETE

Drilled-in-concrete anchors are typically used in existing concrete for anchorage when lack of design information precludes use of anchor bolts in new construction, to correct improper anchorage placement during construction, and when structure loads are small. The basic principles used in these anchors are:

1. friction between the anchor and the concrete,
2. expansion of a sleeve as a tensile load is applied, and
3. adhesion using epoxy between a threaded rod and the concrete.

In substation applications, the adhesive type of anchorage has some additional benefits over expansion anchors. These benefits include better performance for dynamic and earthquake loads, uniform distribution of the load to the concrete, and elimination of voids between the anchor and concrete, which could accumulate moisture and promote corrosion.

Manufacturer guidelines and codes, such as the International Conference of Building Officials and the International Code Council Evaluation Service Reports, specify the allowable and ultimate shear and tension values to be used for design. Anchor embedment, group spacing, and edge distance all affect the design loads, and manufacturer's guidelines should be consulted to take these factors into account. Anchor bolts should be galvanized, coated, or stainless steel for use in outdoor applications in

the substation. In earthquake environments, the materials of the drilled-in anchorage should meet the recommendations of Section 7.1.

7.5 EXAMPLES

7.5.1 Base Plate on Concrete

Structure type: Line dead-end A-frame structure, 4 anchor bolts on 14-in. square bolt pattern, ASTM A36/A36M (2005) material ($F_{ut} = 58\,\text{kip/in.}^2$).

Applied Loads with Load Factors

$P_{u\text{-comp}} = 75\,\text{kip}$
$P_{u\text{-uplift}} = 58\,\text{kip}$
$V_u = 14\,\text{kip}$
$M_u = 51\,\text{ft-kip}$

Calculate Load per Bolt

$$P_u = \frac{58\,\text{kip}}{4\,\text{bolt}} + \frac{51\,\text{ft-kip}\,(12''/\text{ft})}{14''(2\,\text{bolt})} = 14.5\,\text{kip} + 21.9\,\text{kip} = 36.4\,\text{kip}$$

$$V_u = \frac{14\,\text{kip}}{4\,\text{bolt}} = 3.5\,\text{kip}$$

$$A_a = \frac{P_u}{F_{dt}} = \frac{P_u}{\phi F_y} = \frac{36.4\,\text{kip}}{0.9(36\,\text{kip/in.}^2)} = 1.12\,\text{in.}^2\ \text{(controls)}$$

or

$$A_a = \frac{P_u}{0.8F_{ut}} = \frac{36.4\,\text{kip}}{0.8(58\,\text{kip/in.}^2)} = 0.78\,\text{in.}^2$$

$$A_v = \frac{V_u}{\phi F_y} = \frac{3.5\,\text{kip}}{(0.85)36\,\text{kip/in.}^2} = 0.11\,\text{in.}^2$$

$$A_s = A_a + A_v = 1.12\,\text{in.}^2 + 0.11\,\text{in.}^2 = 1.23\,\text{in.}^2$$

$$d = (2)\left(\frac{A_s}{\pi}\right)^{1/2} + \frac{0.974}{n}$$

Assume $n = 6$ threads/in.

$$d = 2\left(\frac{1.23\,\text{in.}^2}{\pi}\right)^{1/2} + \frac{0.974}{6}$$

$$d = 1.41\,\text{in.}$$

Use 1.5-in. diameter anchor bolt.

7.5.2 Base Plate on Leveling Nuts

Structure type: Line dead-end A-frame structure, 4 bolts on a 14-in. square bolt pattern, ASTM A36/A36M (2005) material.

Applied Loads with Load Factors

$P_{u\text{-comp}} = 75\,\text{kip}$
$P_{u\text{-uplift}} = 58\,\text{kip}$
$V_u = 14\,\text{kip}$
$M_u = 51\,\text{ft-kip}$

Assume base plate clearance above the top of the concrete (h) > 2 × the diameter of the bolt.

Calculate Load per Bolt

$$P_u = \frac{75\,\text{kip}}{4\,\text{bolt}} + \frac{51\,\text{ft-kip}\,(12''/\text{ft})}{14''(2\,\text{bolt})} = 18.8\,\text{kip} + 21.9\,\text{kip} = 40.6\,\text{kip}$$

$$V_u = \frac{14\,\text{kip}}{4\,\text{bolt}} = 3.5\,\text{kip}$$

$$A_a = \frac{P_u}{F_{dt}} = \frac{P_u}{\phi F_y} = \frac{40.6\,\text{kip}}{0.9(36\,\text{kip/in.}^2)} = 1.25\,\text{in.}^2$$

or

$$A_a = \frac{P_u}{0.8F_{ut}} = \frac{40.6\,\text{kip}}{0.8(58\,\text{kip/in.}^2)} = 0.88\,\text{in.}^2$$

Assume 2.25-in. anchor bolt (4.5 UNC threads, per ASTM A1554 (2004)):

$$h = 2(2.25\,\text{in.}) = 4.5\,\text{in.}$$

$$A_b = \left[\pi\left[\frac{5hV_u}{2\phi F_y}\right]^2\right]^{1/3} = \left[\pi\left[\frac{5(4.5'')(3.5\,\text{kip})}{2(0.9)(36\,\text{kip/in.}^2)}\right]^2\right]^{1/3} = 1.67\,\text{in.}^2$$

$$A_v = \frac{V_u}{\phi F_y} = \frac{3.5\,\text{kip}}{0.85(36\,\text{kip/in.}^2)} = 0.11$$

$$A_s = A_a + A_b + A_v = 1.25\,\text{in.}^2 + 1.67\,\text{in.}^2 + 0.11\,\text{in.}^2 = 3.03\,\text{in.}^2$$

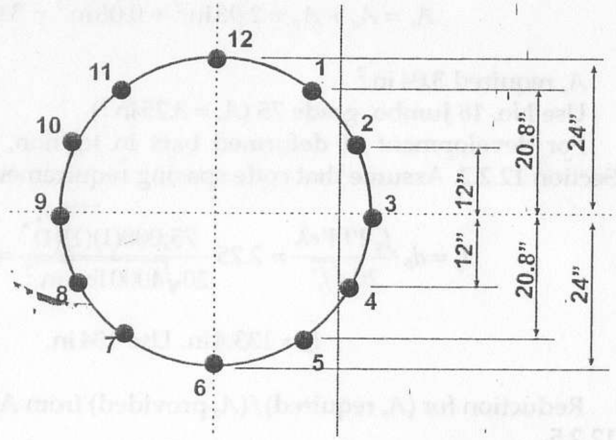
$$d = 2\left(\frac{A_s}{\pi}\right)^{1/2} + \left(\frac{0.974}{n}\right) = 2\left(\frac{3.03}{\pi}\right)^{1/2} + \left(\frac{0.974}{4.5}\right) = 2.18\,\text{in.}$$

Use 2.25 in. ϕ anchor bolt.

7.5.3 Base Plate on Leveling Nuts in a Drilled Pier

(Distance from the top of the concrete to the bottom of the baseplate (h) < 2 × the bolt diameter.)

Structure type: Single pole dead-end structure, 12 anchor bolts on a 48-in. bolt circle, ASTM A615/A615M (2006), Jumbo, grade 75 rebar ($F_y = 75\,\text{kip/in.}^2$, $F_{ut} = 100\,\text{kip/in.}^2$).



Applied Load with Load Factors

$P_u = 34\,\text{kip}$
$V_u = 46\,\text{kip}$
$M_u = 2380\,\text{ft-kip}$

Calculate Load per Bolt

$$P_u = \frac{34\text{ kip}}{12\text{ bolt}} = 2.83\text{ kip}$$

$$V_u = \frac{46\text{ kip}}{12\text{ blot}} = 3.83\text{ kip}$$

Maximum bolt load, from M_u

$$P_6 = P_{12} = 198\text{ kip}$$

$$P_{\text{total}} = 2.83\text{ kip} + 198\text{ kip} = 201\text{ kip}$$

$$A_n = \frac{P_u}{F_{dt}} = \frac{P_u}{\phi F_y} = \frac{201\text{ kip}}{0.9(75\text{ kip/in.}^2)} = 2.98\text{ in.}^2\text{ (controls)}$$

or

$$A_v = \frac{V_u}{\phi F_y} = \frac{3.83\text{ kip}}{0.85(75\text{ kip/in.}^2)} = 0.06\text{ in.}^2$$

$$A_n = \frac{P_u}{0.8F_{ut}} = \frac{201\text{ kip}}{0.8(100\text{ kip/in.}^2)} = 2.51\text{ in.}^2$$

$$A_s = A_n + A_v = 2.98\text{ in.}^2 + 0.06\text{ in.}^2 = 3.04\text{ in.}^2$$

A_s required 3.04 in.^{2}
Use No. 18 Jumbo, grade 75 ($A_s = 3.25\text{ in.}^2$)
For development of deformed bars in tension, use ACI 318 (2005), Section 12.2.2. Assume that code spacing requirements are met.

$$l_d = d_b \frac{f_y \Psi t \Psi e \lambda}{20\sqrt{f_c'}} = 2.25\frac{75{,}000(1)(1)(1)}{20\sqrt{4000\text{ lb/in.}^2}} = 133.4\text{ in.}$$

$$l_d = 133.4\text{ in. Use 134 in.}$$

Reduction for (A_s required)/(A_s provided) from ACI 318 (2005), Section 12.2.5.
A_s required = 3.25 in.^{2} (Use actual No. 18J bar A_s of thread area).
A_s provided = 4.00 in.^{2} (area of No. 18J bar).

$$l_d = 134\text{ in.}\left(\frac{3.25}{4.00}\right) = 109'' = 9'-1''\text{ embedment}$$

CHAPTER 8
QUALITY CONTROL AND QUALITY ASSURANCE

8.1 GENERAL

To ensure quality of the product, a good quality control (QC) and quality assurance (QA) program should be instituted by both the fabricator and purchaser throughout the entire production process. A QC/QA program will ensure the purchaser that the fabricator has the personnel, organization, experience, procedures, knowledge, equipment, capability, and commitment to produce the specified structure.

Quality control is the responsibility of the fabricator, and quality assurance is the responsibility of the purchaser. Quality control guidelines used by the fabricator should be clearly defined and available for review and approval by the purchaser. The purchaser should also specify any additional requirements to achieve the desired degree of quality. The QC/QA programs must be agreed on between the fabricator and the purchaser before the start of any fabrication. The extent of QC/QA programs may vary based on initial investigations, the purchaser's experience, the fabricator's experience, the fabricator's past performance, and the degree of reliability required for the specific job.

8.2 STEEL STRUCTURES

8.2.1 Material

The fabricator should review all the material that is used in the fabrication of the entire structure, all mill test reports for material compliance, all material suppliers for their manufacturing procedures and quality control programs, and all welding electrodes.

The purchaser should review and agree on the fabricator's material specifications, supply sources, material identification, storage, traceability procedures, and certified mill test reports.

Typical materials used for steel structures include ASTM A36/A36M (2005), ASTM A53/A53M (2006), ASTM A500 (2003), ASTM A572/A572M (2006), ASTM A588/A588M (2005), and ASTM A871/A871M (2003). Connection materials commonly used include ASTM A193/A193M (2006), ASTM A307 (2004), ASTM A325 (2006), ASTM A325M (2005), ASTM A394 (2005), ASTM A449 (2004), and ASTM F1554 (2004).

For additional information on recommended materials, refer to the appropriate sections of ASCE/SEI 48 (2005) and ASCE 10 (1997).

8.2.2 Welding

The fabricators' welding procedures and welder certification programs should be in accordance with the latest revision of the American Welding Society's Structural Welding Code for Steel (AWS D1.1/D1.1M 2006).

The purchaser should establish requirements for the review of and agreement on the fabricators' welding procedures for various types of welds and their welding certification programs.

8.2.3 Fabrication Inspection

8.2.3.1 Visual Inspection. Visual inspections address the following typical areas:

- dimensional correctness,
- fabrication straightness,
- cleanliness of cuts and welds,
- surface integrity at bends,
- condition of punched and drilled holes,
- hardware fit and length,
- weld size and appearance, and
- overall workmanship.

8.2.3.2 Specific Inspection Methods of Welds. Several methods of nondestructive testing may be used to detect weld flaws. They include the following:

- Magnetic particle testing (MT) is a practical method for detecting tight surface cracks. MT inspection should be in accordance with ASTM E709 (2001).

- Dye penetrant testing (PT) is a reliable method for detecting ar cracks or porosity that are open to the test surface. PT inspectic should be in accordance with ASTM E165 (2002).
- Ultrasonic testing (UT), using longitudinal angle beams, is the pr ferred method of determining weld quality in base plates and flang connection welds. It is also reliable in detecting small cracks an internal flaws in other complete penetration welds. UT can also b used to verify that the specified percentage of penetration on partia penetration welds has been achieved. AWS D1.1/D1.1M (2006) doe not provide any specific guidelines for ultrasonic testing of plate less than 5/16 in. (8 mm) thick or for welds using backing bars. Th fabricator should follow the procedure established by AWS D1.1 D1.1M (2006) in developing a specific inspection procedure. U inspection should be in accordance with ASTM A435/A435M-9 (2001) and ASTM E164 (2003).
- Radiographic testing (RT) provides a permanent record of the tes results. However, its use is limited on many of the weld types (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 possibl to miss tight cracks that lie normal to the RT source and film.
- Eddy current testing (ET) techniques have limited application in th determination of weld penetration and the detection of cracks.

8.2.3.3 Test Assembly. The full or partial assembly of a complicated structure by the fabricator before galvanizing and final shipment may be beneficial in verifying design and detailing correctness. Field construction problems and delays can be minimized if a test assembly of the structure reveals any errors. Missing or mispunched holes can be easily corrected at the fabrication shop instead of causing construction delays if left undone until final assembly.

8.2.4 Structure Coating

Where painting is required, the system, procedures, and methods of application should be acceptable to both the purchaser and the fabricator. Also, the system should be suitable for both the product and its intended exposure. The coating thickness should be checked to ensure that the minimum dry-film thickness meets both the purchaser's specification and the paint supplier's specification. In addition, a thorough visual inspection should be made to detect pinholing, cracking, and other undesirable characteristics.

Where galvanizing is required, the procedure and facilities should be agreed on by the purchaser and the fabricator. After hot-dip galvanizing, nondestructive testing may be specified to ensure that there have been no adverse changes to the finished product. The magnetic thickness measurement method is usually the test that is used to determine the thickness of the zinc coating.

Where metallizing is required, the procedures and facilities should be in accordance with the coating supplier's recommendations and acceptable to both the fabricator and purchaser. The metallized coating should be inspected for thickness by a magnetic thickness gauge. Also, an adherence test may be made by cutting through the coating with a knife. Bond is considered unsatisfactory if any part of the coating lifts away from the base metal 0.25 in. (6.35 mm) ahead of the cutting blade.

Where bare weathering steel is specified, the need for blast cleaning the steel should be decided and agreed on by the purchaser and fabricator. Blast cleaning is desirable if a clean and uniformly weathered appearance is important in the structure's first years of exposure. In time, even a nonblast-cleaned steel structure develops a uniform oxide coating.

8.3 ALUMINUM STRUCTURES

Many of the requirements, procedures, testing, and handling associated with steel structures are also applicable to aluminum structures.

8.3.1 Material

The purchaser should review and agree on the fabricator's material specifications, supply sources, material identification, storage, traceability procedures, and certified mill test reports.

8.3.2 Welding

Quality control for welded connections is most critical. Accordingly, care in the welding of these joints should be the highest priority. Welding specifications, preparation, procedures, and welder qualifications and inspection should be in accordance with the Aluminum Association (1981) and AWS D1.2/D1.2M (2003).

8.3.3 Fabrication

The fabricator should follow the guidelines listed below:

- Shearing, sawing, and arc cutting are acceptable methods of cutting. Flame cutting is not acceptable.
- Holes may be punched if the taper of the hole does not exceed a diametric difference of 0.03125 in. (0.80 mm). Otherwise, subpunching, reaming, or drilling is required.

- All forming and bending should be carried out cold, unles indicated on the shop drawings or specially approved by the desig engineer.

8.3.4 Inspection

At a minimum, visual inspection of all welded joints is necessary, an additional testing of welded joints, if required, should be agreed on by th purchaser and fabricator.

8.3.5 Structure Coating

Aluminum structures are normally supplied with a standard mill finis (i.e., not painted or deglared). If the structures are to be provided deglared the purchaser and fabricator should resolve the type of treatmen allowable gloss, and other requirements. Aluminum structures are no ordinarily painted, except in extremely corrosive conditions or if i contact with dissimilar materials, such as steel. If required, paintin should be performed in accordance with the paint manufacturer' recommendations.

8.4 CONCRETE STRUCTURES

For concrete poles, the material should satisfy the requirements a specified by ASCE (1987), Section 3.8 and Chapter 6.0.

8.4.1 Reinforced Concrete

For typical concrete structures, concrete proportion and mix desig should satisfy the durability requirements as specified in Chapter 4 of AC 318 (2005). Reinforced concrete structures should also comply with AC 318 (2005) Section 7.7 for the minimum concrete cover requirements and Chapter 5 for quality and placement criteria.

Under special conditions where corrosion of reinforcement bars is possible, a cathodic protection or protective epoxy coating is recommended to ensure the long-term integrity of the concrete elements.

8.4.2 Prestressed Concrete

For prestressed concrete structures, the material should satisfy the requirements as specified by PCI MNL-116 (1999).

8.5 WOOD STRUCTURES

8.5.1 Material and Treatment

The material and treatment of wood members should satisfy the requirements of Section 10 in IEEE 751 (1991), using the applicable standard listed under Section 10.2.2.

8.5.2 Manufacturing and Fabrication

The manufacturing and fabrication of wood members should satisfy the requirements of Section 10.1.4 of IEEE 751 (1991). Any special fabrication details or tolerances should be included by the purchaser in a purchase specification.

8.5.3 Inspection

The inspection of wood members should satisfy the requirements of AITC 200 (2004).

8.6 SHIPPING

At a minimum, the fabricator should comply with the shipping procedures as listed below:

- Check packaging to minimize shipping damage.
- Check items to ensure that they have completed specified inspections.
- Check that specified items are included with the shipment.

Before the start of fabrication, the purchaser should review the fabricator's methods and procedures for packaging and shipping and agree to the mode of transportation. When receiving materials, all products should be inspected for shipping damage before accepting delivery. If damage is apparent, the purchaser should immediately notify the delivering carrier. If the shipments are fee on board (FOB) destination, the shipper is responsible for damage repair. Therefore, the purchaser should notify the fabricator of any damage and then cooperate in filing damage claims with the carrier.

When receiving materials, the purchaser is also responsible for checking to see that all materials listed on the packing lists were delivered. Where a discrepancy exists, both the carrier and the fabricator should be notified.

8.7 HANDLING AND STORAGE

The fabricator should provide written procedures for handling an storing materials to prevent damage, loss, or deterioration of the structur The purchaser should review and approve these procedures before th shipment of any materials.

At a minimum, stored material should be placed on skids, platforms, c other supports above the ground and away from any vegetation. Decaye or decaying material supports should not be permitted to remain unde stored material. Special care should be exercised while storing materia Materials such as steel and aluminum stored within an energize substation, on nonconducting material, can develop an induced electrica potential, and proper precautions should be used for worker safety. Prope initial placement of material sections can increase the efficiency of the fina assembly process. Material identification marks should be visible whe the material is stacked and should remain legible for the period of tim specified by the purchaser.

CHAPTER 9

TESTING

Full-scale structural proof tests are rarely performed on substatio structures that support wires. It is not cost-effective to perform a full-scal test because substation structures, unlike transmission structures, are no fabricated in large quantities. Full-scale testing should be considered if particular substation structure is a standard and will be used in larg quantities or if the structure uses a unique structural system not typical o current practice. Component testing (e.g., a section of the tower o connections) may be cost-effective for substation structures. If componen or full-scale tests are required, they should comply with the following standards and manuals: ASCE 10 (1997), ASCE/SEI 48 (2005), and ASCI (1987).

Seismic response (dynamic loading) requires that the support structure and equipment be seismically tested and evaluated as a system. Seismic tests are performed in accordance with IEEE 693 (2005).

CHAPTER 10
CONSTRUCTION AND MAINTENANCE

10.1 CONSTRUCTION

The design engineer should anticipate construction loads imposed c substation structures and ensure that proper construction methods an quality materials are used to prevent undue stresses. Outdoor substatior are frequently erected by people with varied levels of experience a structural erectors. Therefore, simplicity of design, details, and erectior with a specific schedule of inspections should be considered. Inspectior should be during but not limited to grading, foundation placemen structure erection, and structure loading.

10.2 MAINTENANCE

The design engineer should consider accessibility of equipment fc maintenance and operation. Equipment should have provisions for acces: i.e., crossover platforms or working platforms. These access provision need to be considered, especially around large transformers with coolan and fire protection piping.

Structures and foundations should be inspected a minimum of eacl time the equipment being supported is inspected or maintained. Thes structures should be checked for damage, corrosion, loose members, anc connection bolts. Also, check for settlement, cracking, and deterioration o foundations. In addition, when equipment is upgraded, structures shoulc be carefully checked for structural adequacy, electrical clearances, anc deterioration due to corrosion. When possible, the interior of hollow tubular members should be periodically inspected for corrosion.

Structures painted before 1975 were typically painted using lead-based paint. All structures that are painted should be checked for lead-based paint. If there is lead, then all OSHA and Environmental Protection Agency (EPA) requirements should be followed when removing the paint.

Weathering steel should be inspected periodically for abnormal oxidation at the member connections.

10.3 WORKER SAFETY

All structures and equipment that will be inaccessible with bucket trucks or small ladders should be considered for climbing devices mounted to the structure with fall protection devices. Elevated locations on the structure that require workers to move from one location to another (as defined in IEEE 1307 2004), either during construction or for maintenance, may require fall protection devices, such as safety cables for attachment to workers. In all cases, OSHA and local codes should be adhered to, especially in energized substations. IEEE 1307 (2004) is one source of information for worker safety during climbing of utility structures.

REFERENCES

Abild, J., Andersen, E. Y., and Rosbjerg, L. (1992). "The Climate of Extrem Wind at the Great Belt, Denmark." *J. Wind Engrg. Industrial Aerody* 41(1–3): 521–532.

Aluminum Association. (1981). "Aluminum Construction Manual," Th Aluminum Association, Arlington, Va.

Aluminum Association. (2005). "Aluminum Design Manual, Includin Specification for Aluminum Structures—Allowable Stress Design an Specification for Aluminum Structures—Load Resistance Fact Design," The Aluminum Association, Arlington, Va.

American Concrete Institute (ACI) 318. (2005). "Building Cod Requirements for Structural Concrete (with Commentary)," AC Committee Publication No. 318–05, American Concrete Institut Detroit, Mich.

ACI 349. (2001). "Code Requirements for Nuclear Safety Related Concret Structures and Commentary," Appendix B with Commentary, AC Committee Publication No. 349–01, American Concrete Institut Detroit, Mich.

ACI 355.1. (1997). "Report on Anchorage to Concrete," ACI Committe 355.1, American Concrete Institute, Detroit, Mich.

ACI 440.1R. (2006). "Guide for the Design and Construction o Concrete Reinforced with FRP Bars," American Concrete Institut Detroit, Mich.

American Galvanizers Association. (1984). "Recommended Details fo Galvanized Structures," Aurora, Colo.

American Galvanizers Association. (2001). "The Design of Products To Be Hot Dip Galvanized after Fabrication," Aurora, Colo.

American Institute of Steel Construction (AISC). (2003). "Manual of Steel Construction: Load and Resistance Factor Design, 3rd Ed., American Institute of Steel Construction, Chicago, Ill.

AISC (2005). "ASD/LRFD Steel Construction Manual," 13th Ed., American Institute of Steel Construction, Chicago, Ill.

American Institute of Timber Construction (AITC) 200. (2004). "Manufacturing Quality Control Systems Manual," American Institute of Timber Construction, Englewood, Colo.

American National Standards Institute (ANSI)-O5.1. (2002). "American National Standard, Wood Poles—Specifications and Dimensions," New York.

ANSI-O5.1c. (2004). "American National Standard, Wood Poles—Specifications and Dimensions," Supplement to ANSI O5.1 (2002), New York.

American Society of Civil Engineers (ASCE). (1961). "Wind Forces on Structures," *Transactions*, Vol. 126, Paper No. 3269, pp. 1124–1197.

ASCE. (1972). "Guide for the Design of Aluminum Transmission Towers," *J. Struct. Div.* 98(ST12).

ASCE. (1987). "Guide for the Design and Use of Concrete Poles," American Society of Civil Engineers, New York.

ASCE. (1997). "Wind Loads and Anchor Bolt Design for Petrochemical Facilities," American Society of Civil Engineers, New York.

ASCE 7. (2005). "Minimum Design Loads for Buildings and Other Structures," American Society of Civil Engineers, New York.

ASCE 10. (1997). "Design of Latticed Steel Transmission Structures," American Society of Civil Engineers, Reston, Va.

ASCE 74. (1991). "Guidelines for Electrical Transmission Line Structural Loading," American Society of Civil Engineers, Reston, Va.

ASCE 91. (1997). "Design of Guyed Electrical Transmission Structures," American Society of Civil Engineers, New York.

ASCE-PCI. (1997). "Guide for the Design of Prestressed Concrete Poles," *PCI J.*, 24(6): 94–134.

ASCE/SEI 48. (2005). "Design of Steel Transmission Pole Structures," American Society of Civil Engineers, Reston, Va.

American Society for Testing and Materials (ASTM) A36/A36M. (2005). "Standard Specification for Carbon Structural Steel," American Society for Testing and Materials, West Conshohocken, Pa.

ASTM A53/A53M. (2006). "Standard Specification for Pipe, Steel, Black and Hot-Dipped Zinc-Coated, Welded and Seamless," American Society for Testing and Materials, West Conshohocken, Pa.

ASTM A123/A123M. (2002). "Standard Specification for Zinc Coating (Hot-Dip Galvanized) on Iron and Steel Products," American Society for Testing and Materials, West Conshohocken, Pa.

ASTM A153/A153M. (2005). "Standard Specification for Zinc (Hot-Dip) on Iron and Steel Hardware," American Society for Testing and Materials, West Conshohocken, Pa.

ASTM A193/A193M. (2006). "Standard Specification for Alloy-Steel a Stainless Steel Bolting Materials for High Temperature or High Pressu Service and Other Special Purpose Applications," American Society Testing and Materials, West Conshohocken, Pa.

ASTM A242/A242M. (2004). "Standard Specification for High-Streng Low-Alloy Structural Steel," American Society for Testing a Materials, West Conshohocken, Pa.

ASTM A307. (2004). "Standard Specification for Carbon Steel Bolts a Studs, 60,000 PSI Tensile Strength," American Society for Testing a Materials, West Conshohocken, Pa.

ASTM A325. (2006). "Standard Specification for Structural Bolts, Ste Treated, 120/105 ksi Minimum Tensile Strength," American Society f Testing and Materials, West Conshohocken, Pa.

ASTM A325M. (2005). "Standard Specification for Structural Bolts, Ste Heat Treated 830 MPa Minimum Tensile Strength (Metric)," Americ Society for Testing and Materials, West Conshohocken, Pa.

ASTM A394. (2005). "Standard Specification for Steel Transmission Tow Bolts, Zinc-Coated and Bare," American Society for Testing a Materials, West Conshohocken, Pa.

ASTM A435/A435M-90. (2001). "Standard Specification for Straight-Bea Ultrasonic Examination of Steel Plates," American Society for Testi and Materials, West Conshohocken, Pa.

ASTM A449. (2004). "Specification for Hex Cap Screws, Bolts and Stuc Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, Gener Use," American Society for Testing and Materials, West Conshohocke Pa.

ASTM A500. (2003). "Standard Specification for Cold-Formed Weld and Seamless Carbon Steel Structural Tubing in Rounds and Shapes American Society for Testing and Materials, West Conshohocke Pa.

ASTM A572/A572M. (2006). "Standard Specification for High-Streng Low-Alloy Columbium-Vanadium Structural Steel," American Socie for Testing and Materials, West Conshohocken, Pa.

ASTM A588/A588M. (2005). "Standard Specification for High-Streng Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Poi to 4 in. [100 mm] Thick," American Society for Testing and Material West Conshohocken, Pa.

ASTM A615/A615M. (2006). "Standard Specification for Deformed an Plain Carbon-Steel Bars for Concrete Reinforcement," American Socie for Testing and Materials, West Conshohocken, Pa.

ASTM A706/A706M. (2006). "Standard Specification for Low-Alloy Ste Deformed and Plain Bars for Concrete Reinforcement," America Society for Testing and Materials, West Conshohocken, Pa.

ASTM A871/A871M. (2003). "Standard Specification for High Strengt Low-Alloy Structural Steel Plate with Atmospheric Corrosio

Resistance," American Society for Testing and Materials, West Conshohocken, Pa.

ASTM B695. (2004). "Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel," American Society for Testing and Materials, West Conshohocken, Pa.

ASTM E164. (2003). "Standard Practice for Ultrasonic Contact Examination of Weldments," American Society for Testing and Materials, West Conshohocken, Pa.

ASTM E165. (2002). "Standard Test Method for Liquid Penetrant Examination," American Society for Testing and Materials, West Conshohocken, Pa.

ASTM E709. (2001). "Standard Guide for Magnetic Particle Examination," American Society for Testing and Materials, West Conshohocken, Pa.

ASTM F1554. (2004). "Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength," American Society for Testing and Materials, West Conshohocken, Pa.

American Welding Society (AWS) D1.1/D1.1M. (2006). "Structural Welding Code—Steel," American Welding Society, Miami, Fl.

AWS D1.2/D1.2M. (2003). "Structural Welding Code—Aluminum," American Welding Society, Miami, Fl.

Bayar, D. C. (1986). "Drag Coefficients of Latticed Towers," *J. Struct. Div., ASCE*, 112(2), 9–167.

Beedle, L. S. (1958). *Plastic Design of Steel Frames*. John Wiley & Sons, New York.

Bleich, F. (1952). *Buckling Strength of Metal Structures*, 1st Ed., McGraw-Hill Book Co., New York.

Blodgett, O. W. (1966). "Design of Welded Structures," The James F. Lincoln Arc Welding Foundation, Cleveland, Ohio.

British Electrical and Allied Industries Research Association (BEAIRA). (1935). "Wind Pressure on Latticed Towers—Tests on Models, Report (F/T 84)," *J. Inst. Electrical Engrs.* 77, 189–196.

Cook, R. D. (1974). *Concepts and Applications of Finite Element Analysis*, John Wiley & Sons, New York.

Davenport, A. G. (1979). "Gust Response Factors for Transmission Line Loading," Proceedings of the Fifth International Conference of Wind Engineering, Pergamon Press, July, Fort Collins, Colo., 866–909.

Deaves, D. M., and Harris, R. I. (1978). "A Mathematical Model of the Structure of Strong Winds," United Kingdom Construction Industry Research and Information Association, Report 76, London, England.

Federal Emergency Management Agency (FEMA) 450. (2004). "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures," 2003 Ed., Federal Emergency Management Agency, Building Seismic Safety Council, National Institute of Building Sciences, Washington, D.C.

Federal Specification TT-P-645B. (1990). "Primer, Paint, Zinc-Molybda Alkyd Type," March, http://www.wbdg.org/ccb/FEDMIL/TT64 pdf.

General Order 95. (2006). "State of California Rule for Overhe Electric Line Construction," Public Utility Commission, Sacramen Calif.

Gere, J. M., and Carter, W. O. (1962). "Critical Buckling Loads for Taper Members," *J. Struct. Div., ASCE*, 88(ST1), 1–11, Proc. Paper 3045.

Hoskings, J. R. M., and Wallis, J. R. (1987). "Parameters and Quant Estimation for the Generalized Pareto Distribution," *Technometr* 29(3): 339–349.

Institute of Electrical and Electronics Engineers (IEEE) C37.04a. (200 "Standard Capacitance Current Switching Requirements for Hi Voltage Circuit Breakers," The Institute of Electrical and Electron Engineers, Piscataway, N.J.

IEEE C37.32. (2002). "American National Standard High-Voltage A Disconnect Switches, Interrupter Switches, Fault Initiating Switch Grounding Switches, Bus Supports, and Accessories Control Volta Ranges—Schedules of Preferred Ratings, Construction Guidelines a Specifications," The Institute of Electrical and Electronics Enginee Piscataway, N.J.

IEEE C57.19.01. (2000). "Standard Performance Characteristics a Dimensions for Outdoor Apparatus Bushings," The Institute Electrical and Electronics Engineers, Piscataway, N.J.

IEEE C57.19.100. (1995). "Guide for Application of Power Apparat Bushings," The Institute of Electrical and Electronics Enginee Piscataway, N.J.

IEEE C57.32. (2002). "American National Standard for Switchgear—Hi Voltage Air Switches, Bus Supports, and Switch Accessories—Schedu of Preferred Ratings Manufacturing Specifications, and Applicati Guide," The Institute of Electrical and Electronics Engineers, Piscatawa N.J.

IEEE 605. (2006). "Guide for Design of Substation Rigid-Bus Structure Draft 10, The Institute of Electrical and Electronics Enginee Piscataway, N.J.

IEEE 691. (2001). "Guide for Transmission Structure Foundation Desi and Testing," The Institute of Electrical and Electronics Enginee Piscataway, N.J.

IEEE 693. (2005). "Recommended Practice for Seismic Design Substations," The Institute of Electrical and Electronics Enginee Piscataway, N.J.

IEEE 751. (1991). "Trail-Use Design Guide for Wood Transmissi Structures," The Institute of Electrical and Electronics Enginee Piscataway, N.J.

IEEE 1307. (2004). "Fall Protection of the Utility Industry," The Institute of Electrical and Electronics Engineers, Piscataway, N.J.

IEEE 1527. (2006). "Recommended Practice for the Design of Flexible Buswork Located in Seismically Active Areas," The Institute of Electrical and Electronics Engineers, Piscataway, N.J.

Jones, K. F. (1996a). "A Simple Model for Freezing Rain Load," Proceedings, Seventh International Workshop on Atmospheric Icing of Structures, June, University of Quebec, 412–416. Chicoutimi, Quebec, Canada.

Jones, K. F. (1996b). "Ice Accretion in Freezing Rain," CRREL Report 96–2, Cold Regions Research and Engineering Laboratory, Hanover, N.H.

Kissell, J. R., and Ferry, R. L. (1995). *Aluminum Structures, A Guide to Their Specifications and Design*, John Wiley & Sons, New York.

MacNeal, R. H., and Harder, R. C. (1985). "A Proposed Standard Set of Problems to Test Finite Element Accuracy, *Finite Elements in Analysis and Design* 1(1): 3–20, MSC Software Corporation, Santa Ana, Calif.

Mooers, J. D. (2006). "Aluminum Substation Structures," Proceeding ASCE Electrical Transmission Line and Substation Structures Conference, Birmingham, Ala., 137–148.

Mueller, W. H., Ostendorp, M., and Kempner, L. (1991). "LIMIT: A Nonlinear Three Dimensional Truss Analysis Program," Technical Research Report to Bonneville Power Administration, Portland, Oreg.

National Design Specification (NDS). (2005). "National Design Specification for Wood Construction," American Forest and Paper Association, Washington, D.C.

National Electric Safety Code (NESC). (2007). "National Electric Safety Code (ANSI C2)," The Institute of Electrical and Electronics Engineers, Piscataway, N.J.

Norris, C. H., and Wilbur, J. B. (1960). *Elementary Structural Analysis*, 2nd Ed., McGraw-Hill Book Company, New York, 277.

PCI MNL-116. (1999). *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*, 4th Ed., Precast/Prestressed Concrete Institute, Chicago, Ill.

PCI MNL-120. (2004). *Design Handbook, Precast and Prestressed Concrete*, 6th Ed., Precast/Prestressed Concrete Institute, Chicago, Ill.

Peterka, J. A. (1992). "Improved Extreme Wind Prediction for the US," *J. Wind Engrg. Industrial Aerodyn.* 41(1–3): 533–541.

Scruton, C. (1963). "On the Wind-Excited Oscillations of Stacks, Towers, and Masts," Proceedings of the First International Conference on Wind Effects on Buildings and Structures, London, England, 798–837.

Wang, Q. J. (1991). "The POT Model Described by the Generalized Pareto Distribution with Poisson Arrival Rate," *J. of Hydrology*, 129, 263–280.

White, D. W., and Hajjar, J. F. (1991). "Application of Second Order Elastic Analysis in LRFD: Research to Practice," *Engrg. J.* 28(4), 133–148.

INDEX